WARSAW UNIVERSITY OF TECHNOLOGY Index 351733
FACULTY OF CIVIL ENGINEERING
COMMITTEE FOR CIVIL AND WATER ENGINEERING
POLISH ACADEMY OF SCIENCES ISSN 1230-2945

DOI: 10.24425/ace.2025.155085

ARCHIVES OF CIVIL ENGINEERING

Vol. LXXI ISSUE 3 2025

pp. 49 –61

Research paper

Experimental tests of steel balcony connections – part 2

Maciej Tomasz Solarczyk[1], Paweł Piotrkowski[2], Maciej Niedostatkiewicz[3]

Abstract: Paper presents theoretical analysis and results of experimental tests of three prefabricated balcony sets in natural scale with dimensions (width × length × height): 2.0 m × 2.78 m × 0.186 m (in a slope to 0.17 m) and one with dimensions: 2.0 m × 2.78 m × 0.2 m, consists of reinforced concrete slabs connected with each other with steel balcony connections. The impact of variable parameters (elongation of anchorage of balcony connections in ceiling slab, concreting of test elements in two stages, using of muffs as couplers to connect the longitudinal reinforcement bars in balcony sets and different height of the balcony slab) on the load bearing capacity of the elements are analysed. During the experimental tests deformations in the balcony connection using strain gauge sensors were measured. Global safety factor for all tested elements are determined. A numerical shell model of balcony connection is presented, validated using the results from experimental tests.

Keywords: balcony, balcony connections, experimental tests, reinforced concrete structures

[1]MSc., Eng., Gdansk University of Technology, Faculty of Civil and Environmental Engineering, Gabriela Narutowicza 11/12, 80-233 Gdansk, Poland, e-mail: maciej.solarczyk@pg.edu.pl, ORCID: 0000-0001-6070-0736
[2]DSc., PhD., Eng., Gdansk University of Technology, Faculty of Civil and Environmental Engineering, Gabriela Narutowicza 11/12, 80-233 Gdansk, Poland, e-mail: piotrkow@pg.edu.pl, ORCID: 0000-0001-7860-673X
[3]DSc., PhD., Eng., Prof. GUT, Gdansk Universityn of Technology, Faculty of Civil and Environmental Engineering, Gabriela Narutowicza 11/12, 80-233 Gdansk, Poland, e-mail: mniedost@pg.edu.pl, ORCID: 0000-0002-6451-6220

1. Introduction

This paper is a continuation of the publication [1].

In [1] authors presents theoretical analysis and results of experimental tests of three prefabricated balcony sets in natural scale with dimensions (width × length × height): 2.0 m × 2.78 m × 0.186 m (in a slope to 0.17 m) and one with dimensions: 2.0 m × 2.7 m × 0.2 m, consists of reinforced concrete slabs connected with each other with steel balcony connections. The impact of variable parameters (elongation of anchorage of balcony connections in ceiling slab, concreting of test elements in two stages, using of muffs as couplers to connect the longitudinal reinforcement bars in balcony sets and different height of the balcony slab) on the load bearing capacity of the elements were analysed. In [1] the test stand was described. During these experimental tests, the crack morphology was determined, displacements and crack widths were measured. The paper contains also a review of the scientific papers in the field of balcony connections.

In this paper deformations in the balcony connection using strain gauge sensors are described. A numerical model of balcony connection is presented, validated using the results from experimental tests.

Preliminary experimental tests of steel balcony connections in the aspect of load bearing capacity of balcony – slab joint was presented in [2]. Experimental tests of full-scale prefabricated balcony sets consist of reinforced concrete slabs (balcony and ceiling) connected with each other with double-type balcony connections was described in [3].

2. Materials and methods

The authors tested full-scale prefabricated balcony sets: three with dimensions (width × length × height): 2.0 m × 2.78 m × 0.186 m (in a slope to 0.17 m) and one with dimensions: 2.0 m × 2.78 m × 0.2 m. The assumed differences between the sets were discussed in [1]. The sets consists of reinforced concrete slabs (balcony and ceiling) connected with each other by balcony connections. The overhang of the cantilever (measured to the edge of ceiling slab) equals L_{eff} = 1.78 m. Concrete C30/37 and reinforcement steel K500-B-T were used to build the balcony sets.

Experimental tests were carried out on a specially designed test stand (Fig. 5 in [1]), which was adapted to the Zwick-Roell 500 kN testing machine. The two hydraulic jacks of testing machine were placed at a spacing of 2.0 m (one act as a ballast). It was assumed that balcony sets would be loaded quasi linearly, parallel to the outer edge of balcony slab by use of set of beams on which the concentrated load (F) of the hydraulic jack of the testing machine was applied.

Loading scheme during experimental tests corresponded to the theoretical (standard) loading scheme of the balcony slab. During tests strains of steel (strain gauges with an accuracy of $0.1 \times 1/200$ mm/m) were tested [4, 7].

In balcony set I_ZB_1_1 a 36 strain gauges – Fig. 3, while in II_ZB_1_2, II_ZB_1_3 and II_ZB_1_4 a 12 strain gauges in each balcony sets were used – Fig. 4.

Strain gauges in strain diagrams of balcony connections were marked as follows (Fig. 1):

1 _ **T1**
(1) (2)

Fig. 1. The method of marking strain gauges

(1) – (1, 2, 3, 4, 5, 6, 6', 5', 2' or 1') – reinforcement bar on which a strain gauge was glued – numeration according to Fig. 3 and Fig. 4.

(2) – (T1, T2, T3, T4, T5 or T6) – number of strain gauge – numeration according to Fig. 3 and Fig. 4. On Fig. 2 a photo of the balcony connections before concreting, explaining the location of strain gauges was presented.

In terms of strain in steel measurements, the following sign convention was assumed: "+" denotes compression and "–" denotes tension.

In the strain diagrams in section 3. (Fig. 5–Fig. 10), example results (due to the fact that in all balcony sets the stresses in T1, T2, T3, T4, T5 and T6 strain gauges did not differ significantly between I_ZB_1_1, II_ZB_1_2, II_ZB_1_3 and II_ZB_1_4) of tests for each of measured points are shown (from T1 to T6).

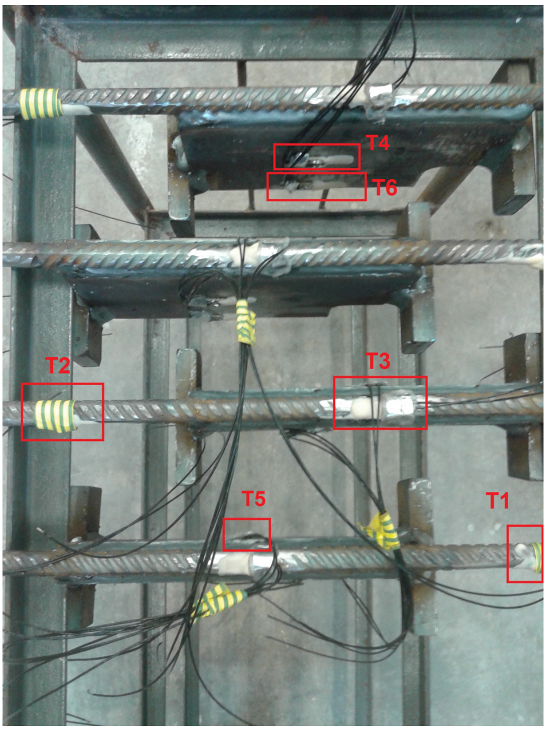

Fig. 2. Strain gauges glued on balcony connections (plan view) in full set of strain gauges (see Fig. 3 and Fig. 4)

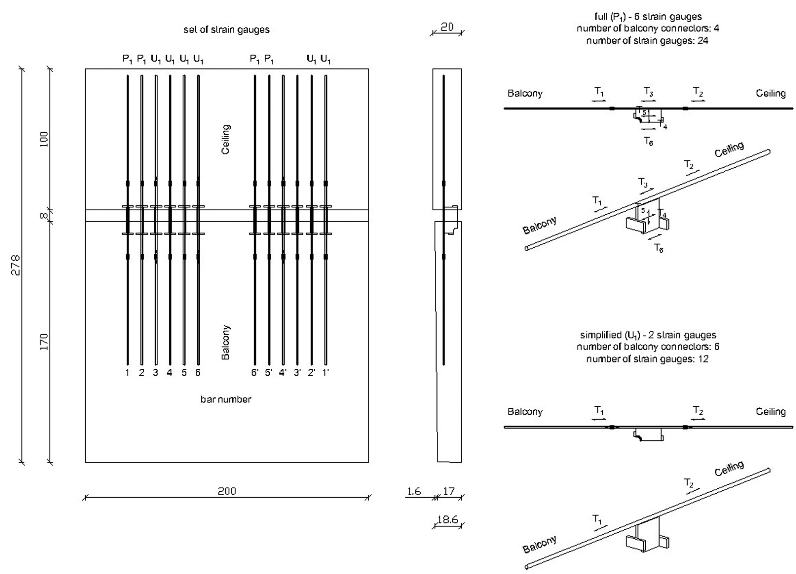

Fig. 3. Arrangement of strain gauges on balcony connection during Stage I of tests

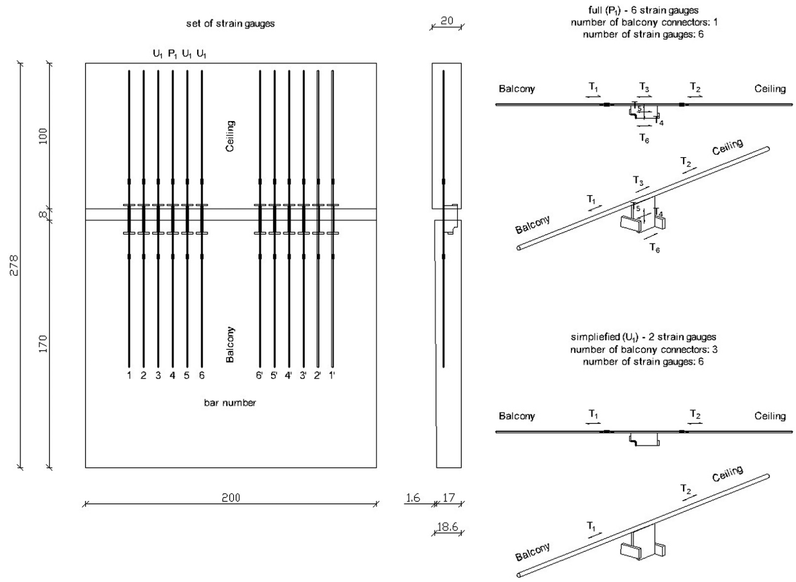

Fig. 4. Arrangement of strain gauges on balcony connection during Stage II of tests (height of balcony slab: 20 cm for II_ZB_1_4)

The maximum value of load obtained in tests was assumed as the failure force (F_u). During testing, the value of the force was taken as the load from the hydraulic jack of the Zwick–Roell 500 kN testing machine, given with an accuracy of 0.1 kN.

3. Experimental results

The amount of strains in gauges placed on bars (see T1 and T2 – Fig. 5 and Fig. 6) showed that most of the load was taken by the bars on the ceiling slab side (T2), which is natural due to the greater overhang and related with that – higher values of bending moments. The first bars on the ceiling slab (Fig. 6) reaches the yielding point at a load level (F/F_u) of about 0.6, while on the balcony slab (Fig. 5) this point is reached at a load level of about 0.85.

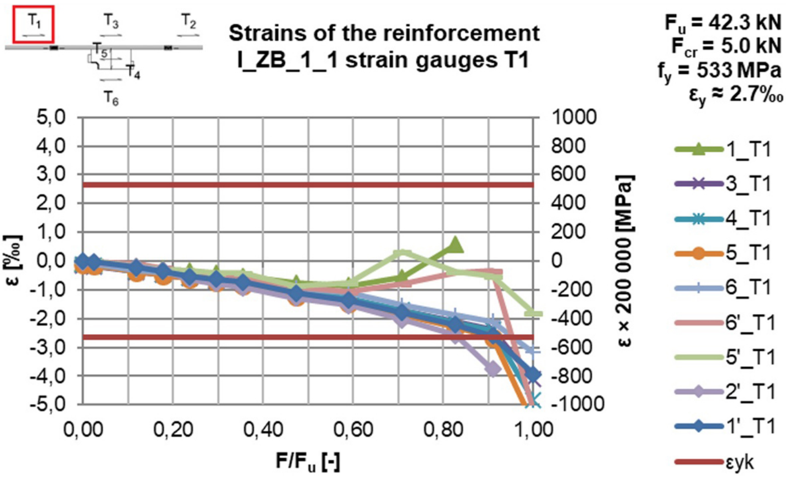

Fig. 5. Strains of the reinforcement in balcony set I_ZB_1_1 measured for T1 strain gauges during Stage I of tests

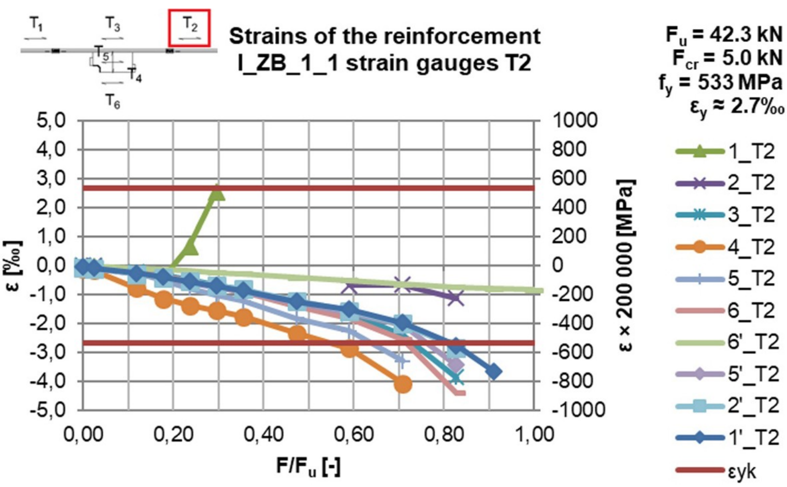

Fig. 6. Strains of the reinforcement in balcony set I_ZB_1_1 measured for T2 strain gauges during Stage I of tests

Strain gauge T3 (Fig. 7) was placed on the top of the balcony connection (in the middle), on the reinforcement bar welded to plate. Tensile stresses values were below the yield point, and were lower than stresses obtained in the T1 and T2 strain gauges (see Fig. 5, Fig. 6 and Fig. 7) – reaching a maximum σ = 150 MPa.

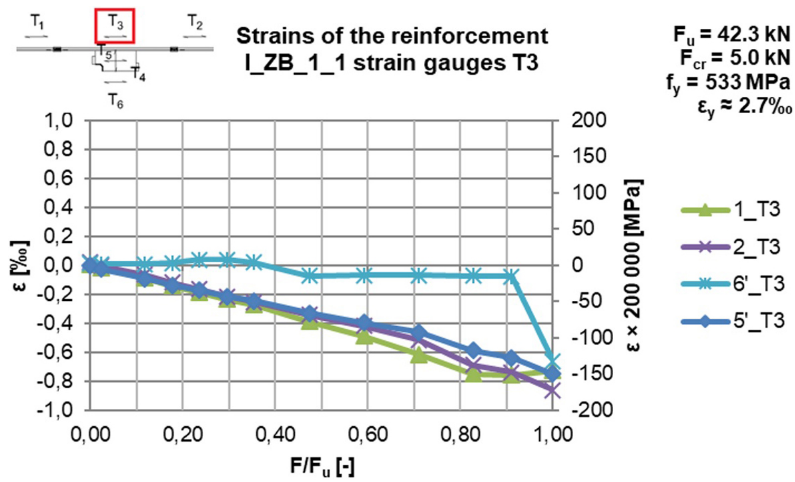

Fig. 7. Strains of the reinforcement in balcony set I_ZB_1_1 measured for T3 strain gauges during Stage I of tests

For strain gauge T4 (Fig. 8) placed in the middle of plate – horizontally (see Fig. 2, Fig. 3 and Fig. 4) the stresses were constant, regardless of the load level.

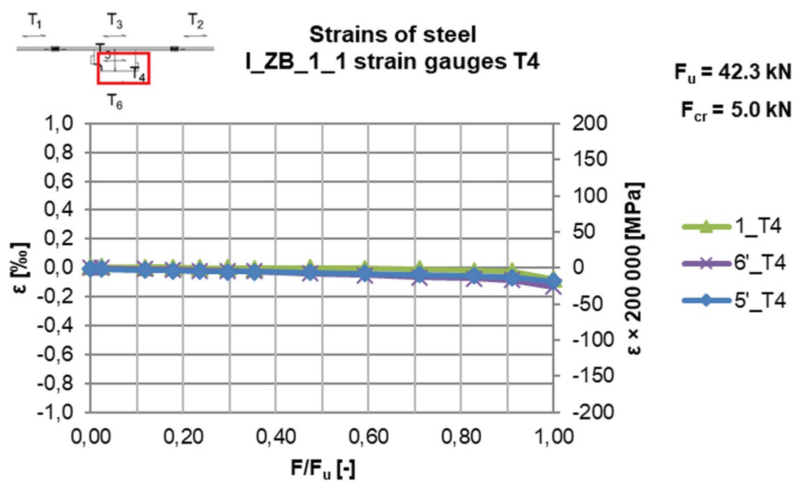

Fig. 8. Strains of the reinforcement in balcony set I_ZB_1_1 measured for T4 strain gauges during Stage I of tests

Strain gauge T6 (Fig. 9) was placed on the bottom of the balcony connection in compression zone. The stress values increased almost linearly at subsequent load levels. The stress σ = 100 MPa at failure was approximately twice lower than the value of the tensile stress at failure in the T3 strain gauge σ = 200 MPa (see Fig. 7).

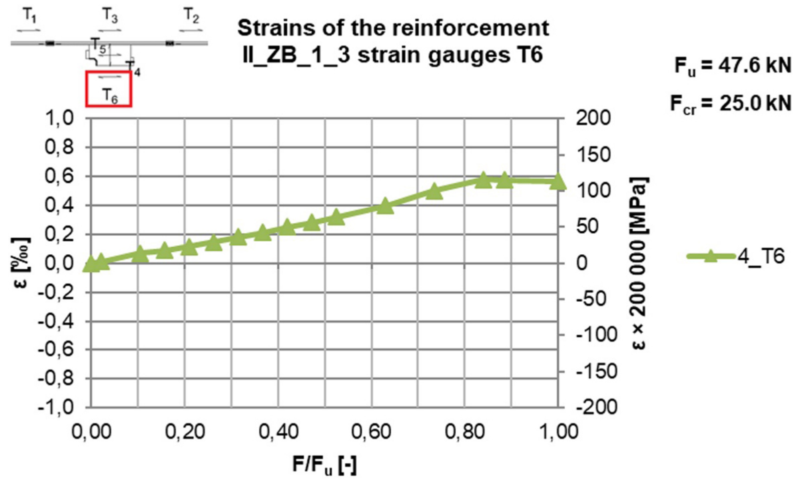

Fig. 9. Strains of the reinforcement in balcony set II_ZB_1_3 measured for T6 strain gauges during Stage II of tests

For strain gauge T5 (Fig. 10) placed in the middle of plate – vertically (see Fig. 2, Fig. 3 and Fig. 4) the stresses were constant, regardless of the load level and oscillated around 0 MPa over the entire load range.

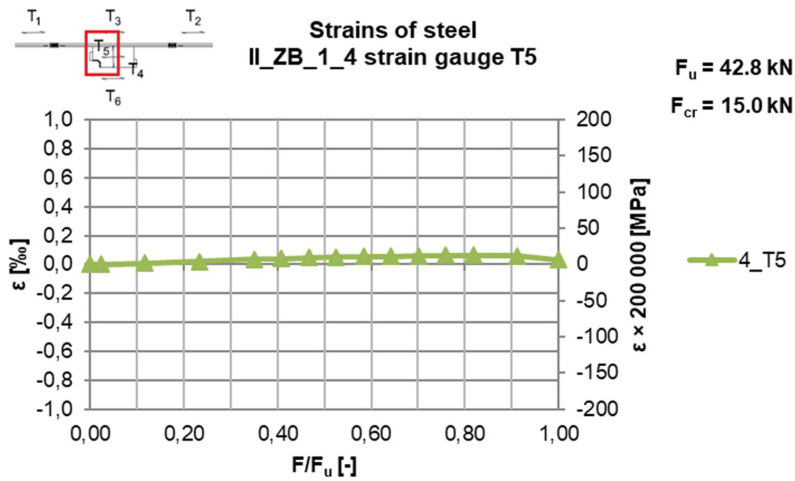

Fig. 10. Strains of the reinforcement in balcony set II_ZB_1_4 measured for T5 strain gauges during Stage II of tests

4. Static calculations

The values of internal forces (M_{Ek}) were determined from standard actions [8] (also using standards [9], [10]) and was described in [1].

Section modulus before cracking for balcony: $W = \dfrac{B \cdot h^2}{6} = \dfrac{200\ \mathrm{cm}\cdot(18.6\ \mathrm{cm})^2}{6} = 11532.0\ \mathrm{cm}^3$ ($13333.3\ \mathrm{cm}^3$ for II_ZB_1_4 due to different balcony slab height – see [1])

Mean yield strength of reinforcement: $f_{\mathrm{y}} = 533$ MPa (obtained from testing samples of reinforcing bars with a diameter of φ 12 mm).

Effective depth of cross section: $d = 9.6$ cm (for II_ZB_1_4: $d = 6.6$ cm – measured from the bottom of the transverse plate of the balcony connection).

As lever arm of internal forces z, the distance between the centroid of the tension reinforcement of the balcony and the centroid concrete compression force (which was calculated as the force acting on the transverse plate of the balcony connection) was assumed.

Table 1. Calculated theoretical cracking moment and resistance due to bending moment for balcony connections in Stage I and Stage II of tests

	I_ZB_1_1	**II_ZB_1_2**	**II_ZB_1_3**	**II_ZB_1_4**
Mean value of concrete compressive strength: f_{cm} [MPa]	38.0	for ceiling slab: 53.0 for balcony slab: 43.0	38.1	37.9
Calculated cracking moment: $M_{\mathrm{cr,calc}} = W \cdot f_{\mathrm{ctm}}$ [kN·m]	33.4	36.9	33.3	33.6
Mean value of resistance due to bending moment: $M_{\mathrm{Rk}} = A_{\mathrm{s1}} \cdot f_{\mathrm{y}} \cdot z$ [kN·m]	62.3	64.3	62.3	49.1

5. Analysis of experimental results

Results of experimental tests – deflections, crack morphology and crack widths were described and analysed in [1].

The failure of all tested balcony sets was caused by gradual pulling out of the balcony connections from the concrete of the ceiling slab. At that time, cracks and fragments of concrete which had split off from the front of ceiling slab were visible. This corresponded to a significant deflection of about 19 cm.

Calculated resistance due to bending moment M_{Rk} (assuming yielding in reinforcement bars of balcony connections) related to failure moment M_u (value taken from Table 1 in [1]) was on average $\eta_u = \dfrac{M_{Rk}}{M_u} = 0.69$.

For balcony connections, the average global safety factor was $\gamma = 2.19$ and additionally for all of sets the global safety factor was higher than 2.0.

Table 2. Summary of the determined safety factors and load level factors for balcony connections in Stage I and Stage II of tests

Description		Symbol	I_ZB_1_1	II_ZB_1_2	II_ZB_1_3	II_ZB_1_4
Ratio of the calculated resistance to failure moment	calc / test	$\eta_u = \dfrac{M_{Rk}}{M_u}$	0.76	0.69	0.70	0.59
Load level from standard actions	calc / test	$\eta = \dfrac{M_{Ek}}{M_u}$	0.48	0.43	0.45	0.49
Central safety factor [11]	calc / calc	$\dfrac{M_{Ek}}{M_{Rk}}$	0.63	0.61	0.63	0.83
Global safety factor	test / calc	$\gamma = \dfrac{M_u}{M_{Ek}}$	2.08	2.35	2.25	2.06

6. FEM model of balcony connection

In order to carry out a numerical analysis [12] of the balcony connection, a shell model was created in the Autodesk®Robot™Structural Analysis Professional version 2015 (educational license) – ARSA.

The Delaunay meshing method with quadrilaterals 4-node finite elements with a mesh size of 2.5 mm was used.

An analysis of the convergence of the FE mesh was performed with another doubled mesh refinement for element sizes: 10 mm, 5 mm, 2.5 mm and 1.25 mm. In the presented calculations 1% difference in internal forces results is taken as a limit for the subdivision of FE mesh.

The dimensions of the elements of the balcony connection were taken from Fig. 1 in [1]. In the ARSA program a ϕ12 mm bar was modeled as a shell element with equivalent cross-sectional area equals area of a reinforcing bar (1.13 cm^2) made of steel with a yield strength of 533 MPa, undercut plate with thickness of 20 mm made of S355J2G3 steel and two transverse plates with thickness of 12 mm made of S355J2G3 steel.

As the supports: fixed displacement in x-direction (see orientation – Fig. 11) on the reinforcing bar ϕ12 mm and on a transverse plate on the ceiling side of seta and fixed displacement in the z-direction on a fragment of the undercut plate on the ceiling side of set (see Fig. 12) were used. Additionally, single elastic supports were introduced to ensure the stability of the model.

The load to the balcony connection model was applied as a tensile force of 60 kN acting on the reinforcing bar (causing yielding of the ϕ12 mm bar with f_y = 533 MPa), a compressive force of 60 kN (providing the equilibrium of forces in the structural system) applied to the transverse plate and the undercut fragment of plate (up to the effective height of the compression zone) and a shear force of 4.9 kN (the force value was calculated from the model from which the yielding of the longitudinal reinforcement was obtained).

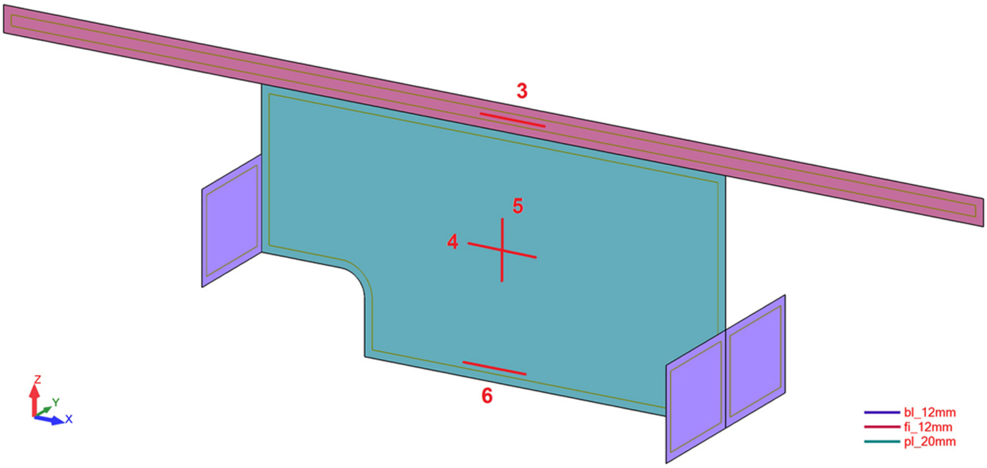

Fig. 11. Shell computational model of the balcony connection with assumed thicknesses of elements

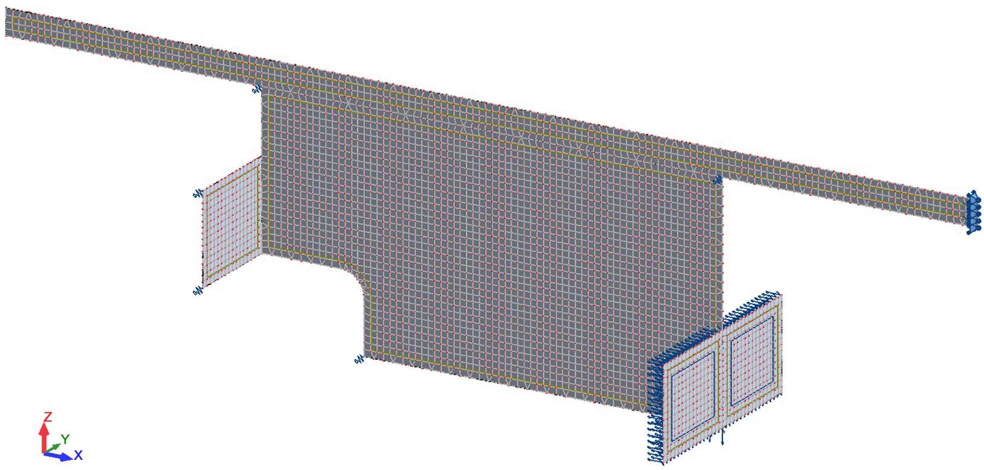

Fig. 12. Finite Element mesh of the balcony connection (with size of 2.5 mm) with supports in the ARSA program

Based on the linear-elastic analysis, it can be concluded that the stress values (using the HMH (Huber–Mises–Hencky) hypothesis [13, 14]) in the plate elements of the balcony connection do not exceed the yield point. The stress values (Fig. 13) on the top (strain gauge T3) and the bottom (strain gauge T6) of the balcony connection do not differ significantly from those determined from strain gauge measurements (see Table 3 – place 3 and place 6 – difference less than 5%). A significant difference in the results is observed for strain gauges 4 and 5 (especially 4) – however it should be underlined that this place is a complex state of stress and any mistake in gluing the strain gauge (a slight difference in the angle of its attachment) may result in divergent results. Moreover, it should be noted that the stresses in this place are low (a few MPa) – therefore, percentage differences in subsequent results are additionally sensitive to any fluctuations.

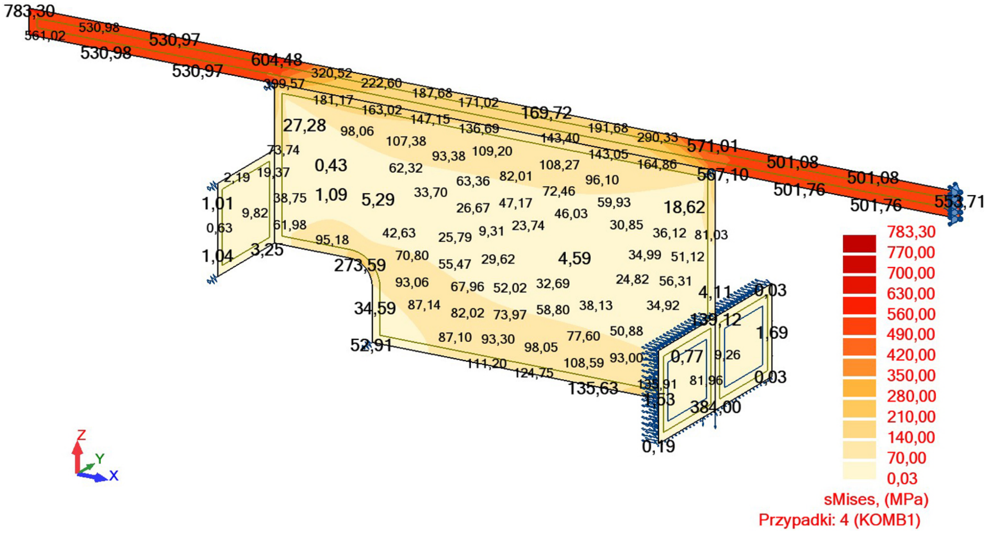

Fig. 13. Complex stresses in the linear-elastic state by HMH (Huber–Mises–Hencky) hypothesis in the balcony connection in the ARSA program. The stresses in the balcony connection plate are lower than the yield strength

Table 3. Comparison of the test results with obtained from simplified numerical model (convention: "+" – tension, „–" – compression)

Place (see Fig. 11.): type of stress	σ_{test} **[MPa]**	σ_{calc} **[MPa]**	**Difference [%]**	**Test results**
3: σ_{xx}	171.9	171.72	0.1	see Fig. 7.
4: σ_{xx}	–26.6	–11.55	130.3	see Fig. 8.
5: σ_{yy}	12.6	10.73	17.4	see Fig. 10.
6: σ_{xx}	–115.3	–120.44	4.3	see Fig. 9.

Based on the numerical results obtained from the ARSA program, it can be concluded that the connector model has been properly validated against experimental data.

7. Conclusions and summary

The mode of the failure of all tested sets was similar and showed gradually pull out of balcony connections from the ceiling slab and fragments of concrete that had split off from the front of the ceiling slab were visible. The pull out of balcony connections occurred in subsequent steps of loading after reaching the failure force, in which the deflection increased (rotation of the balcony connection), with the decrease in force.

Despite the increase in anchorage length of the balcony connection in the ceiling slab and the thickness of balcony slab (set II_ZB_1_4), no significant increase of load-bearing capacity was observed. The reduction of effective depth of a cross section is the reason for this, and it is not compensated by the increase of anchorage length and balcony slab thickness.

The results of the deflections show that the modified geometry of balcony connection (elongation of anchorage) in set II_ZB_1_4 did not reduced the vertical displacement of the balcony (see Fig. 15 in [1]).

No negative effects on use of muff on longitudinal reinforcement bars were observed.

Concreting the elements in two stages (ceiling slab and balcony slab build with a two-day interval) did not result in significant changes in the load-bearing capacity of the balcony sets (the basic difference is the result of obtaining higher concrete strength in this element).

In a further stage of work, an attempt to optimize the geometry of the balcony connection will be made, in order to improve the economic parameters (lower steel consumption in balcony connection) with relatively low losses in the load-bearing capacity or serviceability of the balcony connection.

References

[1] M.T. Solarczyk, P. Piotrkowski, and M. Niedostatkiewicz, "Experimental tests of steel balcony connecions – part 1", *Archives of Civil Engineering*, vol. 71, no. 2, pp.279–293, 2025, doi: 10.24425/ace.2025.154121.

[2] M.T. Solarczyk, P. Piotrkowski, and M. Niedostatkiewicz, "Wstępne badania eksperymentalna stalowych łączników balkonowych w aspekcie nośności złącza balkon – strop", in *Wybrane problemy naukowe budownictwa 65. Konferencja Naukowa Komitetu Inżynierii Lądowej i Wodnej PAN oraz Komitetu Nauki PZITB*, W. Drozd, P. Kozioł, and K. Zima, Eds. Warszawa: PWN, 2020, pp. 10–29.

[3] M.T. Solarczyk, P. Piotrkowski, and M. Niedostatkiewicz, "Experimental tests of steel double-type balcony connections", *Technical Transactions*, no. 1, 2024, doi: 10.37705/TechTrans/e2024001.

[4] Z. Orłoś, *Doświadczalna analiza odkształceń i naprężeń.* Warszawa: Państwowe Wydawnictwo Naukowe, 1977.

[5] Z. Rolinski, *Tensometria oporowa*. Warszawa: Wydawnictwo Naukowo-Techniczne, 1981.

[6] Z.L. Kowalewski and T. Szymczak, "Podstawy tensometrii elektrooporowej oraz praktyczne jej zastosowania", presented at XIX Seminarium Nieniszczące badania materiałów, Zakopane, 2013.

[7] M. Chybiński, J. Dębiński, A. Glema, J. Grzymisławska, D. Jezierski, Ł. Polus, and W. Szymkuć, "A comparison of elasto-plastic parameters of S355 steel obtained in tensile tests using an extensometer, a strain gauge and an ARAMIS 3D DIC system", *Archives of Civil Engineering*, vol. 68, no. 4, pp. 199–217, 2022, doi: 10.24425/ace.2022.143034.

[8] EN 1991–1-1:2002 Eurocode 1: Actions on structures – Part 1–1: General actions – Densities, self-weight, imposed loads for buildings.

[9] EN 1992-1-1:2004 Eurocode 2: Design of concrete structures - Part 1–1: General rules and rules for buildings.

[10] PN-B 03264:2002 Konstrukcje betonowe, żelbetowe i sprężone – Obliczenia statyczne i projektowanie.

[11] M. Gwóźdź and A. Machowski: *Wybrane badania i obliczenia konstrukcji budowlanych metodami probabilistycznymi*. Kraków: Wydawnictwo Politechniki Krakowskiej, 2011.

[12] O.C. Zienkiewicz, *The Finite Element Method in Structural and Continuum Mechanics*. New York: McGraw-Hill, 1967.

[13] M.T. Huber, *Właściwa praca odkształcenia jako miara wytężenia materyału – Przyczynek do podstaw teorii wytrzymałości*. Lwów: Towarzystwo Politechniczne we Lwowie, 1904.

[14] R. von Mises, "Mechanik der festen Körper im plastisch-deformablen Zustand", *Gesellschaft der Wissenschaften zu Göttingen. Mathematisch-Physikalische Klasse*, vol. 1913, pp. 582–592, 1913.

Badania eksperymentalne stalowych łączników balkonowych – część 2

Słowa kluczowe: badania eksperymentalne, balkony, łączniki balkonowe, konstrukcje żelbetowe

Streszczenie:

W artykule przedstawiono analizę teoretyczną oraz wyniki badań eksperymentalnych dotyczących prefabrykowanych zestawów balkonowych w skali naturalnej: trzech o wymiarach (szerokość × długość × wysokość): 2.0 × 2.78 × 0.186 m (w spadku do 0.17 m) oraz jednego o wymiarach: 2.0 × 2.78 × 0.2 m, składających się z żelbetowych płyt połączonych ze sobą za pomocą stalowych łączników balkonowych. W publikacji przeanalizowano wpływ parametrów zmiennych (zwiększona długość kotwienia łączników, betonowanie elementów badawczych w dwóch etapach, zastosowanie muf na prętach zbrojenia podłużnego oraz zwiększona grubość płyty balkonowej) na nośność elementów. W trakcie badań mierzono odkształcenia w łączniku balkonowym za pomocą czujników tensometrycznych. Wyznaczono globalny współczynnik bezpieczeństwa dla wszystkich badanych elementów. Przedstawiono powłokowy model numeryczny łącznika balkonowego, zwalidowany przy pomocy wyników badań eksperymentalnych.

Received: 2024-02-15, Revised: 2024-04-08