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Analysis of the progressive collapse of a reinforced concrete structure using the finite element method

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Abstract. The loss of a load support element due to an extreme event can result in the progressive collapse of an entire building. There is a great complexity involved in modeling the behavior of a structure subject to this type of event, which makes it difficult for designers to access it. This work realized successive analysis of structural models of a reinforced concrete building, adopting a progressive complexity to each of them, using the alternative path method. At the end, the effectiveness and importance of each of the analysis steps employed were analyzed.

Keywords: Progressive collapse, Finite elements analysis, Reinforced concrete buildings

1 Introduction

Buildings with the most diverse purposes are subject to extreme events, such as vehicle impact, explosions, and sudden loss of members essential to their stability. The loads from such events are known as low probability and high damage, making it difficult to predict their occurrence and measure their magnitude. Progressive collapse can be defined as the process in which a localized damage leads to chain failure, causing the fall of an entire building or a large part of it. The sudden loss of an essential member due to exceptional loads can trigger the progressive collapse of a structure, causing enormous human and economic damages.

The press has spotlighted a significant number of cases of progressive collapse over the past few decades. The collapse of the Ronan Point building in London (1968) is considered to be the first documented case of progressive collapse and acted as a catalyst of changes in UK design codes for reinforced concrete structures, resulting in redundancy and ductility recommendations. Recent events, such as the bombing of the Alfred P. Murrah Federal Building in Oklahoma City (1995) and the collapse of the World Trade Center in New York City (2001), have prompted the interest in the research theme again [1]. All these events pointed to the need of buildings that can resist local damage without collapsing, especially those which are part of critical infrastructure of a city or are occupied by a large number of people.

Brazilian normative prescriptions related to progressive collapse of structures are discreet. The Brazilian design standard for reinforced concrete structures [2] deals succinctly with the topic, without defining or characterizing it. The standard does not establish an assessment criterion of the risk which the structure would be subject or describes an implicit mechanism in its calculation model of the reinforcements that contributes to its design improvement. By establishing design procedures and safety limits the standards are responsible for guaranteeing safety. It is then necessary that these mechanisms are evaluated and quantified in structures designed according to the national criteria.

The complex nature of the progressive collapse of reinforced concrete structures makes it difficult to model. Diverse variables are involved in the analysis of the phenomenon, such as: the consideration of large deformations, the adoption of macro or microelements in the finite element model, the dynamic or static modeling of the problem, and considerations about the physical properties of concrete and steel, the properties of the links between materials and structural elements, the time and damage dependent characteristics [1]. The amount of details that may be considered in a model that aims to analyze this problem makes it difficult for designers and there is no consensus on what should be modeled for each kind of problem.

2 Progressive collapse analysis procedure

2.1 Problem approach

Two different approaches can be found in previous works and normative texts about progressive collapse evaluation, the indirect or direct approaches. The indirect method refers to design and detailing practices which increase robustness and guarantee ductility for the structure. In contrast, the direct procedure evaluates the structure behavior after losing a support element.

The alternative load path method (ALP) removes a support member and tries to demonstrate directly the structure robustness. The method aims to show the structure ability in redistributing additional forces in remaining members over areas subject to local damages. The ALP approach requires the evaluation of numerous scenarios, which combined can lead to distinct robustness solutions. These considerations include a refined review of damage (dynamic analysis), material nonlinearity, large deformations and link ductility. In practice, the detail level in ALP analysis depends on building risk classification, made by normative prescriptions and performance criteria.

2.2 Analysis procedure

Due to the need of a reliable methodology for progressive collapse assessment, Marjanishvilli [3] proposed a multiple steps procedure with increasing degrees of sophistication between them (Figure 1). The incorporation of successive analysis aggregates strong points of each step to the final result, providing safety decisions for engineers. The criteria adopted for inserting loads and acceptability of the model results comes from the GSA standard [4].

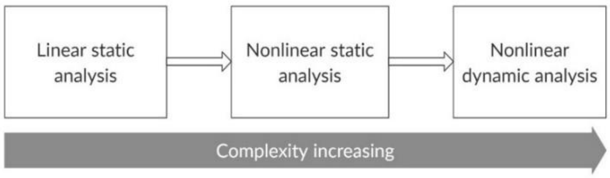

Figure 1. Analysis procedure

The process starts with a linear static analysis (LSA), which is more conservative due to the assumed simplifications. The lack of dynamic effects (inertial and damping forces) during this step leads to cautious results compared to the other types of analysis. However, this method has the easier modeling process and does not demand high computational costs, being a good starting point. Two load combinations were employed during the LSA, the first one was present in the directly affected area where eq. (1) is used, while the other acted on the rest of the model using eq. (2).

$$G_{LD} = \Omega_{LD}[1{,}2DL + 0{,}5LL] \tag{1}$$

$$G = 1{,}2DL + 0{,}5LL \tag{2}$$

The nonlinear static analysis (NSA) is an iterative method with increasing load steps applied to the structure until the maximum displacement. The method allows structural members to reach beyond the elastic limit, a situation that happens when they are subjected to extreme situations as the loss of a support element. In contrast, the modeling process is more complex than the first presented and can still result in cautious results. As in the previous step, two combinations of loads were employed at this time, in the area directly affected area where eq. (3) was used, while in the rest of the structure eq. (2) was employed.

$$G_N = \Omega_N[1{,}2DL + 0{,}5LL] \tag{3}$$

Dynamic analysis is more realistic because it accounts for inertial and damping forces, resulting in numerical models with closer results to those observed experimentally. The first one to be performed, following the complexity increase, is the dynamic linear analysis. Its use is indicated for structures for which large displacements are not expected. The nonlinear dynamic (NDA) analysis is the last performed step due to its complexity. The geometric and physic nonlinearities can be considered during this step, reaching the most realistic results.

Nevertheless, this analysis has a big computational cost involved. Due to the structural behavior expected by the structure, the linear dynamic analysis was not performed in this work. The only combination of loads used in this model is the same as shown in Equation (1).

The acceptance criteria of results were based on demand-capacity ratio (DCR), as predicted by standard text [4]. The DCRs presented by columns and beams are obtained by eq. (4) and must be greater than 1, Q_R is the expected strength of the element and Q_D is the resulting action in it.

$$DCR = \frac{Q_D}{Q_R} \tag{4}$$

The validation of results is a crucial point of analysis. Marjanishvilli [3] proposes that results from more complex analyzes be validated by results from previous steps, exploiting to the maximum the similarities between the models. The adoption of such methodology makes it possible to inherit the main advantages of simple procedures and to add to them the robustness of the answers coming from a more complex analysis.

A single analysis cannot be enough to validate an evidence, as some sensitive parameters must be varied within the same model until their importance is measured and correctly assimilated [1]. Among the parameters analyzed will be: finite element mesh size, increment size, convergence criteria and physical properties of materials in non-linear analyzes. The validation of the results will be based on numerical evidence and previous results from other researchers.

3 Reference models analysis

The validation of numerical models for the analysis of the progressive collapse phenomenon is a critical point of this work. The validation of the results of the reference models employed in this work was expected in the analysis procedure and was recommended by other authors [1] [3]. This step had as the main objectives the proof of effectiveness and increased confidence in the modeling strategy adopted. This stage of the work searched for models that were experimentally validated. Factors as material uncertainty and parameters involved in modeling served as a filter, so we selected works with the greatest amount of information.

The reference model chosen was analyzed by Sasani *et al.* [5]. The work showed numerical and experimental results about progressive collapse resistance of a reinforced concrete building. To evaluate this phenomenon, the authors removed four columns from the structure and measured the vertical displacement at certain points. The 11-story building has a discontinuous structure. Transition beams were present at the end of the first floor, which was succeeded by flat slab pavements. The structural layout employed in the model was complex (Figure 2), having several beams and columns transverse sections. The materials that compose the model were obtained through experimental results (Table 1).

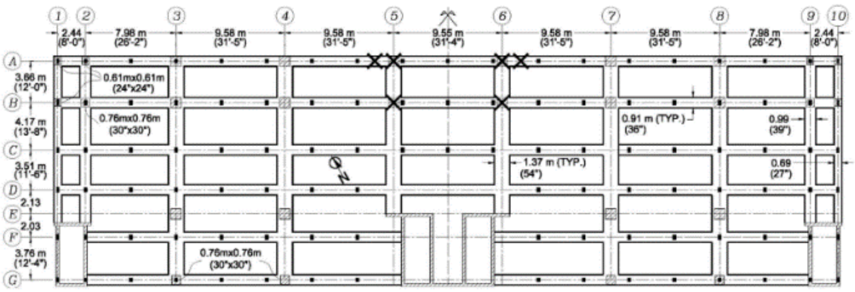

Figure 2. Plan of structure of second floor [5]

The model was subjected to a nonlinear dynamic analysis. The structure was analyzed without the presence of external loads, under its self-weight only. Four columns and two beam segments were removed from the model. The nonlinear behavior of beams and columns was modeled with FPH (fiber plastic hinges) placed along the member.

Table 1. Element features of validation model

Element features						
Element	Geometric	Physical				
	Transverse sections (cm)	Material	fck (MPa)	fy (MPa)	E (MPa)	
Beams	91x206 137x206 61x206	Concrete Steel rebars	27,6 -	- 517	24800 200000	
Columns	61x61 76x76 91x91 28x33 28x41	Concrete	34,4	-	27500	
Slabs	15 (thickness)	Concrete	27,6	-	24800	

Despite the information presented in the article, some parameters adopted during the modeling step were not clearly discussed. The presence of reinforcing rebars in the transverse sections of the elements has not been clarified at all, and therefore in the validation model only the beam placed along the B-axis had reinforcing rebars. Information about the mesh refinement was not provided, so the elements in the validation model had closer dimensions to the ones shown in the analytical second floor model, which were 30cm.

The discretization of elevator boxes was not discussed in the text. The authors adopted a 0,05 proportional damping ratio in the first natural frequency of the structure, so a modal analysis was needed in order to discover the correct value. The way these elements were modeled interfered in that number and the speed with which energy was dissipated in the system.

The spacing between FPH plastic hinges is crucial for the correct modeling of the loss of stiffness in the member which they are placed. Although information was provided about the length of these elements, it was not clear how much was used in each member. Four FPHs were adopted for each element in the validation model.

The critical region was bounded by axes A and D and lines 4 and 7, close to the removed columns. Similar to the strategy adopted by the authors, the slabs included in this region were modeled using beam-column elements with four FPHs. The rectangular section of these elements in the validation model was 120x15cm, applied using the analytical plan. The torsional stiffness of these elements has been reduced by half.

The first compared results were the vertical displacements at nodes A6 and B6. It was noted that the maximum deflection, which occurs at 0.19s (Figure 3), was bigger than that obtained by the model by Sasani *et al*. [5] in a similar analysis, about 15% higher (Table 2). The energy dissipated by the validation system occurred in a similar way to the reference model, with the maximum displacement occurring almost at the same time and converging as quickly to the stabilization point.

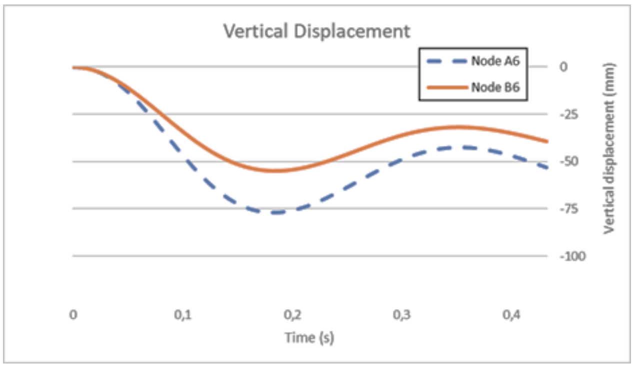

Figure 3. Vertical displacement of nodes A6 and B6

Table 2. Comparison between maximum vertical displacements

Maximum vertical displacement (mm)			
Node	Sasani *et al.* (2011)	Validation model	Difference
A6	66	75,1	13,79%
B6	46	52,9	15,00%

Other compared data between the models were the maximum positive and negative bending moments. The results were obtained between lines 7 and 6 of axis B (Table 3), with greater variation in positive moments.

Table 3. Comparison between bending moments on B axis

Comparison between bending momennts - B axis			
Instant	Validation model (kNm)	Sasani *et al.* (2011) (kNm)	Difference
0,00s	-1482,15	-1350	8,90%
0,19s	-12558,41	-14000	-11,50%
	9974,47	12000	-16,90%
Final	-9110,13	-9300	-2,10%
	7907,42	10600	-25,40%

The axial loads on the columns were also compared. For this purpose, the values of these loads were observed in the models before and after the removal of the support members. The results were similar to those obtained by the reference (Table 4).

Table 4. Variation of axial loads in columns

Variation of axial loads in columns						
Column	Before the damage (kN)			After the damage (kN)		
	Sasani *et al.* (2011)	Validation	Difference	Sasani *et al.* (2011)	Validation	Difference
A7	-930	-1097,90	-18,05%	-3470	-3775,44	-8,80%
A8	-1070	-1026,25	4,09%	-90	-109,61	-21,79%
B7	-4180	-4720,62	-12,93%	-9240	-9222,25	0,19%
B8	-4360	-4304,79	1,27%	-3380	-2933,99	13,20%

The results were satisfactory and within an acceptable range. The absence of certain information related to the modeling process was a key point to the distancing between the validation and reference model results. The resulting data showed a similar behavior related to the load redistribution in the structure after the loss of a load bearing element.

4 Proposed model analysis

4.1 Building description

The studied model was a 10-story reinforced concrete building composed by beams, columns and slabs (Figure 4). The effective length between the columns in the short side of the building was 4m and the bigger side had 7m. The first-floor columns were 5m tall while the others were 3.5m. The beams had transverse sections of 20x60cm, the columns had sections of 40x70cm and the slabs had 15cm thickness.

The structure was designed considering accidental and permanent loads (Table 5). The superstructure was designed with Brazilian normative criteria [2] which resulted in the reinforcing areas shown in Table 6. The concrete chosen for the model was C30, which had elasticity modulus equal to 27000 MPa, Poisson's ratio equal to 0,2 and estimated compressive strength of 30 MPa. The reinforcement steel chosen for the model was CA-50, which has an elasticity modulus equal to 210 GPa, Poisson's ratio equal to 0,3, yield and ultimate stresses of 500 MPa and 540 MPa, respectively.

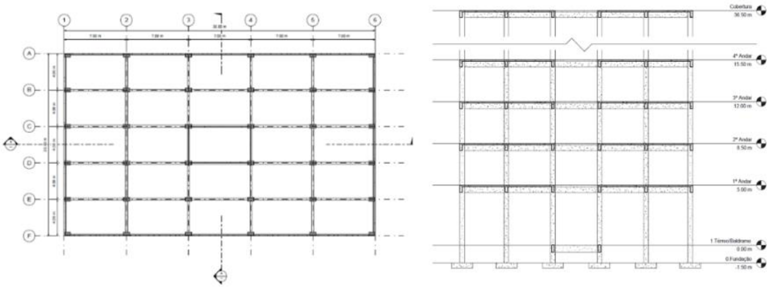

Figure 4. Structural plan of second floor and section

Table 5. Loads applied on the structural model

Loads present on the structure			
Permanent loads		**Accidental Loads**	
Concrete weight	25 kN/m³	Office	2,5 kN/m²
Ceramic mansory	5 kN/m		
Drywall	1,45 kN/m	Roof	1 kN/m²
Floor covering	1 kN/m²		

Table 6. Reinforcement ratio of the members present in the model

Reinforcement ratio			
Element	Concrete	As (cm²)	As' (cm²)
Ground longitudinal beam	C30	2,50	2,50
Ground transverse beam		2,50	2,50
Edge longitudinal beam	C30	3,23	5,00
Edge transverse beam		2,50	4,00
Internal longitudinal beam		6,00	10,00
Internal transverse beam		2,50	6,30
Superior longitudinal edge beam	C30	2,26	4,30
Superior transverse edge beam		2,50	2,50
Superior longitudinal internal beam		6,00	10,00
Superior transverse longitudinal beam		2,50	6,30
Column (ground floor)	C30	37,80	-
Column		15,00	-

4.2 Finite element model description

Two types of macro-elements were employed to discretize the finite element model. Tridimensional beam-column elements were used to model the beams and columns present in the structure, the elements had six degrees of freedom in each node and its formulation includes axial, bi-axial, torsional and shear deformations. Its physical nonlinear behavior was incorporated into the model by employing fiber plastic hinges (FPH) along the length of the elements. The presence of reinforcement rebars along the element provides more stiffness to the member and was adopted. The slabs were discretized with four node thin shell elements, with each node having six degrees of freedom. The sizes of the shell elements were obtained through a sensitivity analysis, which resulted in a 30cm element side.

4.3 Analysis of the results

The analysis was made after removing the P31 corner column located in the ground floor of the model. The vertical displacement of the node immediately above the removed column was obtained in different analyzes (Figure 5). The bending moments and their respective DCRs were obtained and presented the lowest values in the nonlinear dynamic analysis (Table 7). Finally, axial loads were obtained on the columns of the critical region adjacent to the removed element (Table 8). The lowest internal reactions observed in the most realistic analyzes happened by considerations in the load combinations employed in these stages, which were less conservative than in other steps. The columns and beams were closer to reaching the expected DCRs in nonlinear static analysis.

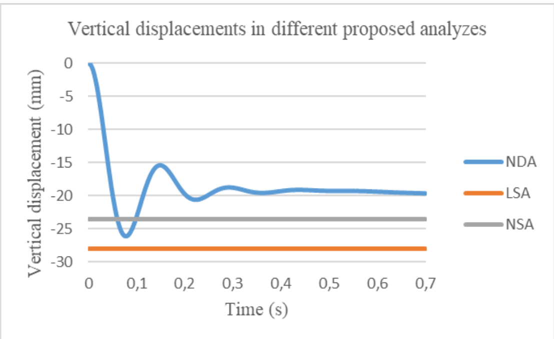

Figure 5. Vertical displacements results

Table 7. Bending moment results in primary beams and respective DCRs

Bending moment and DCRs				
Analysis	Beam	Md (kNm)	Mr (kNm)	DCR
LSA	V6	191,96	76,92	2,50
		-524,76	116,77	4,49
	V7	393,19	60,01	6,55
		-594,58	94,46	6,29
NSA	V6	109,60	76,92	1,42
		-266,31	116,77	2,28
	V7	178,48	60,01	2,97
		-270,30	94,46	2,86
NDA	V6	126,01	76,92	1,64
		-289,17	116,77	2,48
	V7	182,75	60,01	3,05
		-312,40	94,46	3,31

Table 8. Axial loads results in columns and respective DCRs

DCRs in columns				
Case	Column	Nd (kN)	Nr (kN)	DCR
LSA	P25	6344,77	4854,05	1,31
	P26	6364,48		1,31
	P32	5530,56		1,14
NSA	P25	4911,91	4854,05	1,01
	P26	5101,26		1,05
	P32	4321,53		0,89
NDA	P25	5732,79	4854,05	1,18
	P26	6082,73		1,25
	P32	5037,96		1,04

5 Conclusion

The DCRs decreased in both beams and columns as the degree of complexity of the analysis was increased. There was a significant reduction of the margin between the support capacity of the structure designed using the Brazilian normative rule and that necessary to meet the requirements of the GSA [4]. Despite the reduction of this difference, the structure with the physical characteristics coming from the ultimate limit state (ELU) dimensioning is not able to withstand the requirements foreseen by the GSA [4] to resist the progressive collapse. More complex analyzes result in a lower cost of adapting the structure, since the interventions necessary to meet the regulatory requirements are less. The results obtained by the successive analyzes fulfilled their roles of mutual validation predicted by Marjanishvilli [3].

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