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FAILURE ANALYSIS ON EDGE FLAT-SLAB COLUMN CONNECTIONS WITH SHEAR REINFORCEMENT

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Abstract

Flat-slab column connections are susceptible to brittle failure, which lead to the necessity of improving ductility and ultimate strength. In case of edge connections, the behaviour at ultimate state is highly influenced by nonsymmetrical distribution of stresses originated by a moment transfer between the slab and the column. The paper presents the test results of three full-scale reinforced concrete flat-slab edge connections with stud-rail shear reinforcement subjected to concentrated load. The in plane dimensions of the slabs are constant, 2200 mm x 2000 mm, whereas the slab thickness varies from 200 mm to 260 mm. A comprehensive analysis is made regarding the behaviour at failure by means of analytical studies and a number of comparisons to code provisions.

Keywords: Edge flat-slab column connections, Failure analysis, Punching, Test campaign

1 Introduction

Flat slab system is known to be a building system with large applicability due to its architectural and functional advantages. Among them one can emphasize: clear space between the stories, flexibility, reduced erection time, construction simplicity, reduced quantity of formwork, less labour and more economical span range design. From an engineering point of view, the behaviour is complex. It is governed by large deflections at mid-span at serviceability and by high concentration of stresses near the connection with column and limited lateral load capacity, as a moment resisting frame at ultimate state. In case of edge connections the distribution of stresses around the column is uneven, therefore the behaviour is non-symmetric.

Several research studies regarding punching at edge slab connections have been made during the last two decades. Existing literature database gathers a number of 130 edge specimens. Only 40 tests have been found on slabs with shear reinforcement. Among the researchers that studied this topic are: Mortin & Ghali (1991), Rangan & Lim (1995), Sherif (1996), Hegger and Tuchlinksi (2004). The average slab effective depth of tested slabs is approximately 125 mm and solely 29% of the specimens had thicknesses over 200 mm. Accounting for previous research, the present study provides new test data regarding the behavior of edge flat-slab column connections with shear reinforcement. The paper summarizes the testing procedure, test results and presents a study upon the failure mode.

2 Geometrical configuration and test setup

Tests were carried out at the Concrete Laboratory of T.U. Cluj-Napoca, Romania. The test campaign comprised three edge flat slab-column connections. Slab in-plane dimensions were 2200 mm x 2000 mm and column cross-sectional dimensions are 300 mm x 300. The specimens varied in thickness: 200 mm, 230 mm and 260 mm. They were tested upside-down with a load introduction over the column , simulating the case without moment transfer to the slab. The slab was simply supported on seven bearing pads and clamped at one edge (figure 1b).

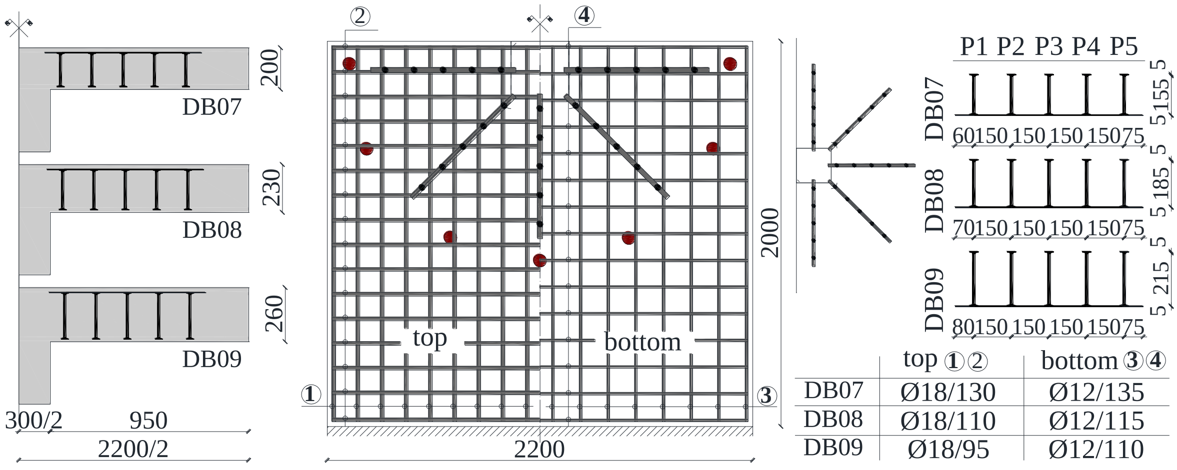


Fig. 1 a) Tested specimens, b) Reinforcement layout and dimensions

The flexural reinforcement ratio on the top face of the slab was $\rho_{l,top}$~1.2% and $\rho_{l,bottom}$~0.5%. on the bottom. All flexural bars were bent at ends at 90^o to avoid any possible bond failure. The steel grade was S500. The bars were arrayed along orthogonal directions. The clear cover for top reinforcement was 25 mm and for the bottom reinforcement was 15 mm.

The tested slabs were shear-reinforced with JDA Jordahl stud-rail shear reinforcement. The critical perimeter was reinforced with five radii of d_{bw}=10 mm bars. The double-headed anchors were made of S500 steel connected with a perforated strip made of structural steel. The position of the shear reinforcement was star like shape (figure 1b). Normal concrete C20/25 was used. Ready mixed concrete with a maximum agregate of 16 mm was used in all specimens. Concrete compressive strength and elastic modulus were obtained from cube samples (table 1b).

3 Test results

All test specimens behaved similarly throughout the entire loading process. Flexural cracking was firstly observed at low load values, staring from 80 to 100 kN (16% to 18% of $V_{u,test}$). Primary cracks had the tendency to develop from the column towards the middle support. Serviceability stage was reached for a the crack of about 0.4mm (56% to 64% of $V_{u,test}$). Due to the location of the supports, at bottom, cracks formed a circular pattern. Cracks developed firstly above the supports and progressively at later loading steps towards the column.

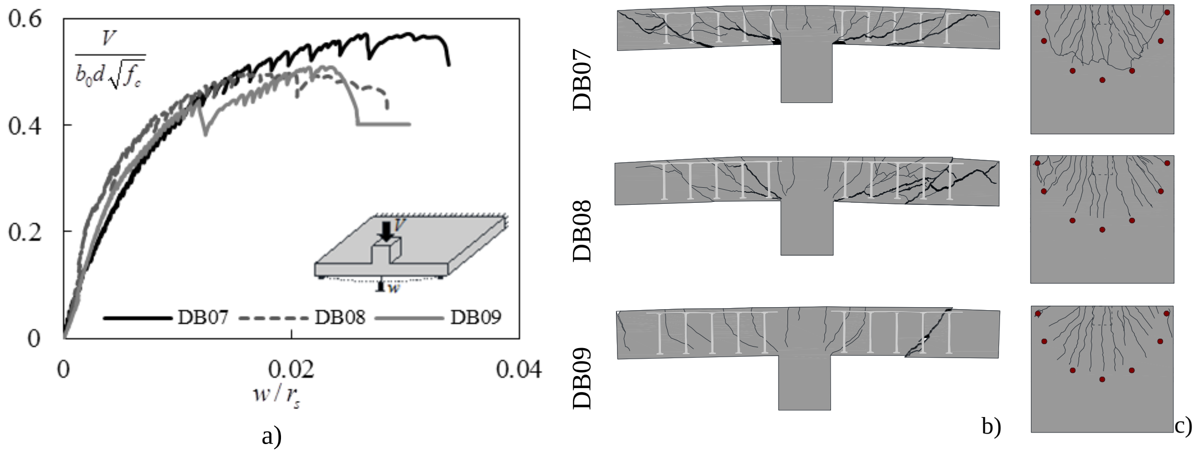
Table 1a
Specimen configuration

	l_x	l_y	h_{col}	c_1	c_2	h_f	d	d_v	$V_{u,test}$	$M_{u,test}$	Δ	FM
DB07	2200	2000	300	300	300	200	157	155	491	0.84	33.3	PI
DB08	2200	2000	300	300	300	230	187	185	551	0.11	31.0	PI
DB09	2200	2000	300	300	300	260	217	215	643	0.01	29.9	LSF

Table 1b
Material characteristics

	f_{cm}	$f_{ct,sp}$	ρ^{top}	ρ^{bot}	Material characteristics				distance s_w [mm]					$A_{sw,I}$
					E_c	$f_{y,top}$	f_{yw}	E_s	P1	P2	P3	P4	P5	P_i
DB07	22.9	1.65	1.22	0.49	24.7	583	525	200	60	210	360	510	660	393
DB08	24.7	1.74	1.25	0.50	d_g	$f_{y,bot}$	d_{bw}	E_{sw}	70	220	370	520	670	393
DB09	22.0	1.55	1.23	0.51	16	536	10	200	80	230	380	530	680	393

Units: forces [kN], moments [kNm], dimensions [mm], strengths [MPa], moduli [GPa], areas [mm2], reinforcement ratio [%], Δ – deflection [mm], FM – failure mode:PI – punching inside the shear-reinforced zone, LSF – local shear failure)



a) b) c)

Fig. 2 a) normalised applied load – deflection curve; crack patterns b) at the free edge c) top face

Only specimen DB07 showed a clear punching failure with the formation of a visible half-circular pattern on the top face. With slab thickness increase, cracks indicating the development of a punching cone were less perceptible. No punching cone was observed for DB09. A local shear failure occurred for this test with an inclined crack near the supporting plate (refer to figure 2b).

Figure 2a presents the normalised load-deflection curve and the influence of the thickness upon the ultimate strength of the connection, where b_0 is the control perimeter at *d/2* of the edge of the support and r_s refers to the distance from column axis to the l of contraflexure of radial moment (r_s*~985mm*). It can be observed that all specimens follow the same trend. A slight difference is recorded for DB07 regarding the deflection response related to r_s.

4 Failure analysis

In order to find out supplementary information regarding the failure mode and strength, analytical and numerical studies have been made. The study consisted of analyses using yield line method (Johansen, 1962), comparison to Eurocode 2 (2004) and Model Code 2010 LoAII and LoAIII (2012). Table 2 gathers the values obtained during the physical testing campaign, analytical flexural analyses and design code predictions.

Yield line analysis is a limit analysis giving an upper bound value of the ultimate bending strength of a slab, leading to the minimum punching load. To obtain the exact solution, a search of the optimum failure mechanism was made. The mechanism associated with the minimum failure load is compound of radial yield lines caused by negative bending on the top and positive circular lines on the bottom face (figure 3a).. According to yield line analysis none of the slabs failed in flexure.

Due to the fact that each design code has its own theoretical background, different results are obtained. Differences might come, among others, from the way that: critical perimeter, concrete strength, flexural reinforcement or transfer of unbalanced moment is accounted. Eurocode 2 proposes β coefficient that can be calculated in three ways. In this study a fully plastic distribution was assumed. Model Code 2010 proposes a k_e factor which depends on the eccentricity given by the bending moment and the basic control perimeter.

Table 2

Strength predictions according to Eurocode 2 and Model Code (levels of approximation II and III)

Test	V_{test} [kN]	V_{flex} [kN]	$\frac{V_{test}}{V_{flex}}$ [-]	$V_{cs,EC2}$ [kN]	$\frac{V_{test}}{V_{cs,EC2}}$ [-]	$V_{\max,EC2}$ [kN]	$V_{cs,MC,II}$ [kN]	$\frac{V_{test}}{V_{cs,MC,II}}$ [-]	$V_{cs,MC,III}$ [kN]	$\frac{V_{test}}{V_{cs,MC,III}}$ [-]	$V_{\max,MC,III}$ [kN]
DB07	491	690	0.72	290	1.69	1230	338	1.45	366	1.34	777
DB08	551	975	0.57	351	1.57	1729	421	1.31	459	1.20	1089
Average:					1.63			1.38		1.27	

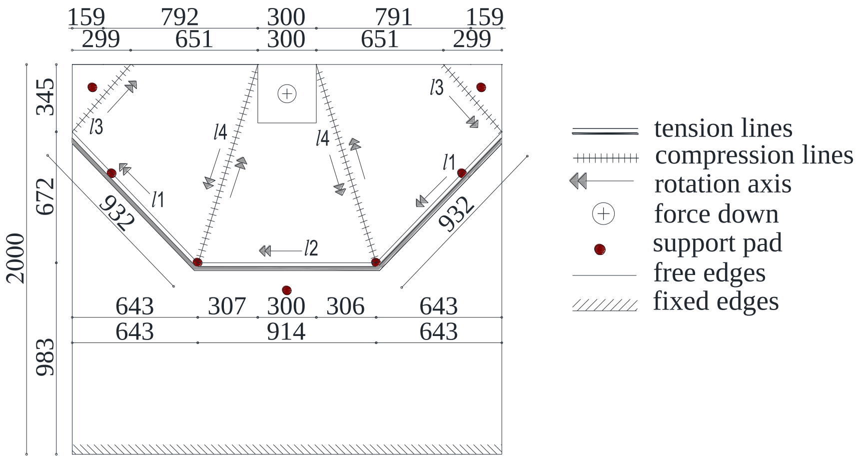


Fig. 3 Yield line pattern

Due to the local mode of failure of DB09, only the other two specimens are accounted in the calculations. According to Model Code 2010, the failure of DB07 and DB08 can be attributed to punching inside the shear reinforced area. It has to be mentioned that the conversion factor of 0.8 between cylinder and cube strength was considered; mean material strengths and no safety factors were used (γ_s=γ_c=1). Based on the results in table 2, Eurocode gives a more conservative approach with an average ratio of 1.63, whereas Model Code offers a better fit to the test results.

As observed in table 2, the level III of approximation of Model Code 2010, which accounts a refined design process, gives closer results to the test ones. In this case, the mean ratio between tests and code provision results is 1.27 on average, whereas in case of a coarse approach (LoA II) the ratio is 1.38 on average. Accounting a k_{sys}=2.8 parameter for stud-rails, the maximum punching capacity provided by MC2010 give higher result values that the ones obtained during testing.

5 Conclusions

The present paper summarizes the test campaign and the analytical analyses made on three edge flat-slab column connections with shear reinforcement. Based on the aforementioned, the following conclusions can be drawn.

1) Punching failure is potentially governing in edge columns of flat slabs even if they are shear-reinforced
2) Design codes tend to give conservative estimates in case of punching inside the shear reinforced area for the tests presented in this paper.
   - Eurocode 2 has the tendency to offer conservative results for the ultimate capacity of slabs at edge connections
   - Level II of Approximation of Model Code gives a more realistic fit to test results
   - The predicted results indicate a better estimation of ultimate bearing capacity when higher accuracy of analysis is used (LoA III).

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