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SIMPLIFIED MODELLING STRATEGY FOR EVALUATING FAILURE PROPAGATION IN REINFORCED CONCRETE STRUCTURES

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ABSTRACT

The progressive collapse of reinforced concrete (RC) structures involves a series of complex load-resisting mechanisms, including flexural, arching and catenary actions, among others. A detailed model comprised of 3D-solid elements with refined mesh and complex material models is typically adopted to accurately simulate these phenomena in new or existing buildings. However, this approach is computationally expensive and unsuitable for practical applications, especially when dealing with large models of real buildings. This study proposes practical modelling strategies that balance the accuracy of an RC structural response to an initial failure with a practical method of preparing and analysing the models. The proposed modelling approaches include a) the PMM–lumped hinge (based on the FEMA-356 guideline), b) a modification to the FEMA hinge, and c) the fibre–distributed hinge. Several progressive collapse tests from the literature were simulated on RC subassemblies with various boundary conditions and loading scenarios from the literature to systematically validate and compare the different approaches. The results revealed that the PMM-lumped approach cannot accurately simulate the structure's response under large deformation mode (catenary) where plastic hinges (yielding of reinforcement) are typically formed at a more extended region along the member. The present study suggests that the fibre-distributed hinge is the most suitable approach for simulating the progressive collapse of structures. This finding is pertinent (yet also alarming) as some new studies in the progressive collapse field still adopt the traditional PMM-hinge approach, which may eventually lead to invalid conclusions.

KEYWORDS: Reinforced concrete; Progressive collapse; Failure propagation; Column removal; Practical modelling strategies; Simulation.

1. INTRODUCTION

The progressive collapse of structures has been the subject of a significant number of research projects [1]. This type of collapse is typically initiated by local damage that propagates to neighbouring elements (chain reaction) and may potentially result in total damages which are disproportionate to their origin. These phenomena are generally caused by extreme loads or events. During such circumstances, the structures may develop complex resisting mechanisms, such as bending, arching actions and catenary effects, among others. A highly detailed model of 3D-solid elements with refined mesh and complex material models is usually adopted to reproduce these phenomena precisely. However, this approach is computationally expensive and thus unsuitable for practical applications, especially when analysing large models of actual buildings. As an alternative, this paper proposes a practical approach using a simplified modelling strategy to identify cases which likely lead to collapse. The modelling strategy adopts the use of 2D-line elements where nonlinearity is assumed to occur either concentrated in specific critical locations (lumped-hinges approach) or distributed along the members (distributed-hinges approach). Several experimental tests on progressive collapse tests of 2D-frame structures from the literature were simulated to validate and compare the applicability of the proposed approach. Based on these simulations, the advantages and limitations of the proposed approach will be discussed, along with some potential improvements.

2. SIMPLIFIED MODELLING STRATEGY

2.1 Methodology

The proposed strategy aims to develop a practical approach that can accurately simulate the response of RC structures subjected to a local initial failure and identify the risk of subsequent collapse propagation. The methodology consists of two primary steps (see Figure 1). First, numerical simulations will be performed using a commercial structural analysis program (SAP2000, CSI [2]) while identifying the key parameters affecting the predictions. In SAP2000, various types of analysis can be performed, including a simple linear elastic to a more refined nonlinear analysis (either static or dynamic). In this study, we adopted nonlinear static analysis for reproducing the quasi-static tests presented in Section 3.1, whereas nonlinear dynamic analysis was adopted for performing the tests with sudden column removals in Section 3.2. In such analyses, both nonlinearities due to materials (cracking, yielding, rupture) and geometry (large deformation) are considered explicitly.

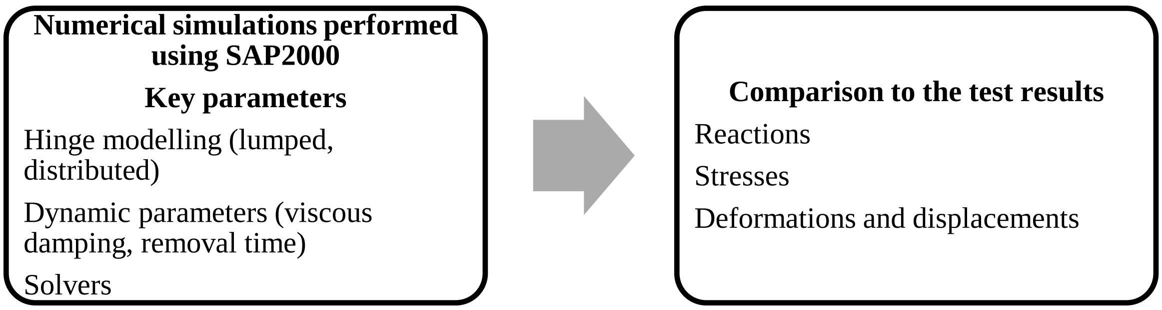

Figure 1. Proposed methodology.

To simulate a realistic behaviour of such reinforced concrete structures, one of the most important aspects is to identify the regions with the highest stress concentrations (cracking, yielding) so that the influence of nonlinearity on the overall structural responses can be captured adequately. In structural engineering terms, such a region is generally known as the plastic hinge region. Two approaches are available to model the plastic hinge region: the lumped-hinge and the distributed-hinge approach. Section 2.2 provides a brief overview of the hinge modelling. About the dynamic analysis, the initial local failures must be reproduced with the rapid removal of these elements. Other modelling assumptions adopted in the present study are described below:

- Beam and column are modelled as 2D-line elements where nonlinearity due to concrete cracking and reinforcement yielding are represented either by the lumped-PMM hinge or distributed-fibre hinge approach. For the lumped approach, plastic hinges are placed at locations with the expected highest bending moments, including at the column face and the mid-span. For the distributed-fibre approach, the relative length of each zone is assumed to be 0.1 of the element's total length, resulting in 10 hinges distributed along the length.
- The steel rebar is not modelled explicitly as a line or beam element; instead, it is modelled as a steel material inside the cross-section definition of the beams and columns. The stress-stress relationship for the steel rebar adopts a multilinear law where the elastic, yielding, strain-hardening, and rupture stages are all considered.

- The analysis explicitly considers nonlinear geometry (large deformation) essential in capturing critical phenomena like arching and catenary actions.
- Shear failure is not considered, which is deemed acceptable as the failure of specimen tested under middle columns loss (as studied here) is mainly governed by the rupture of the flexural reinforcing bars or crushing of concrete in compression but not shear.
- Beam-column joint is assumed to behave as a rigid panel zone.
- In the quasi-static test (Section 3.1), the external load is applied as a downward displacement (incremental) in the middle column with a displacement-controlled procedure. In contrast, for the sudden column removal (Section 3.2), point loads are initially applied to represent the hanged weights in the first loading stage. Then, in the second stage, a dynamic removal of the middle column element is performed, assuming a removal time of about 1/10 of the specimen's fundamental vertical vibration mode [3]. A constant modal damping of 5% is adopted for the dynamic simulations. The integration of the dynamic equations is solved using the Newmark-Betha method with average constant acceleration (middle point rule with $\gamma = 0.5$ and $\beta = 0.25$).

2.2 HINGE MODELLING

This section deals with the different methods of modelling hinges, including a) the PMM–lumped hinge (based on the FEMA-356 provision), b) a modification of the FEMA hinge to account for the presence of catenary action, and c) the fibre–distributed hinge.

2.2.1 PMM-lumped hinge according to FEMA-356

According to the FEMA-356 definition [4], a plastic hinge is described by a load-rotation or load-displacement curve (see Figure 2.a). The a and b parameters refer to the part of the deformation that occurs after the yielding of the reinforcing bars. Parameter c represents the reduced resistance after the sudden reduction of C and D, generally caused by the crushing of the concrete. The a, b and c parameters are defined according to FEMA-356 recommended values. Our preliminary analyses indicated that the parameter provided by FEMA-356 produced premature failures due to the short rotation limit of point E. This is unsurprising as the FEMA-356 model was originally developed for seismic assessment (cyclic loading), where no catenary action was explicitly considered. For this reason, the original FEMA-356 was not used in this work; instead, we adopted the modified version as described in **Section 2.2.2**.

2.2.2 Modified PMM-lumped hinge

According to UFC 4-023-03 [5], the beams can rotate up to 0.20 rad before the catenary actions break down (fracture of the longitudinal reinforcing bars)Haga clic o pulse aquí para escribir texto.. To account for this phenomenon, point E in the original FEMA-356 hinge was extended to 0.2 rad, as illustrated in Figure 2.

2.2.3 Fibre–distributed hinge P-M2-M3

The fibre–distributed hinge (P-M2-M3) is used to represent the axial behaviour of a set of representative axial "fibres" distributed throughout the element's cross-section. Each fibre has a specific position, a tributary area and a curve that describes the stress/deformation ratio. The axial stresses of all the fibres are integrated within the cross-section, which allows the value of P (axial force), M2 (bending moment of axis 2) and M3 (bending moment of axis 3) to be calculated. Also, the axial deformations U1 and rotations R2 and R3 are used to determine the axial deformation of each fibre. The plane sections remain plane throughout the analysis, which provides a valid approach to simulating the structural behaviour in these conditions.

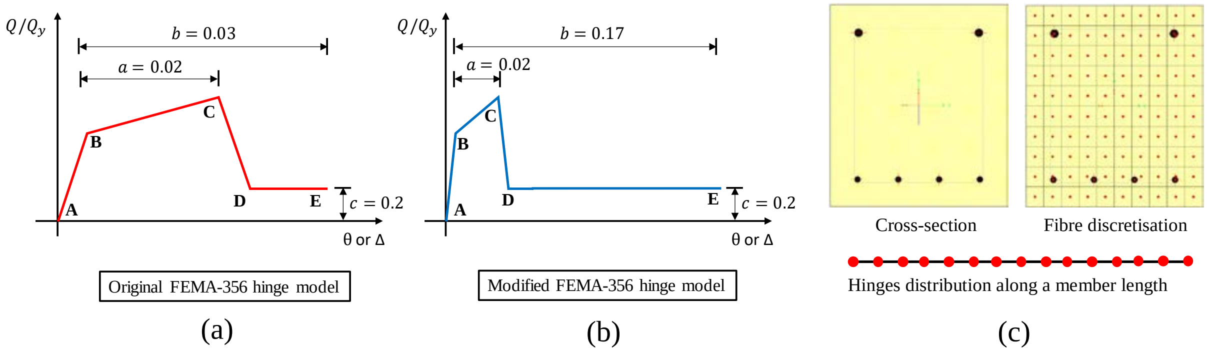

Figure 2. a) Hinge model according to FEMA-356. b) Modified FEMA-356 hinge c) Distributed-fibre approach.

3. VALIDATION AGAINST EXPERIMENTAL TEST

Several progressive collapse tests reported in the literature in the form of 2D-RC frames (subassemblies) were simulated to systematically validate and compare the proposed approach's suitability.

3.1 Test with quasi-static load application

The first case study selected for the validation purpose was carried out by Deng et al. [6]. It consisted of tests on RC beam-column subassemblies subjected to the removal of a central column (see Figure 3) for the SAP2000 model, including the discretisation of the PMM-lumped and distributed-fibre hinge). In the present paper, only subassemblies made of normal-strength concrete were simulated (NSC-8, NSC-11, and NSC-13). The only difference between these three specimens was the slenderness (span-to-depth ratio). The beam dimensions were 250 mm (height) x 150 mm (width). The reinforcement configuration at the support region was 3T12 (top) and 2T12 (bottom), whereas in the mid-span, it was 2T12 (top) and 2T12 (bottom). The shear stirrups were R6 spaced every 100 mm everywhere. The edge column dimensions were 400 x 400 mm^2 (square shape) with 12T16 flexural reinforcing bars and 4-legged stirrups of R6, also spaced every 100 mm. The summary of the testing conditions is described in the following:

- The subassembly's edge columns were restricted laterally and vertically. In particular, the lateral restraints were designed to accommodate both the arching and catenary actions.
- A hydraulic jack was placed over the central column to apply a downward vertical displacement (gradual – quasi statically).
- Load cells were installed to measure the vertical and lateral reactions whereas LVDTs were used to monitor the vertical displacement of the beam.

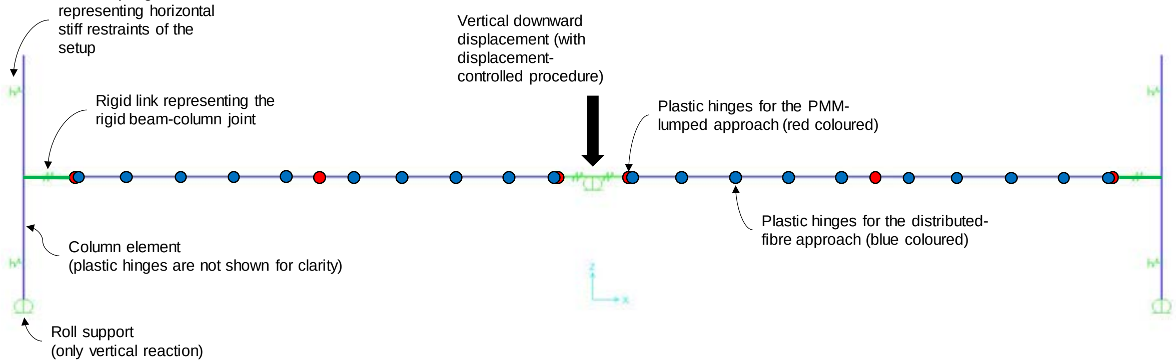

Figure 3. SAP2000 model used in the present study of the test campaign by Deng et al.

Three different modelling approaches were adopted to simulate these three tests: 1) Model 1 adopts the modified FEMA-356 PMM model but only places the hinges at a few locations with the highest bending moments (i.e., at the column face and the midspan of the beam); 2) Model 2 is similar to Model 1, but more hinges were placed along the beam, spaced every 0.1 of the beam length; 3) Model 3 adopts the distributed-fibre hinge approach, also spaced every 0.1 of the beam length. Figure 4 shows the vertical reactions vs displacements predicted using these three models for specimens NSC-8, 11, and 13.

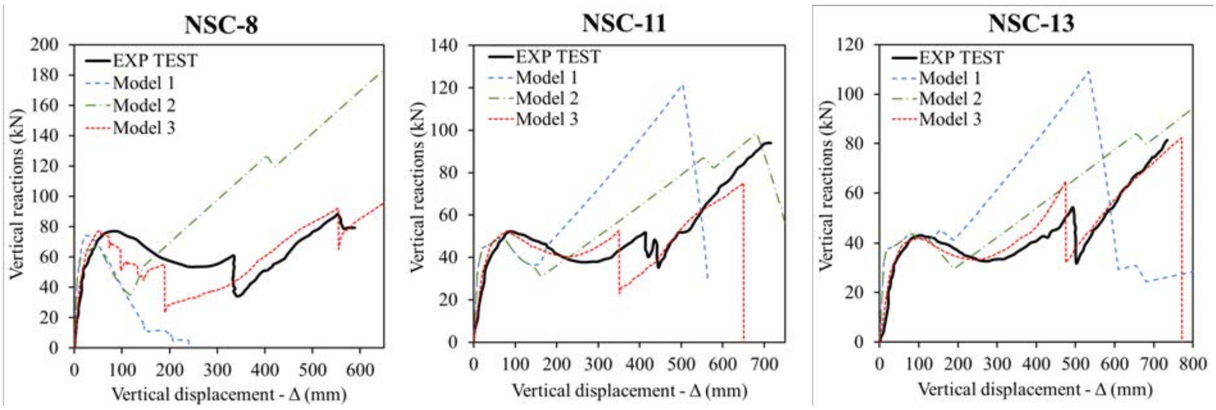

Figure 4. Comparison of the measured vs. predicted responses under quasi-static tests.

In general, it could be seen that the most consistent results were produced by the distributed-fibre hinge approach (Model 3), as it accounts explicitly for the interaction between the axial and moments at the cross-sectional (fibre) level. In contrast, both Models 1 and 2 are less consistent; sometimes, they can underestimate or overestimate the specimens' resistances at different phases. For instance, the PMM-hinge model cannot capture the beneficial effect of the compressive forces on the element resistance as the neutral axis shifting due to flexural cracking is not well-captured (i.e., no contribution of arching action). About the failure mode, the test reports fracture of the bars at the column face for all three specimens. The most consistent mode was also captured by Model 3, where a significant drop in vertical reactions was observed after reaching the peak load. In contrast, Models 1 and 2 did not always produce consistent behaviour as, in some cases, the reaction keeps increasing beyond the observed failure point from the tests (refer to the Model 2 predictions for NSC-8 and NSC-13 in Figure 4).

3.2 Tests with sudden (dynamic) column removal

The second case-study selected for the validation was related to the dynamic column removals by Zhou et al. [7], in which two specimens (subassemblies) were investigated, one with cast-in-place concrete and another with precast concrete elements. The present study only focuses on the cast-in-place concrete specimen. The beam dimensions were 300 mm (height) x 200 mm (width). The reinforcement configuration at the support region was 2T18 (top) and 2T18 (bottom) with stirrups configuration of R6 spaced every 50 mm, whereas, in the mid-span, the flexural reinforcement was the same but the stirrups was R6 spaced every 100 mm. The edge column dimensions were 350 x 350 mm^2 (square shape) with 8T16 flexural reinforcing bars and 3-legged stirrups of R6, spaced every 50 mm (at the plastic hinge region) and 100 mm (elsewhere). The test setups modelled in SAP2000 are shown in Figure 5. The test procedure involved a sudden removal of the central column using a quick-release device (see Figure 5). Six different loading phases were investigated, where the hanging weights gradually increased until the specimens finally collapsed. For this test series, only two numerical models were employed: Model 1 and Model 3 (see the description in Section 3.1). In the present study, only the results of stages 2-5 will be presented.

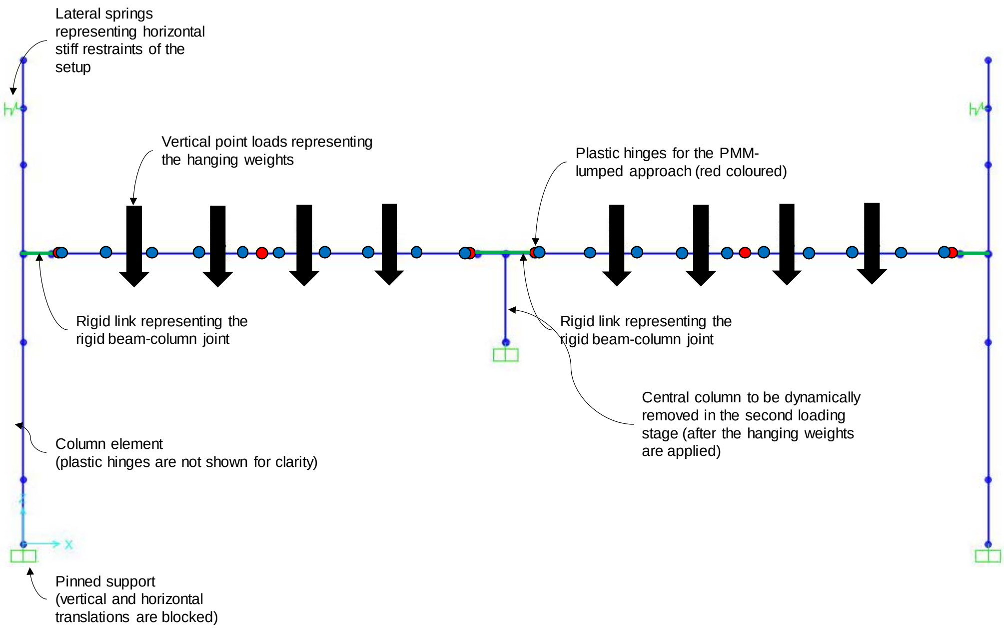

Figure 5. SAP2000 model used in the present study of the test campaign by Zhou et al.

Figure 6 compares the measured and predicted responses in terms of vertical displacement vs time at the location of the removed column. The graphs suggested that, at smaller loads, the PMM-hinge model based on the modified FEMA 356 (**Model 1**) overestimated the system's stiffness, resulting in smaller deflection. However, at larger load magnitudes, **Model 1** produced more accurate predictions. **Model 3,** based on the distributed-fibre hinge approach, provided consistent results for all magnitude of loads.

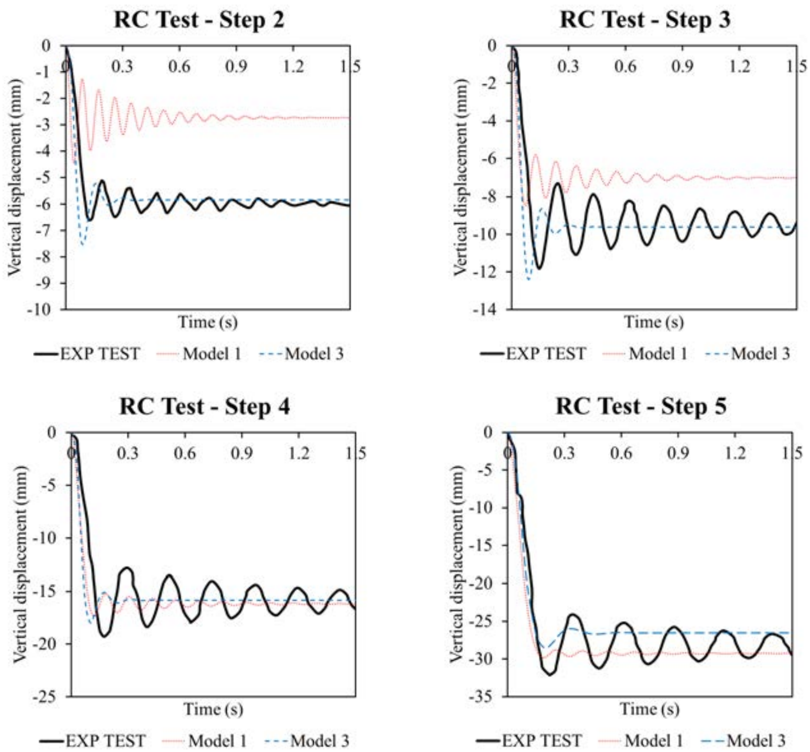

Figure 6. Comparison of the measured vs predicted responses under dynamic removal tests.

4. CONCLUSIONS

This study proposed a simplified modelling strategy for modelling RC structures subjected to column removals, representing building responses under extreme events. The proposed strategy was meant to serve as an alternative tool that is more computationally efficient than 3D-solid models when dealing with a large structural model (a global building behaviour). Two different modelling strategies were evaluated, including the use of lumped-hinge and distributed-hinge (fibre) approaches. For the former approach, the FEMA-356 model was adopted and modified to account for the catenary stage. Selected tests from the literature were used to validate the applicability of the proposed modelling strategy. These include tests with quasi-static and dynamic column removals. The comparison between the measured and predicted responses suggested that the distributed-fibre approach produced more reliable and consistent results when compared to the lumped-hinge approach. This can be explained as, during the catenary stage, damages (cracking and yielding) are expected to form along the member. This phenomenon cannot be adequately captured with the lumped-hinge approach. In future works, we plan to extend the validation of the modelling strategy (based on the distributed-fibre approach) to analyse complete structural systems, including the contribution of the slabs and 3D framing systems.

5. ACKNOWLEDGEMENTS

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