Comparison of the Performance of Prestressed Ribbed Panels and Hollow Core Panels Supported on Four-Edges

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Abstract. Tests of full-scale models of the precast slab with dimensions of 6.30 x 6.30 m, built of two different precast panels, were carried out under short-term load. The models were made of autoclaved aerated concrete (700 type) brick with a thickness of 240 mm and a height of 2.24 m. The slabs were supported at four edges. The first slab was precast prestressed ribbed panels with concrete overtopping. The second slab was made of prestressed hollow-core panels. The panels had the same modular width of 600 mm. Tests were carried out under load placed on the top of the slab. The short-term load was applied sequentially, and displacement measurements were measurement by the electronic method. The load was initially applied evenly distributed. In the last step, part of the load was transferred to one-half of the slabs. The obtained load was different for each half of the slab. The first part of the slab were panels 1 to 5, loaded with the value of 1.7 kN/m², and the second part was panels 6 to 10 loaded with the value of 7.7 kN/m². The tests allow determining the difference in slabs' performance depending on shear key construction. The panels maintained the possibility of load redistribution based on their interaction despite the longitudinal joints' work only through the concrete cross-section. The slabs had a different character of transverse displacements depending on the presence of concrete topping. The models revealed a different response to transferring part of the load to one-half of the slabs. There were no cracks in the line of longitudinal joints on the upper surface of the slabs. Also, there were no cracks on the bottom of the panels. At the panels' connection with the wall, rotation and lifting corners of the slabs were noticed. The measured displacements were significantly smaller than for the corresponding models of single-span slabs with a parallel load.

1. Introduction
Precast panel slabs are commonly used as an alternative to monolithic or beam and block slabs. The result is an increasing share in residential and office building floor solutions [1]. Panel slabs consist of a (flat or ribbed) reinforced concrete or prestressed panel in the form of a stay-in-place formwork with the main reinforcement and concrete overtopping.

Recent years have brought research of a composite slab with vertical tongues served as shear key, ensuring the connection of two layers of the slab [2], as well as one-way or two-way slabs on thin prestressed boards (narrow boards) [3]. Various methods of pretensioned have been used, including the basalt bars [4]. There are attempts to use reinforcement in the form of trusses, increasing the load capacity of the interface (bond) at the multi-rib prestressed precast element with the concrete topping - Joint Advanced Slab System (JAS) [5]. Precast elements are also designed with a variable cross-section along panel - Optimized-section Precast Slab (OPS) [6]. Studies of full-size slabs containing more than one precast element were carried out sporadically and mainly concerned with the shape of a longitudinal
joint (shear-key) [7], [8]. An interesting aspect was the transverse reinforcement passed through the panels’ ribs [9] (inverted multi-ribs panel). Despite the many years of using composite slabs as single, one-way (beam) working elements, no tests have been carried out in the conditions of uniform loads. There are no significant comparative studies of large scale short-term and long-term research with a scheme that forces two-way performance. There is still a considerable lack of detailed guidelines to consider the effects of two-way dedicated to precast elements [10], [11]. Indirect information is provided in the standard [12] of hollow-core slabs, which gives the formula for the shear resistance of the transverse joint and introduces a calculation model for the distribution of loads between individual panels. For the purpose to fill the gap, tests of precast panel slabs were carried out. The main aspect was the transverse performance and cooperation of panels. The research was performed on the types of precast panel slabs commonly used in Europe (in single-family, multi-family and office buildings).

2. Description of the research
The purpose of the research was to observe the prestressed ribbed panel’s performance and hollow-core panel slabs under short-term loads. Presented experimental research is a development of already carried out tests on the slab panels without prestressing [13], [14]. The length of the precast unit equal to 60 cm and no transverse reinforcement providing the 2-way behaviour of the slabs were characteristic examined models. The testing aimed to evaluate the behaviour of slabs (qualitative tests). Slabs of prestressed precast ribbed panels (RP) and hollow-core slabs (HC) were examined. Lateral forces were transferred between the panels through concrete topping or the shear-key structure at the HC slab. Material parameters of concrete and steel are presented in tables 1 and 2.

Table 1. Properties of concrete in the models made as specified in the standards [15]–[18]

| Model | Precast concrete | Topping slab concrete |
|-------|------------------|-----------------------|
|       | $f_{c,y}$, N/mm$^2$ | $E_c$, N/mm$^2$ | $f_{c,y}$, N/mm$^2$ | $E_c$, N/mm$^2$ |
| RP    | 56.2              | 36536                | 36.3                | 31815               |
| HC    | 56.0              | 36459                | 40.8                | 32764               |

$f_{c,y}$ – mean compressive strength of Ø150×300 mm cylinders,
$E_c$ – mean elastic modulus of concrete,

Table 2. Properties of reinforcement steel and prestressing steel in the main rebars used in the test model and determined following the standards [19]–[21]

| Model | Steel grade | $R_{eli}$, N/mm$^2$ | $R_m/R_e$ | $A_{gt}$, % |
|-------|-------------|---------------------|-----------|-------------|
| RP    | Y2060S7     | 1927                | 1.12      | 5.4         |
| HC    | Y1860S7     | 1756                | 1.14      | 6.1         |

$R_{eli}$ – mean upper elastic limit,
$R_m$ – tensile strength,
$A_{gt}$ – overall ductility percent at the greatest force

2.1 Construction of the research model
Models were prepared as slabs with the dimensions of ~6.30 × 6.30 m supported on four masonry walls. The models were made of concrete masonry units with a thickness of 240 mm and a height of 2.20 m (figure 1) and second model on autoclaved aerated concrete (700 type) brick with a thickness of 240 mm and a height of 2.24 m. Each model contained ten panels supported on precast bond beam blocks. Two door openings of 1.5 m and 1.8 m in width were performed in the building model to provide access to the slab’s bottom part. The openings were covered with precast prestressed lintel beams, over which bond beam blocks were directly placed.
2.1.1 Model 1 – Prestressed Ribbed Panel
The ribbed panel (RP) model was made of precast prestressed panels composed of the lower concrete slab with a thickness of 40 mm and a width of 600 mm, and two upward longitudinal ribs with a height of 120 mm at the centre-to-centre spacing of 350 mm – figure 2. The prestressed reinforcement of ribs consisted of 7-strand tendons of $\phi 6.85$ mm ($1 \times \phi 2.24$ mm + $6 \times \phi 2.40$ mm) from steel Y2060S7. Voids between ribs in each precast unit were filled with lightweight concrete of 80 mm in thickness. Then, the concrete topping of 40 mm in thickness was laid after erecting the members. The overall thickness of the slab structure was 160 mm. The top reinforcement (with the range of 100 cm) in the mesh with welded rebars with the diameter $\phi 4$ mm and the spacing of 200 mm was placed at supports. Two transverse sets of double rebars of $\phi 10$ mm laid in the concrete topping on the rib surface were shaped towards the short direction to ribs in the precast units. The deadweight of the slab was 3.15 kN/m$^2$.
2.1.2. Model 2 – Hollow Core Panel

Hollow-core (HC) model was prepared from hollow-core slabs with a width of 600 mm and a thickness of 150 mm – Figure 3. Each slab was prestressed from the bottom with four tendons, composed of seven strands $\phi 9.3$ mm (1x3.17 mm; 6x3.08 mm) and two tendons $\phi 6.85$ mm (1x2.40 mm; 6x2.24 mm) in the top part of the section, made of Y1860S7 steel. The thickness of the concrete cover of top and bottom strands was equal to 35 mm. The concrete topping of the precast units was not used intentionally, and only the slab joints were in-situ filled with concrete. At the supports, a rebar of 8 mm in diameter was laid in each joint filled with concrete. Each rebar was placed in the upper part of the shear key. The deadweight of the slab was 2.65 kN/m$^2$.

2.2 Measuring equipment

The displacements were recorded using linear displacement transducers (LVDT) type PJX-10 and PJX-20 with an indication accuracy of 0.002 mm. The sensors were attached to a steel frame based on a reinforced concrete floor. Figure 4 shows the sensors’ arrangement along the door openings axis and the central panels’ joint (panels 5 and 6). The sensors were placed ~ 25 mm from the panel joint, the distance between adjacent sensors was ~ 50 mm. Additional markers for geodetic measurements were located from the bottom of the slab, next to the electronic sensors. Manual measurements were performed as a control read and left for long-term studies.
Figure 4. Arrangement of the transducer and geodetic sensors for measuring vertical displacements on the lower surfaces of the tested slabs: 1 - plate number, 2 - sensor number, 3 - measurement direction transverse to the main direction of the panels (Y-axis), 4 - measurement direction along the length of the central panels joint (X-axis)

2.3 Application method and load schedule
The research model was loaded gravitationally, according to a schedule. Single concrete blocks (350x250x120 mm) and blocks stacked on pallets were used to induce loads. The total load over the own slab weight of 4.7 kN/m² was divided into two parts 1.7 kN/m² (concrete blocks laid on the upper surface of the slab) and 3.0 kN/m² (pallets with concrete blocks laid on previously laid concrete blocks). The load application schedule is shown in figure 5. Automatic displacements registering was made after 15 minutes from the moment the load is placed. Schemes F and L were a continuation of previous schemes E and K, with the displacement registering taken after an hour's break. Scheme L has been left for long term studies. Long-term deflection and faulting tests are scheduled to proceed for 12 months. The description of long-term studies is not part of this article.

Figure 5. Research model: a) LVDT and geodetic sensors, b) Scheme L-12
3. Results and discussions

The research results in the form of graphs showing the deflection of the sensors indicated in figure 4 for situations shown in figure 7 and figure 8. During each stage, the possible cracks of the bottom and top surfaces of the slab was monitored. Throughout every scheme, no cracks were noted on the slabs.

For each of the stages, significantly lower displacement of the slab made of hollow-core panels was recorded. For schemes 8 and 10, the difference was 15% and 20%, and for the last scheme - 8%. These measurements were recorded despite the RP model's higher stiffness (greater thickness of the slab) and the lack of visible cracks. The maximum vertical displacement was 3.90 mm for the RP model and 3.57 mm for the HC model (results in Table 3). The most significant differences were noted in the behaviour of the slab displacement in the transverse direction. The HC slab, despite smaller deflections, had characteristics indicating a weaker influence of transverse cooperation within the entire structure. This model recorded the greatest displacement in half of the max loaded zone, up to the value of 7.7 kN/m². Different results were recorded in the RP model, where the greatest displacement took place in the middle of the slab spans. The different degree of cooperation is noticeable by comparing the displacement characteristics in steps 10 and 12. Even though half of the load was transferred to one part of the slab (panels no. 6 to 10), the RP model recorded only a 5% reduction of displacement in the second half (panels no. 1 to 5). This proves that the loads are transferred through the entire slab. A more negligible effect of cooperation was recorded in the HC model, where the reduction was 22%, and a more significant increase was recorded in the directly loaded zone. The RP slab's displacement curve was similar to the monolithic slab's, even without transverse reinforcement. The HC model's displacement shows higher rotation values in the joint directly below the maximum load. The curve takes the shape of two lines with one breakpoint. The RP model saw a rotation at each node without...
a single dominant breaking point. For this model, smaller displacement was noted at length of 120 cm from the supports than for the HC model.

![Displacement on the Y-axis: a) RP model, b) HC model, c) RP and HC J-10 scheme, d) RP and HC L-12 scheme](image1)

**Figure 7.** Displacement on the Y-axis: a) RP model, b) HC model, c) RP and HC J-10 scheme, d) RP and HC L-12 scheme

![Performance of models on the Y-axis: a) scheme J-10, b) scheme L-12](image2)

**Figure 8.** Performance of models on the Y-axis: a) scheme J-10, b) scheme L-12

The HC model under short-term loads did not record the maximum displacement in the middle of the span but at a distance of 62.5 cm from the model centre. The nature of the displacements and the curve shift towards the side support indicated a smaller HC model's lateral stiffness. Even the lock's hinge nature of the shear key from the beginning of the loading process. The indicate for the hinge work
model may be the increased rotation between panels. The differences in displacement, indicating faulting or the joint's shear damage at the panels' longitudinal joint were not recorded.

Comparative calculations were made for a simply supported beam model with a load of 7.7 kN/m² (table 3). The HC model's deflections constitute 25% of the beam model's values, and for the RP, it is 36%. Calculations for HC panels showed higher values than RP due to the lower bending stiffness of the slab. Opposite results were obtained from studies on the model supported at four edges.

### Table 3. Compared results from tests and calculations for beam model

| Model | Experimentally determined displacement, mm | Beam model calculation, mm | Comparison |
|-------|------------------------------------------|---------------------------|------------|
|       | \( \bar{d}_{\text{obs}} \) | \( \bar{d}_{\text{cal}} \) | \( \bar{d}_{\text{obs}} / \bar{d}_{\text{cal}} \) |
| RP    | 3.90 | 11.0 | 0.36 |
| HC    | 3.57 | 14.0 | 0.25 |

### 4. Conclusions

The research was qualitative, comparing the selected slabs' performance in the transverse and longitudinal directions. An important common feature of the slabs was the lack of joint reinforcement. Despite the longitudinal joints' work only through the concrete cross-section, the panels retained the possibility of mutual load redistribution. Joints without traverse reinforcement should be sought as the reasons for the increased reduction in transverse stiffness. Panels connected with one layer of concrete topping (ribbed panels) showed better transverse cooperation than elements connected only at their joint (HC).

The adopted model of simply supported beam operation, realised mainly in practice, is an approach that does not allow for an accurate assessment of the slab's performance. Research is continued to evaluate the long-term effects and to determine the characteristics of longitudinal joints. A comprehensive study will allow for a thorough assessment of the performance characteristics and the formulation of guidelines.

### References

[1] A. Kisiołek, "The market of flooring systems in Poland," *Innov. Mark.*, vol. 14, no. 1, pp. 13–22, Apr. 2018, doi: 10.21511/im.14(1).2018.02.

[2] M. Li, C. Wang, W. Tao, B. Wang, and Z. Sun, "Analysis of the Shear Force Distribution of Laminated Slab with Shear Keys Simply Supported on Four Sides," *Proc. 2015 Int. Conf. Mechatronics, Electron. Ind. Control Eng.*, vol. 1, no. Meic, pp. 476–479, Apr. 2015, doi: 10.2991/meic-15.2015.110.

[3] R. Szydlowski and M. Szreniawa, "New Concept of Semi-precast Concrete Slab on Pretensioned Boards," *IOP Conf. Ser. Mater. Sci. Eng.*, vol. 245, no. 2, p. 022090, Oct. 2017, doi: 10.1088/1757-899X/245/2/022090.

[4] B. Dal Lago et al., "Full-scale testing and numerical analysis of a precast fibre reinforced self-compacting concrete slab prestressed with basalt fibre reinforced polymer bars," *Compos. Part B Eng.*, vol. 128, pp. 120–133, Nov. 2017, doi: 10.1016/j.compositesb.2017.07.004.

[5] S. J. Han, J. H. Jeong, H. E. Joo, S. H. Choi, S. Choi, and K. S. Kim, "Flexural and shear performance of prestressed composite slabs with inverted multi-ribs," *Appl. Sci.*, vol. 9, no. 22, p. 4946, Nov. 2019, doi: 10.3390/APP9224946.

[6] H. Ju, S. J. Han, H. E. Joo, H. C. Cho, K. S. Kim, and Y. H. Oh, "Shear performance of optimised-section precast slab with tapered cross section," *Sustain.*, vol. 11, no. 1, 2018, doi: 10.3390/su11010163.
[7] I. S. Ibrahim et al., "Ultimate Shear Capacity and Failure of Shear Key Connection in Precast Concrete Construction," *Malaysian J. Civ. Eng.*, vol. 26, no. 3, pp. 414–430, Jan. 2014, doi: 10.1111/mjce.2014.02.350.

[8] R. Shamass, X. Zhou, and Z. Wu, "Numerical Analysis of Shear-Off Failure of Keyed Epoxied Joints in Precast Concrete Segmental Bridges," *J. Bridg. Eng.*, vol. 22, no. 1, p. 04016108, Jan. 2017, doi: 10.1061/(ASCE)BE.1943-5592.0000971.

[9] J. Zhang, B. Liu, B. Han, Y. Ni, and Z. Li, "Analysis Of Out-Of-Plane Performance Of Composite Slab With Precast Concrete Ribbed Panels Under a Hanging Load," *PCI J.*, vol. 64, no. 5, 2019, doi: 10.15554/pcij64.5-02.

[10] *EN 13747:2011 - Precast concrete products. Floor plates for floor systems*. Brussels: CEN, 2011.

[11] *EN 15037-1:2011 - Precast concrete products - Beam-and-block floor systems - Part 1: Beams*. Brussels: CEN, 2011.

[12] *EN 1168:2011 - Precast concrete products - Hollow core slabs*. Brussels: CEN, 2011.

[13] J. Zając, Ł. Drobiec, R. Jasiński, M. Wieczorek, and A. Kisiołek, “Experimental tests of the Vector II slab in field conditions, slab and strip model,” *Civ. Environ. Eng. Rep.*, vol. 1, no. nr 31 (1), p. s. 54-69, Mar. 2021, doi: 10.2478/ceer-2021-0004.

[14] J. Zając, "Field test research of Vector III slabs under short-term and long-term load," *Mater. Bud.*, vol. 1, no. 4, pp. 25–28, Apr. 2020, doi: 10.15199/33.2020.04.01.

[15] *EN 12390-1:2013, Testing hardened concrete – Part 1: Shape, dimensions and other requirements for specimens and moulds*. Brussels: CEN, 2013.

[16] *EN 12390-2:2019, Testing hardened concrete – Part 2: Making and curing specimens for strength test*. Brussels: CEN, 2019.

[17] *EN 12390-3:2019, Testing hardened concrete – Part 3: Compressive strength of test specimens*. Brussels: CEN, 2019.

[18] *EN 12390-13:2014, Testing hardened concrete – Part 13: Determination of secant modulus of elasticity in compression*. Brussels: CEN, 2014.

[19] *EN 10080:2007, steel for the reinforcement of concrete - weldable reinforcing steel – general*. Brussels: CEN, 2007.

[20] *EN ISO 15630-1:2004, Steel for the reinforcement and prestressing of concrete – test methods – Part 1: Reinforcing bars, wire rod and wire*. Geneva: CEN, 2004.

[21] *EN ISO 6892-1:2016, Metallic materials – tensile testing – Part 1: Method of test at room temperature*. Geneva: CEN, 2016.