Stress Behaviour in Compression of Contact-Monolithic Joint of Self-Supporting Wall of Large Panel Multi-Storey Building

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Abstract. The article deals with the behaviour of a contact-monolithic joint of large-panel buildings under compression. It gives a detailed analysis and the descriptions of the stages of such joints failure based on the results of the tests and computational modelling. The article is of interest to specialists who deal with computational modelling or the research of large-panel multi-storey buildings. The text gives a valuable information on the values of their bearing capacity and flexibility, the eccentricity of load transfer from upper panel to lower, the value of thrust passed to a ceiling panel. Recommendations are given to estimate all the above-listed parameters.

1. Introduction

The analysis of methods for assessing the strength and deformability of contact joints of large-panel buildings [1-3] demonstrate that the methods proposed to date provide variegated results and thus require clarification and experimental confirmation. As studies [4-10] show, the accuracy of determining the deformative characteristics of horizontal joints is of great importance.

The use of self-supporting external wall panels in large-panel buildings leads to the necessity of taking into account the efforts of their separation from the internal load bearing structures of the building. Despite this, existing norms only provide indirect methods for estimating the magnitude of such efforts [4].

As is known, the position of the resultant of all vertical forces in the panel depends on the stiffness characteristics of the joints when they rotate. Thus, one of the most important parameter of the joint of the outer wall panel is the eccentricity of the force transfer to the underlying panel [5].

In connection with the foregoing, experimental and theoretical studies aimed at clarifying the work of contact joints of self-supporting external wall panels of large-panel multi-storey buildings were performed.

2. Test procedure

Experimental studies were performed on full-scale fragments of contact joints of self-supporting outer wall panels (figure 1) with a width of 300 mm. In total, four types of samples were tested (KS12-1 ... - 4). The design of the samples provided an opportunity to assess the operability and stiffness of the units, providing the transfer of forces in the panel building.
Each sample consisted of three concrete elements: a fragment of a slab of dimensions 160x300x300 mm and two fragments of external walls measuring 120x300x300 mm and 120x300x200 mm. The thickness of the contact joint is 20 mm. The fragment of the plate rests on the lower part of the wall, and the depth of the supporting surface is 10 mm. This support is carried out through a flexible layer of foam 20 mm thick. The V-shaped emptiness arising between the prefabricated elements of the bottom panel and the overlap is filled with a cement-sand mortar. These elements were joined by horizontal steel tension bars Ø14 mm in order to perceive the tensile force resulting from the compression of the joint between the elements of the slab and the lower wall panel. The transfer of the tensile force from the horizontal ties to the vertical elements of the sample of the type KS12-1 occurred through the steel casing, compressing the latter ones, and at KS12-3 and KS12-4 - through the embedded ones in them, the KS12-2 did not have any ties.

![Figure 1](image)

**Figure 1.** Sample KS12-4 (a), instrument layout (b). 1 – precast concrete element of the top panel; 2 – the same but the bottom panel; 3 the same but the plate; 4 – supporting elements; 5 – steel plate, 6 and 7 – elastic gasket; 8 – tension bar Ø 6mm. All the indicators have a fission value of 0.001mm.

The vertical axial load applied to the test samples was created by means of a test press in steps of 20 kN.

During the testing, absolute horizontal and vertical mutual movements of the prefabricated elements and their deformation of compression were controlled.

**3. The results of tests**

Initial cracks on the surface of the samples were absent, with the exception of KS12-1. In which they were localized vertically (coaxially to the load) in the mortar joint. Their opening width was 0.05 mm and did not change during the loading of the sample.

As for samples KS12-1 and KS12-2, cracks were formed at a compressive load at a joint of 40 kN, cracks formed on the contact of the monolithic V-shaped volume and the prefabricated element of the slab. The width of the cracks increased monotonically with increasing load, and their limiting values were 0.4 mm and 5 mm for the first and second samples, respectively.

Unlike the previous samples, the third and fourth ones did not show such cracks. Instead, at a load of 280 kN to 360 kN, vertical (along the load) cracks appeared in the solution seam. Their opening width did not change during the loading process and was from 0.05 to 0.1 mm.

Absolute deformations of the compression of the solution joints of all samples also increased with increasing load in the joint. In this case, the fastest increment was observed on the indicators of the
plate installed from above, the growth rate at these sites was on the average 1.5-3.2 times higher than from the outside and under the plate.

The horizontal movements of the plate element relative to the wall elements indicate the predominance of the rotational displacement component over the translational component for all samples except KS12-4.

The relative deformations of all the samples had a similar character. So, from the outside of the prefabricated element of the lower wall panel were 0.00050 to 0.0014 at a load of 380 kN to 420 kN. On the inside, the relative deformations first grew then changed sign. Their limiting value was from -0.00012 to -0.00053.

The destruction of the samples was accompanied by the appearance of a large number of vertical (co-axial load) cracks in the upper element of the wall panel, passing into the junctions. The values of the destructive loads are shown in Table. 1.

It is possible to note the following principal differences in the schemes of destruction of specimens with and without tension bars (KS12-2):

1. The absence of tension bars led to the fact that in addition to the upper precast element of the wall panel, the lower one also collapsed.
2. In addition, since the horizontal displacement of the element of the plate is not limited by anything, its separation from the elements of the wall panels occurred, accompanied by the appearance and opening of cracks passing along the faces of the monolithic prism.

Figure 2. Fractures pattern of samples

4. Analysis of test results

4.1. Flexibility of the joint

The design of the joint is such that the transfer of the compressive force occurs partly through the contact pad, and partly through the V-shaped cast element. Taking into account the fact that in the joint there is an eccentric compression, this leads to the rotation of the wall panels relative to each other, constrained by the joint design. The latter confirms the need to take into account the elastic connection of wall elements and the introduction of joint flexibility when turning $\lambda \varphi$.

For the movements of compression and rotation, the corresponding diagrams for each sample were plotted, depending on the load (figure 3, 4). The values of the stiffness for the experimental samples were obtained by approximating the linear dependence for the most characteristic sections of such diagrams. The flexibilities $\lambda c$ and $\lambda \varphi$ were defined as the inverse of stiffness.

4.2. The value of the actual eccentricity

The value of the actual eccentricity was determined on the basis of the obtained relative deformations of the inner and outer faces of the lower precast element, taking into account the assumption of a linear deformation law within the cross-section.

The value of the actual eccentricity of the longitudinal force arising in the lower collection element of the wall panel changed during the loading process and at loads close to the destructive (within 300 kN) exceeds the normalized value 15 mm.
4.3. Thrust force

The thrust force was determined on the basis of the experimental strain values of the tension bars. The maximum stresses occurring in the tension bars were from 4.75 kN to 5.64 kN.

5. Analytical determination of the joint parameters

The bearing capacity of the contact junction N is determined according to the norms in force in [4] according to the formula below and is indicated in table 1.

$$N = R_{bw} \left( b_m - \frac{(2-t_m/b_m) t_m}{1 + 2 R_m / B_m} \right)$$

(1)

$R_{bw}$ – strength of wall concrete under compression; $t_m = 28 \text{ mm}$ – estimated thickness of the mortar joint; $b_m = 70 \text{ mm}$ – width of contact area for wall thickness; $R_m$ – cube strength of mortar, MPa; $B_m$ – the value is numerically equal to the grade of prefabricated concrete;

According to [4], the flexibility of the contact junction at its compression is determined by the formula:

$$\lambda_c = 5 \cdot 10^{-3} \cdot R_m^{2/3} \cdot t_m + \frac{h_{con}}{E_{b,w}}$$

(2)
$h_{\text{con}} = 160 \text{mm}$ – contact junction height; $E_{b,w}$ – modulus of deformation of concrete of wall.

According to [4], the eccentricity is determined by the formula:

$$e^0_j = \frac{(t - h_{\text{con}})}{2}$$

To determine the coefficient that is proportional to the ratio of the thrust force to the compressive force, we propose a formula:

$$k_p = \frac{\Delta b_m / h_p}{1 + \frac{E_{b,w} \cdot b_m \cdot t}{E_p \cdot A_p}}$$

(3)

$E_p \cdot A_p$ – stiffness of the V-shaped cast element, $\Delta b_m / h_p$ – the slope of its faces.

### 6. Numerical simulation of joints

Nonlinear calculation of joints is carried out in the program "Lira-SAPR" by finite element methods. The total load on the joint is assumed equal to 600 kN. The load is applied in stages, the number of steps is taken equal to 30.

![Figure 5. Finite element joint model](image)

**Table 1.** A The results of experiments, numerical modeling and analytical calculations

| Sample | Prism strength, MPa | Mortar strength, MPa | Bearing capacity of the joint, kN/mn | Thrust force parameter tests | Thrust force parameter theory [4] | Thrust force parameter theory [5] |
|--------|---------------------|----------------------|-------------------------------------|-----------------------------|-----------------------------------|-----------------------------------|
| KS12-1 | 17.5                | 13.84                | 1466                                | 865.6                       | 1954                              | 0.01102                           | 0.03727                           |
| KS12-2 | 18.1                | 16.63                | 2000                                | 1600                        | 923.1                             | 2018                              | -                                 | 0.03817                           |
| KS12-3 | 16.5                | 9.9                  | 1400                                | 1400                        | 835.9                             | 1839                              | 0.01131                           | 0.03261                           |
| KS12-4 | 18.4                | 10.3                 | 1460                                | 1600                        | 916.9                             | 2051                              | 0.01288                           | 0.03125                           |

**Table 2.** The continuation of the Table 1.

| Sample | Linear stiffness, kN/(mm·mn) | Angular stiffness, 1/mm | Actual eccentricity, mm |
|--------|------------------------------|-------------------------|--------------------------|
| KS12-1 | 2264.9                       | 425.7                   | 22.09                    |
| KS12-2 | 2367.3                       | 325.5                   | 21.28                    |
| KS12-3 | 1771.1                       | 381.2                   | 14.35                    |
| KS12-4 | 1700.0                       | 368.4                   | 20.3                     |

For the destruction of the joint was taken such a state, when there was a sharp increase in the deformation of the loading area, which corresponds to the destruction of the joint.

The mortar joint was modeled only in a contact area of 90 mm width. V-shaped volume was modeled by elastic bonds, which work only on compression.
The characteristics of the joint materials specified in the design diagram correspond to the actual ones. To take into account the nonlinear work of materials, an exponential deformation law is adopted. For the transition to the strength of a 20 mm thick mortar joint, a conversion factor of \( k = 0.56 \) is used \[6\]. The results of the calculation are summarized in table 1. In addition, the table gives the results of determining the load-carrying capacity according to \[3\]. Proceeding from this work, it is recommended to determine the load-bearing capacity of 1 pm of the contact joint of walls according to the formula

\[
N \leq 1.22R_n t_w
\]  

(4)

7. Conclusion
1. The possibility of determining the load-bearing capacity and compliance of the contact joints was confirmed, according to the existing norms, taking into account the safety factors 1.6. In this case, the angular stiffness, when simulating elastic pinching in a given contact junction between panels, should be taken equal to 3,748 \( \cdot 105 \) kN / mn.
2. The value of the actual eccentricity of the longitudinal force varies during the loading process and exceeds the normalized value at the stages preceding the fracture.
3. The thrust force for this joint is 2.1% of the compression force, this should be taken into account when designing the connections between the outer walls and the slab.
4. The obtained results allow to make a conclusion about the acceptable convergence of the calculation results by the finite element method with the results of laboratory tests.

Acknowledgments
Authors of the article express their deep gratitude to the organization of BETOTEK (Chelyabinsk) for financial support and the Laboratory of Building Structures of SUSU staff for assistance in testing.

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