Progressive collapse of a flat slab building

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Abstract. The possibility of progressive collapse in a flat slab was analyzed using the reinforced concrete structural design softwares Eberick and TQS. After the failure of a slab-column connection, the punching capacity of the slab was verified according to NBR 6118:2014, EUROCODE 2:2004 and ACI 318:2011 and the remaining capacity of the failed floor was assessed by the Yield Line Theory. Integrity reinforcement was also sized according to NBR 6118:2014, CEB:2010 and GSA:2013. It is concluded that there is a possibility of propagation of the failure after an initial failure by punching, and the presence of integrity reinforcement can help preventing the propagation of a localized failure.

Keywords: Progressive collapse, Flat slab, Punching shear, Yield Line Theory.

1 Introduction

Flat slab is a structural system that rests on columns, without the presence of beams. A failure located in a slab-column connection causes a redistribution of reactions and moments in the building and overloads in other supports that can also collapse. Thus, a sequenced effect known as progressive collapse appears, a phenomenon that can generate a large-scale failure in the structure.

Progressive collapse is a propagation of damage that has occurred to a structural element due to improper renovation of structure, fire, vehicle collision, substandard material, design and/or execution errors. One possibility in preventing the occurrence of this phenomenon is creating alternative load paths, an adequate continuity and ductility in the structure [1].

Flat slabs can be very vulnerable at their connections with columns and there is a great concentration of stress at the supports [2], therefore, it is necessary to study the failure mode by punching shear in the slab that occurs with few disorder warnings or pathology.

In this work, both the pattern of a reinforced concrete flat slab building and the possibility of progressive collapse of structure are evaluated, considering the total or partial loss of a structure support due to the slab punching in a specific flat slab building.

2 Structure Analysis

One six floors flat slab building that was studied before using with TQS software by Martins [3, 4] was analyzed with Eberick software. The slabs were 16 cm thickness, the ceiling height was 273 cm, and a more rigid core was formed by the staircases and by the elevators, and openings at the floor for the passage of hydro-sanitary pipes were present. The flat slab floor plan is presented in Fig.1. The concrete resistance was 25 MPa and the loads were according to NBR 6120 (Table 1). Some simplifications on the floor layout had to be performed as Eberick has limitations for analyzing internal columns with beams in flat slabs buildings structures.

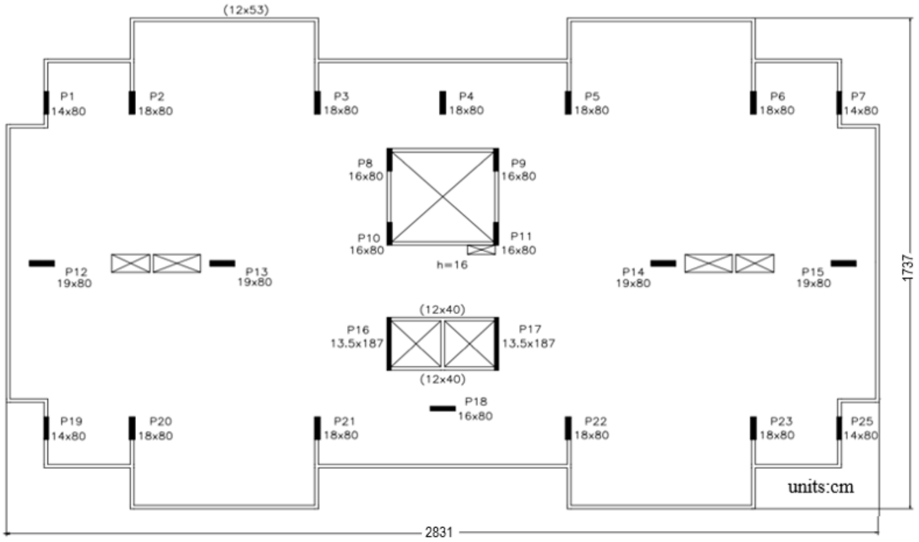

Figure 1. Floor plan of the analyzed structure [3, 4]

Table 1. Load used

Overview	Load
Dead load (distributed):	(kN/m²)
Slab (h = 16.00 cm)	25.00 x 0.16 = 4.00
Cladding	0.50
Wall	1.92
Plaster lining	0.25
Live load:	(kN/m²)
Load in residential building	1.50
Total(kN/m²)	8.17

2.1 Punching Assessment

Table 2 presents the columns loads and moments obtained with Eberick, about 18% different from the results from the TQS [3, 4], depending on the defaults used. Among the most susceptible slab-column connections, P4, P12 and P13 were then checked for punching shear in accordance with NBR 6118:2014, EUROCODE 2:2004 and ACI 318:2011 standards, as shown in Table 3.

Table 2. Reactions and moments related to the support of the structure without damage

Column	Nk (kN)	M1 (kN.m)	M2 (kN.m)	Column	Nk (kN)	M1 (kN.m)	M2 (kN.m)
P1	119.3	1.4	50.2	P13	254.2	18.6	4.6
P2	264.0	1.5	37.8	P14	255.0	23.4	4.6
P3	267.0	7.1	47.6	P15	216.0	10.0	3.8
P4	105.2	1.1	31.7	P16	95.3	2.3	63.6
P5	268.1	0.9	47.6	P17	96.2	0.1	64.0
P6	265.0	2.3	38.1	P18	121.1	2.6	48.4
P7	117.0	1.6	49.9	P19	264.5	1.2	41.6
P8	85.3	0.6	3.2	P20	282.8	6.3	61.8
P9	84.4	0.7	4.0	P21	112.2	0.9	1.4
P10	94.7	1.8	16.0	P22	282.3	0.0	62.3
P11	99.2	2.0	13.1	P23	264.4	2.0	40.7
P12	215.1	5.8	3.5	P24	119.2	2.0	47.8

Table 3. Loads and dimensions for checking the slab-column connections

Column	Nk (kN)	M1 (kN.m)	M2 (kN.m)	C1 (m)	C2 (m)	d (m)
P4	105.2	1.1	31.7	0.18	0.80	0.13
P12	215.1	5.8	3.5	0.19	0.80	0.12
P13	254.2	18.6	4.6	0.19	0.80	0.12

Table 4 presents the punching stresses for the analyzed columns and for P12 and P13 shear reinforcement respectively in 3 and 4 layers with a total of 3.74 cm² per layer was needed.

Table 4. Shear stresses at critical section

Codes	Stress (MPa)	P4	P12	P13
NBR 6118	τ_{rd3}	-	0.76	0.82
	τ_{sd3}	-	0.51	0.66
	τ_{rd2}	0.66	1.12	1.25
	τ_{sd2}	0.57	0.75	1.09
	τ_{rd1}	4.34	4.34	4.34
	τ_{sd1}	1.35	1.49	2.65
EUROCODE	τ_{rd3}	-	0.62	0.67
	τ_{sd3}	-	0.55	0.66
	τ_{rd2}	0.54	0.99	1.12
	τ_{sd2}	0.57	0.75	1.09
	τ_{rd1}	4.50	4.50	4.50
	τ_{sd1}	1.35	1.49	2.65
ACI	$\emptyset V_{n2}$	-	1.47	0.64
	V_{u2}	-	0.77	0.58
	$\emptyset V_{n1}$	0.92	0.64	1.58
	V_{u1}	0.58	0.49	1.19

τ_{rd3} e τ_{sd3} are the strength and acting stresses in the critical section C''
τ_{rd2} e τ_{sd2} are the strength and acting stresses in the critical section C'
τ_{rd1} e τ_{sd1} are the strength and acting stresses in the critical section C
$\emptyset V_{n1}$ e V_{u1} are the strength and acting stresses in the critical section C',
$\emptyset V_{n2}$ e V_{u2} are the strength and acting stresses in the critical section C''

As shown in Figure 2, the acting stress was greater than the strength stress at P4 according to EUROCODE standard. Thus, it is necessary to increase the connection capacity with the use of shear reinforcement or the increase regarding the dimensions of the columns and or the slab.

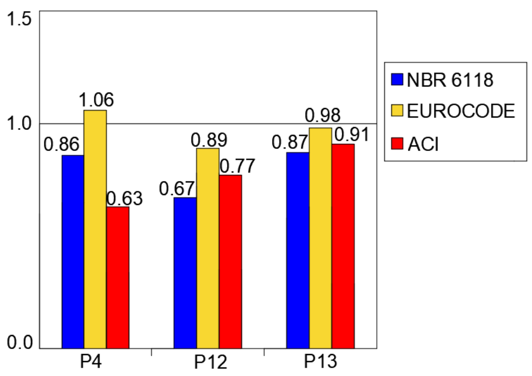

Figure 2. Acting and strength stress ratio at the critical section

2.2 Integrity reinforcement

Table 5 presents the integrity reinforcement steel areas according to the NBR 6118:2014, GSA:2013 and to CEB:2010 and Fig. 3 show the comparison on the deferent codes obtained areas. The steel area obtained by GSA does not vary according to the column reaction, as the steel area depends on the slab section and material strength, and for P12 it is close to the NBR 6118:2014.

Table 5. Integrity reinforcement area

Column	NBR 6118 (cm²)	GSA (cm²)	CEB (cm²)
P4	4.35	8.66	3.22
P12	8.90	8.66	6.59
P13	10.52	8.66	7.79

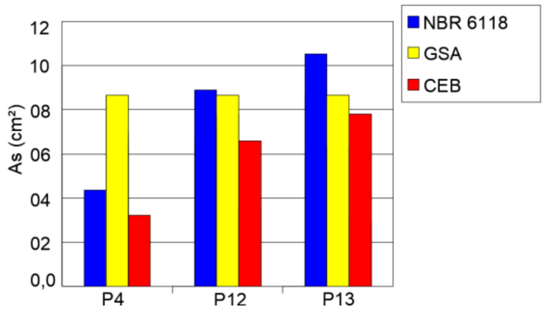

Figure 3. Comparison of reinforcement steel areas

2.3 Post punching pattern

Table 6 shows the reactions and moments that would be obtained if the flat slab had failed by punching shear at column P13, with its reaction going to zero and the columns removed at the calculation and the variations on the columns reactions is shown in Figure 4, and as seen in this figure the most affected connections would be at columns P2 and P12, increasingly the load respectively 18.4% and 31.8%. The remaining connections were then checked to investigate if the failure initiated at the region of column 13 could propagate horizontally to other regions.

Table 6. Reactions and moments of support after P13 punching

Column	Nk (kN)	M1 (kN.m)	M2 (kN.m)	Column	Nk (kN)	M1 (kN.m)	M2 (kN.m)
P1	98.6	0.4	51.6	P13	-	-	-
P2	312.7	1.0	25.5	P14	257.4	25.1	4.7
P3	307.7	8.9	10.3	P15	215.9	9.9	3.8
P4	92.4	0.6	31.3	P16	98.3	3.9	106.7
P5	269.2	0.8	47.7	P17	82.9	0.4	51.2
P6	265.1	2.3	38.1	P18	100.4	1.3	50.5
P7	117.4	1.5	49.8	P19	311.6	1.0	20.0
P8	94.0	1.2	5.9	P20	318.9	7.0	16.9
P9	81.6	0.8	3.7	P21	95.0	4.8	1.4
P10	161.2	5.4	28.5	P22	284.4	0.0	61.2
P11	72.9	2.1	7.8	P23	264.0	2.0	40.6
P12	283.5	113.3	4.9	P24	119.6	1.9	47.8

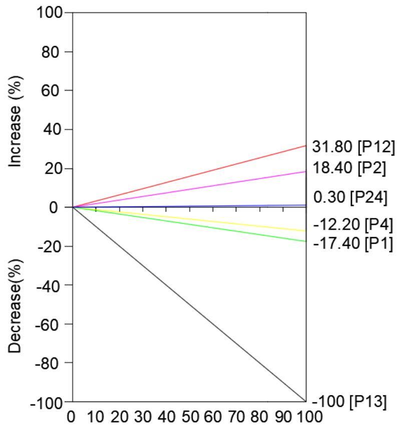

Figure 4. Variation of support reactions after failure at P13

The question that remains is if P12 connection could cope with a 31.8% reaction increase. However, this reaction would come down to 27.0% if a residual reaction of 15% would be assumed at the original damaged connection (P13), and the presence of an integrity reinforcement could allow a residual reaction strength of 60% at P13, decreasing the load rise to 12.7% at connection P12. Figure 5 shows the comparison between the stresses at column P12, respectively for a residual strength of 60% of the original reaction and for no residual reaction, corroborating the importance of the use of the integrity reinforcement in flat slab connections. It is seen that there is a possibility of progressive collapse at the structure if no residual resistance is present at the connection, and that the highest stresses were obtained with Eurocode.

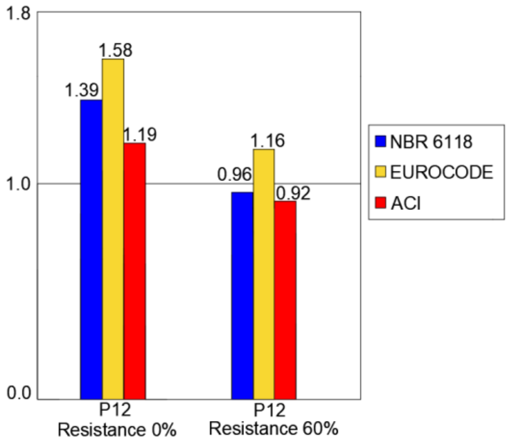

Figure 5. Comparisons in stresses at column P12 for zero and for 60% of residual reaction

2.4 The remaining capacity of the failed floor by the Yield Line Theory after punching around P13

The remaining capacity of the failed floor was determined after the punching failure of an internal slab-column connection (P13), without considering any residual strength. The proposed Yield Line configuration pattern shown in Figure 6 was proposed by Martins [3, 4], with a positive yield line was drawn in the central region of the slab and a negative yield line "cutting" the supports around the failed internal connection. It is assumed that the beams located at the end of the slab would not have enough strength nor rigidity to modify the outline of the positive lines.

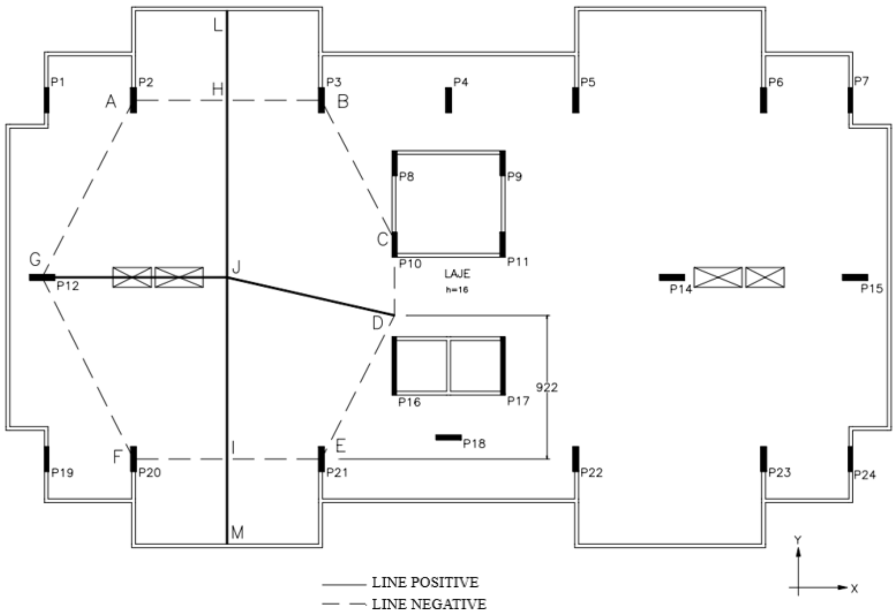

Figure 6. Positive and negative yield line configuration

It was assumed that point “J” had a virtual unitary displacement for the calculations and the positive and negative yield line analysis can be calculated by the Virtual-Work Method considering the external work required by a load uniformly applied to a slab or floor and the internal work used by the slab to deform.

The external work can be determined by the product of the failure load and the volume of the slab displaced after applying the virtual displacement and the internal work can be calculated considering the moment strengths and rotation of the slab, and the yield line must cut perpendicularly the reinforcement present in the slab, considering that the reinforcement had a sufficiently anchorage length [5]. Figure 7 shows the negative moment strengths at the X and Y directions.

Equalizing the load external work with the internal slabs yield lines work it is possible to reach the collapse load of 7.64 kN/m². It is verified that the calculated collapse load was 6.50% lower than the predicted load to act on the slab (8.17 kN/m²). Thus, it can be concluded that there probably would happen a progressive collapse of the structure with punching of the slab around P13 connection.

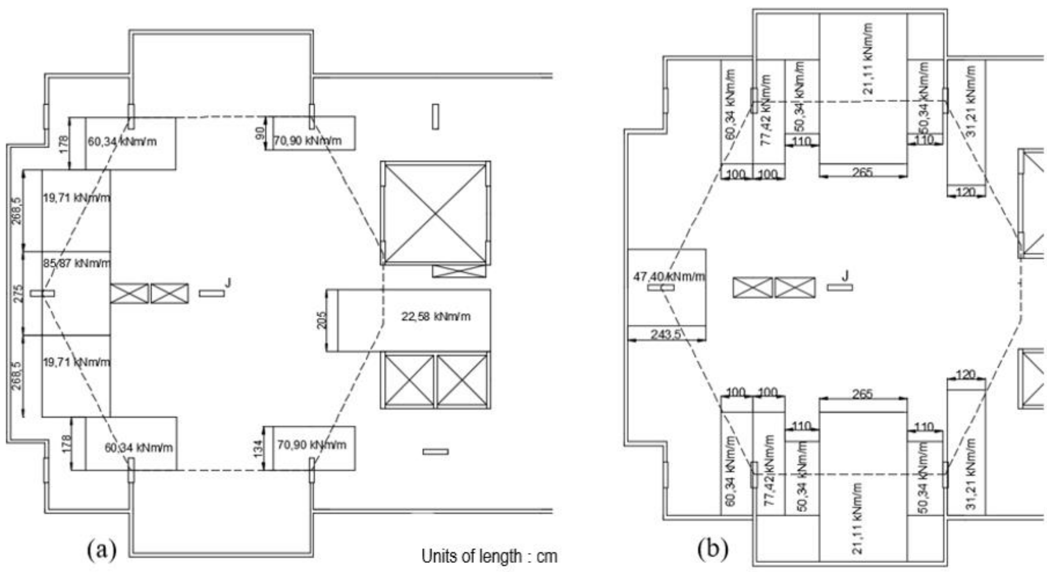

Figure 7. Negative moment strengths in the x (a) and y (b) directions

There are several possible collapse mechanisms according to the yield lines, but there is no need to look for another failure configuration as the slab is already in a critical condition. The slab could of course have a greater collapse load if there was any residual strength of the slab-column connection as seen before.

3 Conclusions

Columns / flat slabs should be designed and detailed to prevent the possibility of a progressive collapse.

The integrity reinforcement is needed for a good performance of the connection and to increase the residual resistance of column / slab connections, may avoiding progressive collapse.

The performance of the connections depends on the post punching capacity of the neighbors' connections and on the flexural capacity of the floor.

The global behavior of the floor can be assessed by Yield Line Theory.

For the example analyzed there was a possibility of a progressive collapse, considering no residual strength of the damaged connection.

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