Tests on a full-scale precast building for robustness assessment

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ABSTRACT

As modern society faces unprecedented challenges related to climate stability and conflict, the need to design more sustainable and resilient buildings is undeniable. Several studies show that building with precast concrete components can result in significant advantages with respect to sustainability. However, the fact that such structures exhibit clear lines of weakness at the joints between elements can make them more vulnerable to progressive collapse. Although this can be overcome by adequate detailing of connections, there exist several uncertain factors which may affect the interaction between different precast elements and the global behaviour of such structural systems. This means that experimental testing is usually required for safe development of adequate solutions. Despite this fact, there have been very few studies on the robustness of precast concrete structures when compared to research on cast-in-place and steel structures. As such, this work presents the results of an ambitious experimental campaign involving the sudden removal of 3 different columns from a specifically built 15 m x 12 m two-storey precast concrete building incorporating simple and practical solutions to enhance robustness.

INTRODUCTION

Today's buildings are increasingly exposed to the devastating consequences of extreme events caused by climate change, terrorist threats or their own ageing, among others. One of the greatest challenges facing architects and engineers today is therefore the design and construction of buildings that can survive an extreme event (e.g. vehicle impact, hurricanes, floods, explosions, terrorist attacks). Extreme events can cause initial-local failures of critical elements of a building structure, which are followed by a chain of failures in the rest of the building. This phenomenon, known as progressive collapse (Starossek 2017) usually causes significant material losses and fatalities (Ellingwood 2006). A resilient society needs buildings that can survive the effects of extreme events. As a result, researchers and practitioners alike have shown great interest in advancing knowledge on progressive collapse over recent years, as evidenced by the growth in the number of scientific publications on the subject (Adam *et al.*, 2018) and by the continuous renewal of design codes (Dusenberry 2022).

Precast concrete is being increasingly used nowadays due to significant advantages in terms of cost effectiveness, quality assurance, and durability, all of which can contribute to more sustainable building practices. However, the fact that precast buildings consist of distinct components joined together means that they are a priori characterised by a greater vulnerability to progressive collapse due to lines of weakness present at joints between components (Van Acker *et al.* 2012). Despite this, most research on the progressive collapse of buildings has so far focused on cast-in-place or steel/composite buildings. This is evident from the results of a search for journal articles indexed in SCOPUS on the progressive collapse of buildings, in which out of more than 600 articles found, only 6% concerned precast concrete structures.

The progressive collapse analysis of precast concrete structures presents several important challenges. Primordially, several uncertain factors can influence the behaviour of the joints between precast elements, which play an important role in determining the global structural response in extreme situations. In addition, accurately reproducing the behaviour of these critical parts using conventional models based on the Finite Element Method (FEM) typically requires the definition of parameters that cannot be easily determined experimentally. As a result, even if certain precast building systems can perform well against progressive collapse, it can be challenging to convince practicing engineers to adopt these solutions for the design of buildings when faced with structural robustness requirements.

To address this challenge, the PREBUST project funded by the Spanish Ministry of Science and Innovation, aimed to experimentally evaluate the effectiveness of simple solutions to improve the structural robustness of precast concrete structures through experimental and computational studies. The project involved building a full-scale structure of 15 m by 12 m with 2 floors, using typical precast concrete construction techniques with hollow-core slabs. Three separate tests were performed on the purpose-built structure involving the sudden removal of a corner column and of two different edge columns. Numerical models based on the Applied Element Method (AEM) were also built and calibrated using data on the structural response that was recorded during the three tests using an extensive monitoring system.

This paper first provides a brief overview of the design and construction of the full-scale building specimen. The tests performed as well as the monitoring scheme employed are then briefly described before presenting the computational modelling strategy used to perform simulations of different scenarios of interest. Key predictions made by simulations are then compared to measurements taken during the tests. Results of computational simulations of collapse are then discussed before summarising the main conclusions that can be drawn from the study so far.

BUILDING DESIGN & CONSTRUCTION

The building specimen constructed for the purpose of this study (Figure 1) has a rectangular shape with 6 bays in plan and two floors. The longest 15 m side consists of three 5 m spans while the shorter 12 m side consists of two 6 m spans.

Figure 1. Completed building specimen prior to testing (Photo courtesy of Manuel Buitrago).

The design of the building was based mainly on conventional techniques typically used for precast concrete construction with hollow-core slabs. The structure consisted of precast columns and beams used to construct a skeletal frame on which the hollow-core slabs were rested before pouring a topping layer. However, some simple design measures were introduced to improve structural robustness. This included casting the concrete columns with prepared sleeves (Figure 2) in order to allow continuous horizontal ties to be placed above all perimetral and central beams in each floor. These ties were then joined via couplers to prepared anchors in corner and relevant edge columns (Figure 3). Additional tying reinforcement was also placed between hollow-core planks based on recommendations in guidelines issued by the Institution of Structural Engineers (IStructE 2010) and the International Federation for Structural Concrete (Van Acker *et al.* 2012).

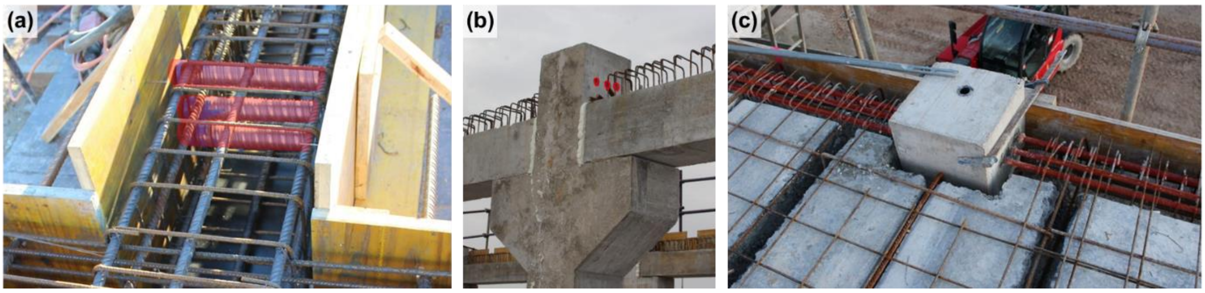

Figure 2. (a) Sleeves for continuous ties in edge column prior to casting; (b) Precast edge column in position prior to placing continuous ties through sleeves; (c) Edge column with continuous ties passing through it prior to casting topping layer (All photos courtesy of Manuel Buitrago).

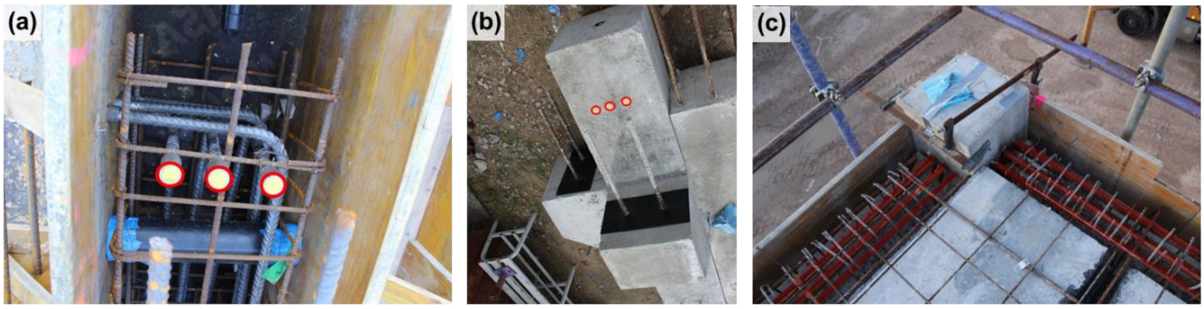

Figure 3. (a) Anchors and threaded couplers in corner column prior to casting; (b) precast corner column; (c) Corner column in position with ties connected to anchors prior to casting topping layer (All photos courtesy of Manuel Buitrago).

EXPERIMENTAL TESTING & MONITORING

As previously mentioned, three tests involving different sudden column removals from the first floor (Figure 4) were performed on the building using a metallic column with a special hinge mechanism (Figure 5). As shown in Figure 4, a corner column removal (Test 1) and two different edge column removals were executed. The edge column removed in Test 2 was part of a frame parallel to the alignment of the hollow-core slabs as well as part of the central frame along the longer building side on which the hollow-core slabs are directly resting. On the other hand, the edge column removed in Test 3 supported only part of a frame perpendicular to the alignment of the hollow-core slabs and on which they are directly resting. For the first test, a distributed load of 4 kN/m^2 was imposed on the bays adjacent to the removed corner column using concrete blocks. This corresponds to the minimum load combination to be considered for accidental design situations according to Eurocode 1 (CEN 2006). For tests 2 and 3, a distributed load of 6 kN/m^2 was imposed on the bays adjacent to the removed column using concrete blocks. This corresponds to the maximum load combination to be considered for accidental design situations according to Eurocode 1 (CEN 2006).

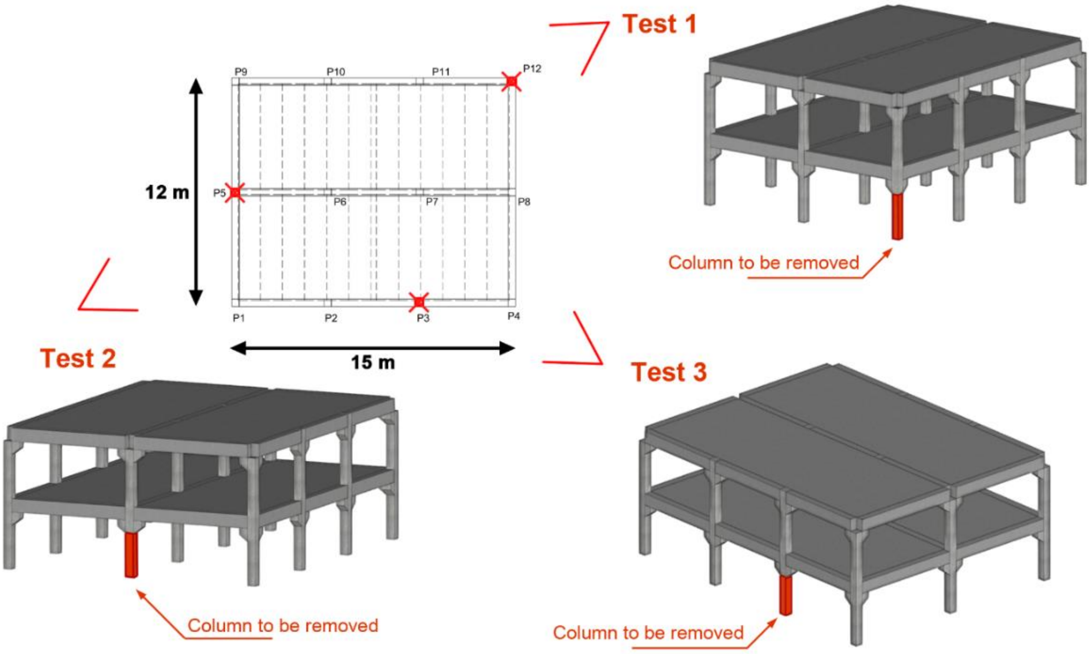

Figure 4. Schematic plan view of test buildings and location of columns suddenly removed during each test.

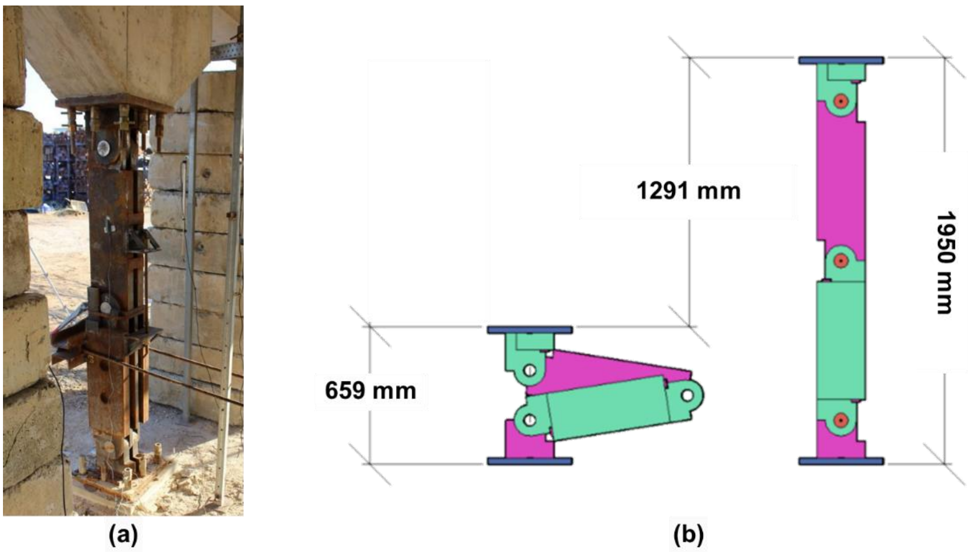

Figure 5. Purposely designed steel column with hinge mechanism for sudden column removal (Photo (a) courtesy of Manuel Buitrago).

The structure was equipped with many sensors to accurately capture the structural response before, during, and after the tests. This included 145 embedded strain gauges (Figure 6(a)) placed at key locations over reinforcing bars, 38 horizontal and vertical displacement transducers per test (Figure 6(b,c)) and a total of 9 accelerometers (Figure 6(d)) used for monitoring both the dynamic response as well as for performing ambient vibration tests before and after testing. For each test, 59 strain gauges were connected to the data acquisition system. Two of these correspond to gauges placed on either side of the collapsible steel column to monitor the change in axial strains due to unloading during sudden column removal while the remaining 57 sensors correspond to selected embedded strain gauges. Displacement transducers oriented horizontally were used for monitoring building drift or for measuring the horizontal extension or contraction at the top or bottom of beam-column joints or between hollow-core planks. Displacement transducers oriented vertically were used either to monitor the settlement of the foundations or to monitor the change in vertical displacement between floors. In the latter case, they were attached to spring-loaded telescopic bars (Figure 6(c)).

Figure 6. (a) Embedded strain gauge placed on rebar prior to being protected; (b) Horizontal displacement transducers; (c) Spring-loaded telescopic bars with transducers to monitor vertical displacement of first floor; (d) Accelerometer (All photos courtesy of Manuel Buitrago).

COMPUTATIONAL SIMULATIONS

As part of this work, the Applied Element Method (AEM) (Meguro & Tagel-Din 2000) was chosen for performing nonlinear dynamic simulations of the building structure after the removal of different columns. This involves discretising a structure into rigid body elements that have six degrees of freedom. These elements are connected by different types of distributed normal and shear springs, which are characterised by material-specific constitutive laws. Although this method is far less widespread compared to the FEM, several previous studies do exist that demonstrate its capability to accurately simulate all phases of collapse, including cracking, element separation, and collision (Tagel-Din & Meguro 2000b, 2000a). The geometry of the different components of the test building as well as reinforcement details were accurately reproduced, and material properties of concrete and steel were set based on results of tests performed on reinforcement bars and cylindrical specimens of concrete. The external damping assigned to the concrete material was computed based on the average values of two experimentally determined parameters: i) the damping ratio estimated using the logarithmic decrement method from vertical displacement records collected during all three tests and ii) the natural period of vibration modes estimated from acceleration records acquired during the tests.

When modelling precast structures with the AEM, it is very important to adequately set properties of the interfaces between precast components (Makoond *et al.* 2022, 2021). Friction coefficients for the interface were based on recommendations included in standards and guides (ACI 2007; CEN 2004; PCI 2010). It is also important to specify a reduced shear stiffness for the interface springs to allow for the localised deformations and cracking that occur at joint locations. This turned out to be the main parameter to calibrate in order to get a good agreement between experimentally observed and simulated structural response.

Two sets of nonlinear dynamic simulations were performed for each of the three column-removal scenarios corresponding to each test. The same load as employed for the experimental tests on the real building structure was imposed on bays adjacent to the removed column in the first set of simulations (Figure 7(a)). These simulations were used to calibrate and validate the modelling strategy. These second set of computations involved using the calibrated models in a series of successive simulations in which the imposed distributed loads were gradually increased until collapse could be observed (Figure 7(b)).

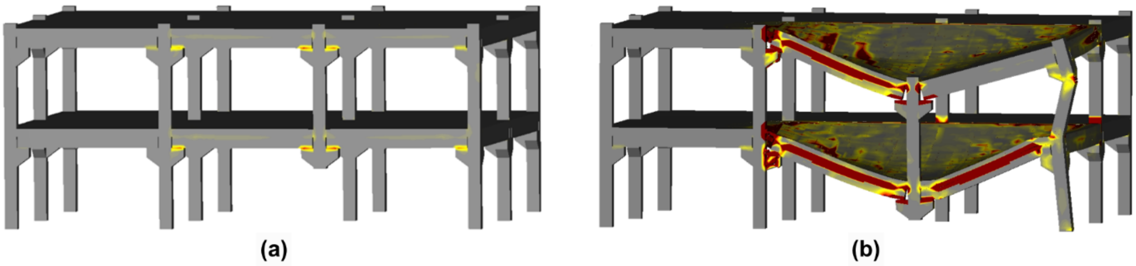

Figure 7. Simulation of column removal scenario corresponding to Test 3 under effect of experimental load (a) and collapse load (b).

RESULTS & DISCUSSION

Comparisons of the time series of vertical displacements predicted by simulations and recorded during the experimental campaign are shown in Figure 8. For each test, the displacements are shown for the locations where sensors were placed closest to the removed column on either side under beams. The average peak and residual vertical displacement of values measured by the three closest sensors to the removed column are also shown in Table 1.

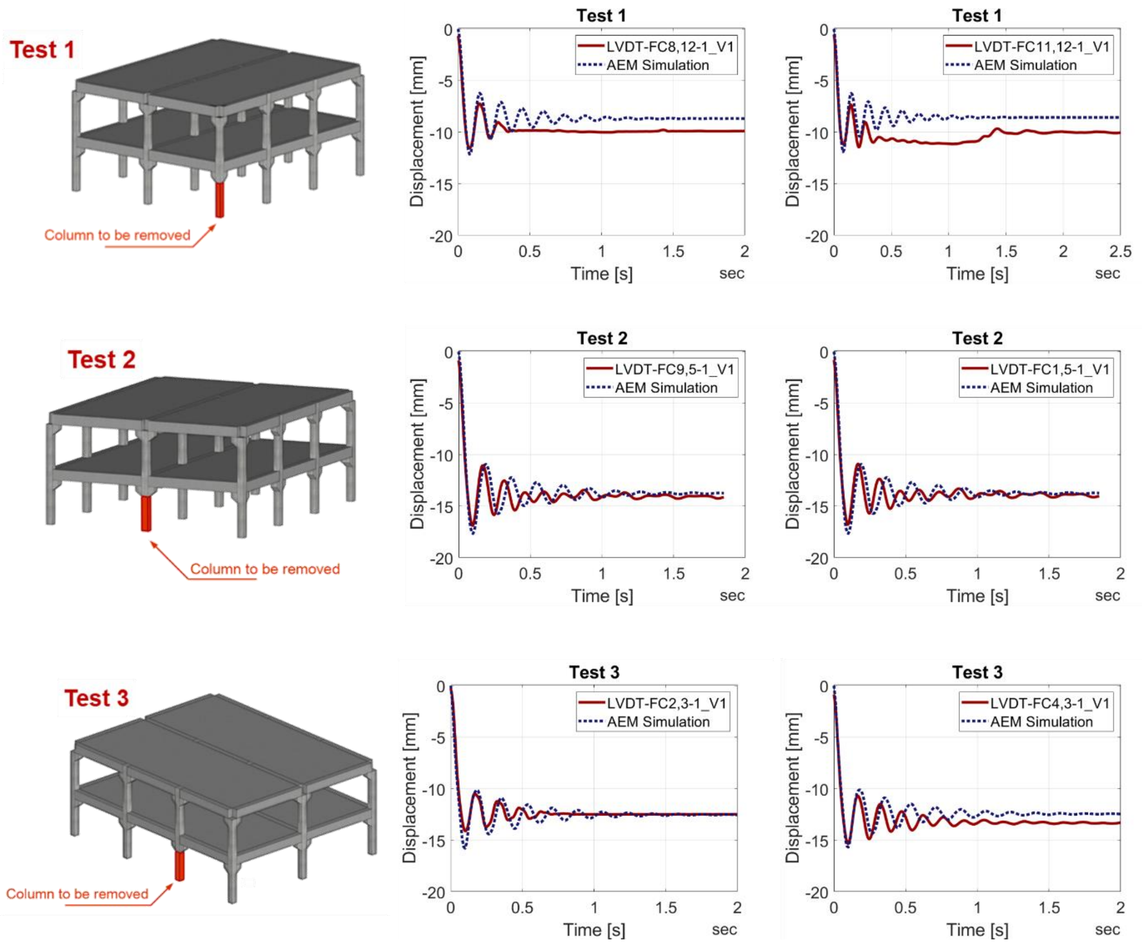

Figure 8. Comparison of predicted and observed dynamic response after sudden column removal for tests 1 to 3.

Table 1. Comparison of experimental and predicted values of maximum and residual vertical displacements.

	Average peak vertical deflection [mm]			Average residual vertical deflection [mm]		
	AEM	**Experimental**	**relative diff.**	**AEM**	**Experimental**	**relative diff.**
Test 1	12.01	11.62	-3%	8.60	9.88	13%
Test 2	17.62	16.56	-6%	13.73	13.82	1%
Test 3	16.12	15.06	-7%	12.96	13.13	1%

It is clear to see that there is a very good agreement between the predictions of the computational simulations and the corresponding values measured during the experimental tests. Figure 8 shows that the numerical model is able to accurately reproduce the damped oscillatory response recorded by the displacement transducers. The peak and residual vertical displacement values shown in Table 1 also reveal a very good agreement with the relative difference with respect to the experimental values always being lower than 10% with the exception of the residual vertical displacement measured for Test 1. It is important to highlight that a lower experimental distributed load was used for this case. It is also interesting to note that the peak displacement predicted by the model is always slightly greater than the measured experimental values whereas the opposite is true for the residual displacement.

As previously mentioned, the calibrated numerical model was then used to perform a second set of computations aiming to estimate the distributed load that would result in collapsing the structure after column removal. The results of this analysis for the column removal case corresponding to Test 3 is shown in Figure 9, while the estimated collapse loads for all three column-removal scenarios are summarised in Table 2. Note that the values shown in parenthesis in Figure 9 refer to the multiplier that needs to be applied to the load combination for accidental design situations (6 kN/m^2) to obtain the distributed imposed load used for each simulation.

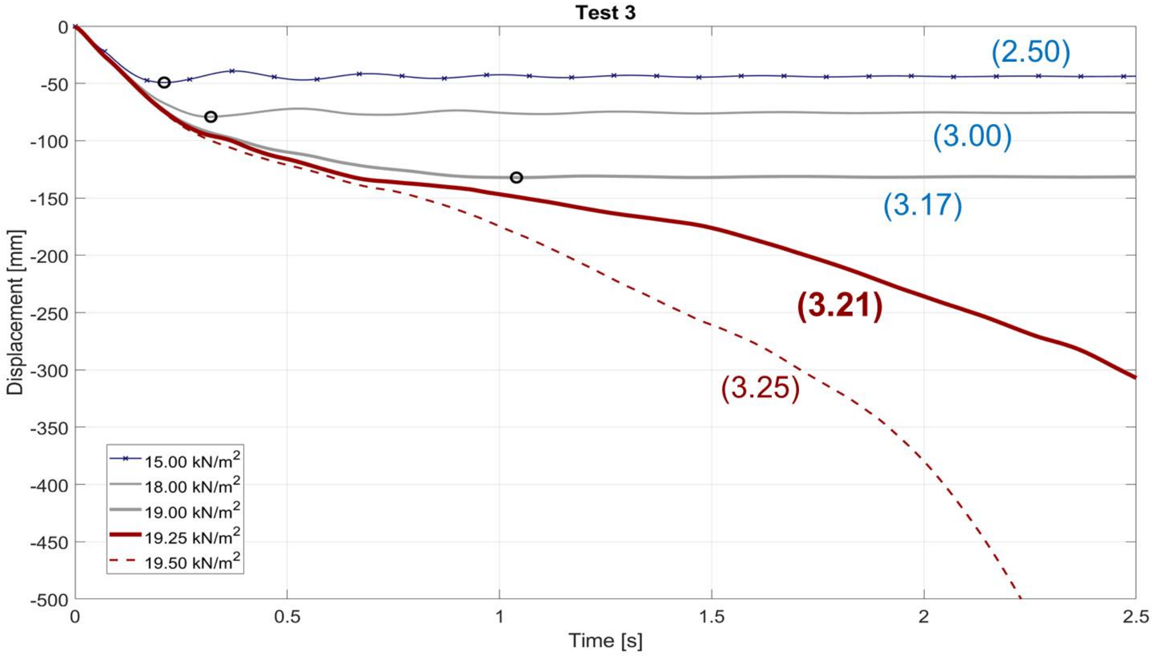

Figure 9. Prediction of vertical displacement of support above removed column for Test 3 under the effect of increasing distributed loads.

Table 2. Estimated collapse load for sudden column removal scenarios corresponding to each test.

	Collapse load [kN]	Multiplier of accidental design load combination
Test 1	27.25	4.54
Test 2	20.25	3.38
Test 3	19.25	3.21

For the corner column removal case, collapse occurred at a load which was more than 4.5 times the load combination to be used for accidental design situations. By analysing contour plots of normal strains at different points in time for the simulation in which collapse occurs, the main resisting mechanisms contributing to the residual capacity of the structure could be evaluated. The time series of the displacements predicted by the simulations are reproduced in Figure 10 together with the corresponding force-displacement curves, whereby the force shown is the maximum axial load in the superior remaining part of the removed column. The green dot indicates the point for which the normal strains are shown in the same figure in the direction indicated by the arrows. These strains are displayed with contour limits set between the peak compressive strain of concrete in blue and the yield strain of steel in red.

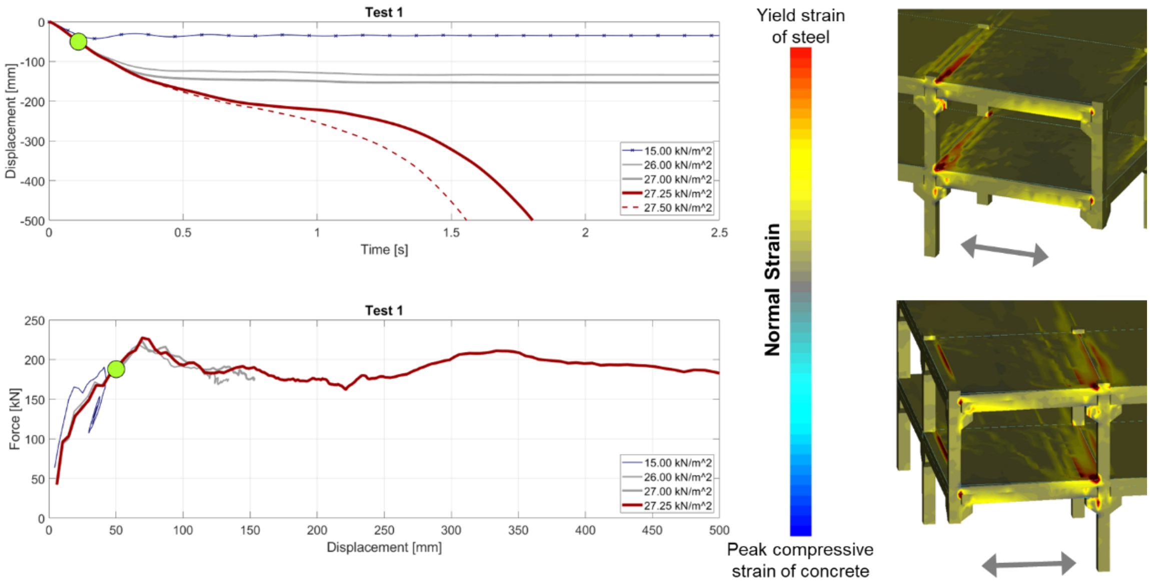

Figure 10. Vierendeel action after column removal for the case of Test 1.

At this stage, it can be observed that both the 5 m and 6 m beams are bending together indicating that resistance is occurring thanks to Vierendeel action. As the displacement increases, some compressive stresses also start to appear in the most constrained part of the slab. This eventually transforms into a form of compressive arching in the shorter 5 m beams as shown in Figure 11. Nevertheless, at this stage it can already be observed that the adjacent corbels have in fact effectively failed and the structure is therefore unable to find a state of equilibrium in which it can resist the load. As such, it can be said that most of the residual capacity of this precast concrete structure after the loss of a corner column can be attributed to Vierendeel action.

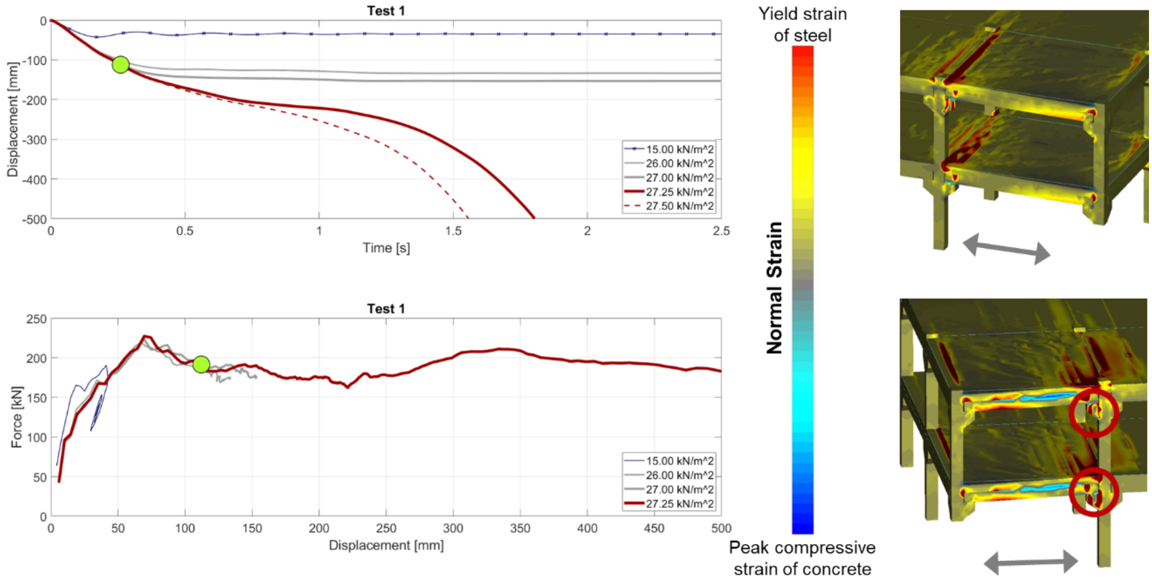

Figure 11. Compressive arching in 5 m beams and failure of corbels after column removal for the case of Test 1.

For Test 2, the simulations predict that it can resist almost 3.4 times the corresponding Eurocode load combination for accidental actions. Once again, it could be observed that the structural system is initially mainly resisting the load through Vierendeel action. As the vertical displacement increases, some noticeable compressive stresses start appearing in the central upper part of the most confined lower 5 m beam. At this stage however, it is worth noting that the adjacent columns have already attained their bending capacity, but some additional plastic deformation is allowed to occur mainly thanks to the ductility of the rebars. As the displacement increases further, the compressive arching action starting in the most confined 5 m beam continues to grow and the slab also collaborates with it to produce a form of compressive membrane action. Finally, some catenary action in the top 6 m beams can also be observed but the columns have already failed in bending and the system is unable to find a state of equilibrium.

For Test 3, the simulations predict that collapse would occur at a slightly lower load than that of Test 2. There are also many similarities with respect to the resisting mechanisms that develop, however there is naturally more asymmetry in the strains after the removal of this penultimate edge column. In this case, the least constrained column clearly fails first once large displacements are allowed to occur.

CONCLUSION

This article presented an experimental and numerical study involving the sudden removal of edge and corner columns from a purposely built two-storey 15×12 m^2 precast reinforced concrete building. The tests performed have shown that simple solutions can enhance the structural robustness of precast concrete structures.

The results presented indicate that simulations using the Applied Element Method can be an effective tool for evaluating progressive collapse resistance. The simulations performed suggest that Vierendeel action is the most significant secondary resisting mechanism contributing to residual capacity after column loss for the scenarios studied.

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