Load Bearing Capacity of Precast Concrete Slab–Wall Connection

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Abstract. Precast concrete slabs and their supporting walls are connected using dowel pins, anchoring loop bars, and cast-in-situ concrete. The design resistance of such connections can be calculated using multiple different methods. The design load bearing capacity can vary depending on the method used and assumptions made during calculation, making it difficult to determine, which methods are reliable and should be preferred. The study ordered by construction company JSC “UPB” discusses two design methods proposed by Eurocode 2 and fib Bulletin 43. A semi full-scale loading tests of concrete walls and slab specimens are carried out. The results show that the design methods considering only the contribution of steel dowel pins, underestimates the actual capacity of the connection by 2 to 4 times. On the other hand, the methods, considering both the contribution of the dowels and the friction between concrete interfaces, correspond well with the experimental results.

1. Introduction
Precast concrete elements are widely used in building structures. There are many advantages of this construction method if compared to cast-in-situ concrete structures. Nevertheless, connections between precast elements demand a special attention both during the design and the construction. Due to the large number of the connections in a multi-storey building, an over conservative solution can influence the costs of construction.

The joint between walls and hollow core slabs is one of the most common connection in precast concrete structures. The two concrete elements are joint together using dowel pins anchored in the walls, enveloped by loop bars anchored in the hollow core slabs, and bonding all together by cast-in-situ concrete. This type of joint is largely subjected to shear forces causing the elements slide against each other at the interface. The dowel pins and the concrete at the interface are the main contributors to the shear capacity of the connection.

There are several design methods available in standards and design guides describing the load bearing capacity of the slab–wall connection. Eurocode 2 [1] gives a method to determine the shear resistance at the interface that considers both the concrete interaction at the interface and the steel reinforcement crossing the connection plane. Fib Model Code 2010 [2], in addition to this method, provides another approach with additional parameters described by Randl [3] and Figueira et al [4]. This method suggests that in case of the slab–wall connection some of the parameters describing the interface should be ignored due to the smooth surface. Fib Bulletin 43 [5] describes several possible approaches
to evaluate the shear resistance at the interface, among which the most conservative method considers
the resistance of the dowel pin only.

As a result, there are several different methods with rather different results. Thus, the practical
designers of structures can get confused, which approach should be taken. The companies that does both
the design and the production and construction are interested in either safe and economic solutions.

To get a better understanding of the actual load bearing capacity of such structural connection, a semi
full scale experimental investigation was performed at the Structures Laboratory, Latvia University of
Life Sciences and Technologies. The specimens were made of elements of real scale thicknesses but
reduced spans. The study was ordered by joint-stock company “UPB”. The aim of the study is to
evaluate the theoretical design methods available for shear connections between precast concrete
elements if compared to semi full scale experimental results. More detailed information about the
experiment can be found in the test report [6].

2. Materials and methods
2.1. Specimens and materials
The specimens were produced in the period from 23.08.2020 to 04.09.2019 and stored outside in the
factory’s territory in Cukura street 34, Liepaja, Latvia. The specimens were specially designed and
prepared for the testing purposes. They were delivered to the laboratory on 15.10.2019 and 26.11.2019
and stored in room conditions till testing (09.01.2020–17.01.2020). The room temperature was 20°C,
±2°C, relative humidity 40% to 50%. There were stiff steel frames used for transportation to avoid
unwanted cracking of the specimens.

The type of the sample is designed to represent a real connection between structural elements used
in precast concrete buildings. They consist of three precast elements (2 walls and 1 hollow core slab)
connected with dowels, anchoring loops and cast-in-situ concrete as shown in figure 1. The
transportation frame was used for lateral stability (see figure 3).

![Figure 1](image)

**Figure 1.** Test specimens with nominal dimensions: 1 – hollow core slab; 2 – reinforced concrete
walls; 3 – cast-in-situ fine grained concrete; 4 – dowel pin; 5 – anchoring loop bar; 6 – neoprene;
7 – flat surface (cut); 8 – uneven surface.
Concrete class C45/55 was used to produce hollow core slabs. The walls were made of reinforced concrete, where concrete cube strength $f_{c,\text{cube}} = 48.98$ MPa and the reinforcement steel was B500B. The dowel pins of diameter 16 mm and yield strength $f_y = 518$ MPa ($f_u = 628$ MPa) were anchored in the concrete walls. The anchoring loops were made of Ø12 mm rebars with yield strength $f_y = 560$ MPa ($f_u = 708$ MPa). The joint between hollow core slab and the walls was filled with cast-in-situ concrete with cube strength $f_{c,\text{cube}} = 52.94$ MPa.

2.2. Test set-up

The specimens were loaded vertically as shown in figure 2. The load was applied to both ends of the hollow core slab (1). Two LVDTs (10, 11) were used to register the vertical displacement of the top of the slab (1) with respect to the walls (2). Two dial indicators (12) were introduced to measure the lateral displacement of the specimens relative to the test frame (9). Between the walls and the base plate, a 20 mm thick layer of elastomeric material (hardness 50 IRHD) was installed. In order to minimise a lateral instability, the specimen was fixed to a steel frame shown in figure 3(a). An additional structure was introduced to keep the loaded hollow core slab tight to the concrete walls during the test figure 3(b).

![Figure 2. Test set-up: 1 – hollow core slab; 2 – reinforced concrete walls; 3 – concrete joint; 4 – gap allowing for displacement of the slab (1); 5 – elastomeric layer; 6 – cross beam; 7 – steel plates; 8 – loading plate with seat-ball; 9 – test frame; 10, 11 – LVDTs for side 1 and 2, respectively; 12 – dial indicators.](image)

2.3. Test procedure

Before each test the initial cracks of each specimen were marked by green colour. The cracks that appeared during the test were coloured in red. For each specimen the load was applied in 4 subsequent steps: 1) controlled by force up to 60 kN with speed 1.0 kN/s; 2) controlled by piston displacement up to 7 mm with speed 0.2 mm/min; 3) controlled by piston displacement up to 30 mm with speed 2.0 mm/min; 4) after unloading to force equal to zero and removing all external measuring devices, a new loading controlled by piston displacement up to 80 mm (the maximum distance possible for these specimens) with speed 0.3 mm/s (18.0 mm/min) was applied.

Besides the piston displacement, the displacement of the hollow core slab relative to its supporting walls in both sides were measured by two LVDT’s. The measuring range of the LVDT’s is up to 10 mm with resolution of 0.001 mm. Lateral displacement of the vertical walls relative to the test frame was controlled by two dial indicators.
2.4. Theoretical design methods

Two methods are discussed in this paper. One approach described in fib bulletin 43 [5] evaluates the load bearing capacity based on dowel action only. In case of one-sided dowel there are three possible failure modes suggested: 1) steel shear failure, 2) concrete splitting failure, and 3) steel flexural failure (combined steel / concrete failure). The load bearing capacity of the connection, if the modes 1 and 3 are combined, can be expressed as follows:

\[ F_{R,dowel} = \min \left\{ \frac{1}{\sqrt{3}} f_{yd} A_s \right. \rightline{a_0 \Omega^2 \sqrt{f_{cd} f_{yd}}} \right. \]

where \( f_{yd} \) – yield strength of the dowel pin, \( f_{cd} \) – tensile strength of concrete, \( A_s \) and \( \Omega \) – cross sectional area and diameter of the dowel pin, respectively.

Eurocode 2 (EC2) provides a method to calculate shear at the interface between concrete cast at different times [1]. It is discussed also in the fib bulletin [5] denoted as the resistance due to friction. The method includes both the interaction of concrete elements at the interface and the influence of reinforcement crossing the contact plane. If expressed as shear resistance force, the EC2 formula can be written as follows:

\[ F_{R,EC2} = \left[ c f_{cd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) \right] b l , \]

where \( c \) and \( \mu \) are factors that depend on the roughness of the interface; \( f_{cd} \) – tensile strength of concrete; \( \rho \) – ratio of the area of the reinforcement crossing the interface over the area of the joint; \( \alpha \) – angle between the dowel and interface plane; \( b \) and \( l \) is the width and the length of the joint, respectively.

3. Results and discussions

3.1. Experimental results

A typical force–displacement curve and crack patterns obtained in the test are given in figure 4. The failure modes of the specimens and the dowel pins are given in figures 5 and 6, respectively.

In all cases the connection failed first in one side (side 1) of the specimen due to concrete failure at the force \( F_{max} \), followed by a drop of the applied force to a level \( F_2 \). Further deformations of the slab in
Figure 4. Typical force–displacement curve, designation and corresponding crack patterns.

Figure 5. Test specimen before and after testing (a) and (b) respectively

Figure 6. Failure mode of the dowel pin: dowel action (a) and failure of the steel (b)
the same side were observed. During this and subsequent phases of the test, bending cracks in the walls appeared due to the rotation of the hollow core slab. The next drop of the applied load (to a level $F_3$) occurred with a sudden opening of the crack in the same joint (side 1) and a crack in the other connection (side 2) appeared and the vertical displacement in the side 2 started to increase. During this phase the test was stopped, the load was withdrawn and the measuring devices removed. During the step 4 (see 2.3. Test procedure) after a certain distance of vertical displacement was reached, another drop of the applied load together with a sudden crack opening on the side 2 occurred. Average values of the forces together with basic statistical results are given in table 1. The obtained force–displacement diagrams for each specimen, where displacement measured by LVDTs on each side of the specimen are shown in figure 7.

![Force-displacement diagrams](image)

**Figure 7.** Obtained force–displacement diagrams for each specimen, where displacement measured by LVDTs on each side of the specimen (represented by different colours)
Table 1. Statistics of the main force levels in the obtained load–displacement curves

|                  | $F_{\text{max}}$ (lower value) | $F_2$ (upper value) | $F_{\text{max}}$ (lower value) | $F_3$ (upper value) |
|------------------|-------------------------------|--------------------|-------------------------------|--------------------|
| Average (kN)     | 715.4                         | 430.7              | 527.8                         | 337.3              |
| Min. (kN)        | 638.3                         | 357.5              | 442.6                         | 288.1              |
| Max. (kN)        | 857.0                         | 473.1              | 705.1                         | 389.4              |
| St. Dev. (kN)    | 86.73                         | 45.21              | 92.80                         | 35.15              |
| Coef. of Var.    | 0.121                         | 0.105              | 0.176                         | 0.104              |

3.2. Theoretical shear capacity

The theoretical shear capacity of the connection is obtained assuming the partial safety factors for materials equal to 1.0. Thus, the strength properties $f_{yd}$ and $f_{cd}$ are 518 MPa and 48.98 MPa, respectively. The diameter of the dowel pin is 16 mm. $\alpha_0$ can be taken 1.0 for design purposes. The load bearing capacity according to (1) in our case $F_{R,dowel} = \min(60.1; 40.8) = 40.8$ kN. This method does not consider the resistance due to friction between concrete elements, therefore it is expected to be rather conservative.

To calculate the shear capacity according to EC2 the following numerical values are assumed: $c$ and $\mu$ are taken 0.2 and 0.6, respectively. Tensile strength of concrete $f_{cd} = 2.81$ MPa. The dowel pin is perpendicular to the interface, thus $\alpha = 90^\circ$. The contact area, where friction can occur, can be divided into two interfaces – 1) the interface between the bottom of the hollow core slab and the wall (see interface 1 in figure 8), the plane where the dowel pin is present; 2) the plane between the end surface (vertical) of the slab and corresponding side of the wall (interface 2 in figure 8). For both surfaces the length $l = 700$ mm. The width of the interface $b_1$, can be taken 85 mm, if the width of the neoprene is ignored. After cracking of the concrete, the width is varying and can be taken as the average of 85 mm and 35…50 mm (thickness of the cast-in-situ joint), which is 60 to 68 mm. The width of the interface 2 is taken as the height of the slab, $b_2 = 265$ mm. Compression stress $\sigma_n$ can occur in the surfaces due to the tie bars used to tighten the hollow core slab to the walls and the specimen to the frame (see 5 and 9 in figure 3). The stresses $\sigma_n$ calculated according to [5] for interfaces 1 and 2 are 0.14 MPa and 0.03 MPa, respectively.

![Figure 8. Interfaces of the connection, where friction can occur: 1 – hollow core slab; 2 – wall; 3 – cast-in-situ concrete; 4 – dowel pin; 5 – anchor bar](image)

There are four different shear resistance forces calculated and presented in table 2. $F_{R,dowel}$ represents the resistance of one dowel pin according to equation (1); $F_{R,EC2}$ and $F_{R,EC2,h}$ stand for the resistance calculated according to equation (2) with full and only horizontal interfaces considered, respectively. In order to compare the total load bearing capacity of the sample with the theoretical one, $F_{R,EC2,tot} = 2 F_{R,EC2}$ assuming $b_1 = 85$ mm is introduced.
Table 2. Theoretical load bearing capacity

| Label          | Numerical value, kN | Description                                                                 |
|----------------|---------------------|------------------------------------------------------------------------------|
| \( F_{R,dowel} \) | 40.8                | resistance of one dowel pin (equation (1))                                  |
| \( F_{R,EC2,h} \) | 89.6                | resistance due to friction after cracking (EC2), only horizontal interface considered |
| \( F_{R,EC2} \)    | 197.3               | resistance due to friction after cracking (EC2), all interfaces considered   |
| \( F_{R,EC2,tot} \) | 417.2               | total load bearing capacity before cracking – both sides considered          |

3.3. Discussion

The theoretical load bearing capacity together with experimental load–displacement diagrams is shown in figure 9. The peak load obtained in the test specimens ranges from 638.3 kN to 857.0 kN that is 1.5 to 2.1 times higher than the estimated design value \( F_{R,EC2,tot} \). However, it does not represent the load bearing capacity of one connection. The actual capacity of the test specimens is not evident in the test results. The obtained force–displacement behaviour is a result of complex simultaneous mechanical processes. The mechanisms that were present in the loading process include: 1) shear at the interface between concrete elements, 2) shear in the dowel crossing the connection, 3) bending of the dowel (dowel action), 4) friction between concrete elements, 5) additional pressure due to rotation of the hollow core slab, 6) shear in the concrete at the dowel, 7) shear in the hollow core slab, and 8) bending in the wall elements. In addition, the total load bearing capacity is a combination of the capacities of each side of the specimen. Some of these mechanisms are present in real design situations, while others are specific to this type of specimens.

In order to exclude some of the effects and to find the load bearing capacity of the connection only on one side \( (F_{R,exp}) \), the following method is proposed. It is suggested here, that the actual capacity is close to the difference of the forces \( F_2 \) and \( F_3 \) given in figure 4. That shows the change in the capacity due to the failure of one dowel. The force–displacement curves on both sides of each specimen obtained are compared and the load difference is obtained by the approach shown in equation (3) and figure 10.

\[
F_{R,exp,i} = F_{disp,1,i} - F_{disp,2,i}
\]

where \( F_{disp,1,i} \) and \( F_{disp,2,i} \) are the registered forces at a displacement \( i \) measured by LVDTs on sides 1 and 2, respectively. In this case side 1 refers to the side of the specimen, where crack appeared first.

The obtained difference between the forces \( F_{disp,1,i} \) and \( F_{disp,2,i} \), which is \( F_{R,exp,i} \), together with theoretical load bearing capacities of one connection are plotted in figure 11. The numerical values of \( F_{R,exp} \) are given in table 3. For specimens SB-3, SB-5, and SB-6 \( F_{R,exp,i} \) is in the range of 82.5 kN to 197.8 kN. For specimen SB-4 the difference is much larger – 345.7 kN in average. The inconsistency can be related to the distinct failure mode of the side 2 for the particular specimen. The force is not determined for specimens SB-1 and SB-2 due to the lack of the LVDT displacement measurements.

Table 3. Numerical values of the experimental shear capacity of one connection (kN)

|        | SB-3 | SB-4 | SB-5 | SB-6 |
|--------|------|------|------|------|
| Min \( F_{R,exp} \) | 171.2 | 316.6 | 82.5 | 121.1 |
| Max \( F_{R,exp} \) | 197.8 | 370.0 | 116.7 | 160.0 |
| Average \( F_{R,exp} \) | 185.4 | 345.7 | 105.0 | 138.2 |
Figure 9. Theoretical load bearing capacity compared with the experimental load–displacement curves

Figure 10. The approach used to evaluate the load bearing capacity of the connection on one side of the specimen, where vertical displacement was measured by LVDTs

Figure 11. Shear capacity of one connection of the specimen
4. Conclusions
The maximum load bearing capacity of the specimens tested in this study ranges between 638.3 kN and 857.0 kN, which is 1.5 to 2.1 times higher than theoretically estimated. The load bearing capacity of the connections for the tested specimens ranges between 82.5 kN and 197.8 kN. For specimen SB-4 the value reaches up to 370.0 kN. If the friction between concrete interfaces is considered according to the method given in EC2, the design shear capacity of one connection ranges between 89.6 kN and 197.3 kN, depending on the area of interfaces considered. Theoretical design methods that considers only the capacity of the dowel (rebars crossing the joint) underestimates the actual load bearing capacity of the connection for around 2 to 4 times.

The mechanical behaviour of the studied specimens was very complex and hard to describe. Different simultaneous processes were observed. This leads to the conclusion that the study gives an overall sense of the behaviour of the dowel connection, but an exact parametric analysis based on these results cannot be derived. Despite all the uncertainties, the research suggests that the design method given in EC2, gives rather precise and thus economic estimate of the shear capacity of the dowel connection between precast walls and hollow core slabs. However, one must evaluate the interfaces to be considered in the design carefully, for it influences the capacity considerably.

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