Engineering Structures 136 (2017) 523–534

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Collapse resistance assessment through the implementation of progressive damage in finite element codes

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ARTICLE INFO

Article history:
Received 23 May 2016
Revised 24 January 2017
Accepted 25 January 2017
Available online 3 February 2017

Keywords:
Structural robustness
Progressive damage
Nonlinear FEM
RC framed structure

ABSTRACT

Different procedures for assessing the robustness of a reinforced concrete (RC) frame under progressive damage are proposed and compared. The removal of a column in a RC frame structure is modeled with a commercial nonlinear finite element software according to three alternative strategies: (i) reduction of mechanical properties of the damaged column, (ii) incremental loading of the structure after total removal of the damaged column, and (iii) incremental unloading of internal forces on the damaged column. Nonlinear analysis is performed under a prescribed load combination on three RC frames designed with three Italian building codes in force in different periods. Despite the differences in the strategies for damage modeling, similarities between structural response predictions are highlighted. In addition, it is shown that seismic design provisions for RC structures increase the ductility of the structure but do not necessarily guarantee robustness to progressive collapse scenarios.

1. Introduction

Structural engineers have been concerned with the resistance of building structures to disproportionate collapse since Ronan Point accident in 1968. In 2001, the catastrophic failures of WTC towers in New York showed the true effects of progressive collapse and raised the public interest on the topic. Specific guidelines have been developed and robustness requirements have been inserted in building codes and laws. EN 1990:2002 (Eurocode 0), which is the document at the base of the modern European structural design standards, states that *a localised failure due to accidental actions may be acceptable, provided it will not endanger the stability of the whole structure, and that the overall load-bearing capacity of the structure is maintained and allows necessary emergency measures to be taken* [6].

In such framework, the researches conducted since the second half of the last century have focused the attention on sudden element removals [23,21,47]. Basically, such situations are induced by explosions and impact loads [34]. For ensuring structural robustness in structures, the modern design philosophies switch from being reliability-based to accounting for consequences of local failure. In this sense, Gudmundsson and Izzuddin [20] argued that the scenario of sudden column loss is an effective and straightforward strategy for integrity assessment. Various design guidelines implement such approach through linear/nonlinear static or nonlinear dynamic analysis [5,12]. On one side, static analysis with dynamic increase factor may lead to conservative rather than unsafe design depending on the structural behaviour and configuration [50]. On the other, a large computational effort is required for detailed nonlinear dynamic analyses [24].

Although the above scenarios are the ones that have engendered and still cause many fatalities for building occupants, the attention has recently switched on other sources of degradation and damage [2,31]. For example, Sun and others dealt with the progressive collapse of steel frames due to fire [44,43] modifying a FEM code developed at the University of Sheffield. Others used commercial FEA software ABAQUS for investigating the behaviour of steel structures and connections subjected to fire loads [30]. Fang et al. [14] proposed a simplified energy-based robustness assessment approach in which the maximum temperature is unknown (i.e., this represents a threat-independent local damage scenario); they assess the integrity of the steel structure subjected to fire through a multistage procedure implemented on ADAPTIC code.

The robustness of concrete buildings subjected to element removal has been usually assessed through numerical, experimental and analytical strategies (see, for example, [48,25,42,15]). In addition, theoretical [35,9,7,8,10] and probabilistic approaches

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http://dx.doi.org/10.1016/j.engstruct.2017.01.058
0141-0296/

[40,4,32] as well as scenario analyses have been already formulated and proposed [33].

This paper aims the introduction of various approaches and to indicate an efficient and precise procedure for modeling progressive damage in a reinforced concrete structure. In order to be feasible, one of the requirements of the proposed procedure is the possibility to be straightforwardly implementable on a finite element structural software. Three different strategies for damage modeling are introduced: one accounts for progressive reduction of mechanical properties of the damaged element, the other two consider an incremental analysis scheme on the damaged frame. Computation times and differences between the results of the three strategies are highlighted and commented. The ability and possibility to assess the robustness of a structure in unexpected scenarios are key elements in a consequence-based structural design [11].

2. Methodology

Progressive damage to a column of the first level of a reinforced concrete structure is simulated. The structure is subjected to the following load combination

$$\Omega_N(1.2DL + 0.5LL), \tag{1}$$

where DL represents dead loads and LL denotes live loads. This load combination is suggested in GSA [19]. The term Ω_N is the dynamic amplification factor. For framed reinforced concrete structures, Ω_N is equal to

$$\Omega_N = 1.04 + \frac{0.45}{\theta_{pra}/\theta_y + 0.48}, \tag{2}$$

in those bays immediately adjacent to the removed element and at all floors above the removed element, and equal to $\Omega_N = 1$ in the floor areas away from the removed column. θ_{pra}/θ_y is the ratio between allowable plastic rotation angle and the yield rotation angle.

The effects of progressive or sudden column removal have been analyzed through the implementation of three different numerical strategies, described in detail in the following subsections.

The response of the structure to the progressive damage is evaluated through the maximum vertical drift of all the beams converging in the top node of the damaged element. Fig. 1 shows a two-bay beam-column subassembly with lacking column below N and vertical drift given by

$$v_{\mathrm{N}} = \max\left\{\frac{\Delta_1}{\ell_1}; \frac{\Delta_2}{\ell_2}\right\}. \tag{3}$$

The simulations were performed on an Intel i7, 3.60 GHz, 64-bit computer with 16 GB RAM. A specific MATLAB script controlled a SAP2000 solver and stored the displacement of the monitored nodes in a database file. Geometrical nonlinearity (second order effects) was considered in terms of large displacements (including P-Delta effects). The computation time related to each simulation was recorded through the software.

2.1. Damage model A

Damage model A implemented a progressive reduction of the cross-section area and inertia. According to Lemaître and Chaboche [26], the degradation of the structural element was controlled by a damage parameter d varying from 0 (no damage) to 1 (total damage). If $d = 1$ the element is totally removed. The geometrical properties of the i-th element, i.e., the damaged element, of the RC frame were

$$A_{id} = A_{i0}(1-d) \qquad J_{id} = J_{i0}(1-d)^2, \tag{4}$$

where A_{i0} and J_{i0} are the cross-section area and second moment of inertia. The damage model acted on the size of the concrete cross section, rather than the area of rebar.

Eight columns were alternatively subjected to damage and the overall response of the frame structure was monitored. The simulations (21 in total) were performed at the following values of the damage parameter d:

- from d = 0.000 to d = 0.700, with step of 0.100;
- from d = 0.700 to d = 0.850, with step of 0.050;
- from d = 0.850 to d = 0.950, with step of 0.025;
- from d = 0.950 to d = 0.990, with step of 0.010;
- at d = 0.995 and d = 1.000.

The step size was variable in order to properly account for large displacements as the damage parameter approached unity (i.e., total removal). For each value of the damage parameter, the reduced geometrical properties derived from Eq. (4) were automatically assigned to the damaged element. For the sake of simplicity, a unique undamaged scheme was considered over all simulations. The external load was applied to the damaged structure with its nominal value according to Eq. (1). SAP2000 solver was set as static nonlinear. Meanwhile, vertical displacements of the top node of the damaged column (i.e., the node set at level II with height +3.20 m), as well as adjacent nodes at the same level, were monitored. This allowed the vertical drift of the beams at +3.20 m to be quickly estimated.

Fig. 2 depicts the flowchart of the implemented procedure. The reference structure, i.e., the undamaged one, is solved at the beginning of the procedure. For each damage level j on each damaged element i the reference structure is loaded, the size of the i-th damaged element is modified accordingly and the structure is solved and the response is stored. These operations are replicated for all the damage levels for all the elements. The total number of cycles, i.e., the number of different damaged schemes, is $n \times m$. It is important to note that the results associated to a value of the damage parameter are not dependent on those corresponding to lower values of the damage parameter. In other words, damage history was not considered throughout the simulations.

2.2. Damage model B

Damage model B considered the total removal of the damaged element and the progressive loading of the structure. Before the incremental loading of the structure, no additional loads were acting on the scheme. The loading process was controlled by the vertical displacement of the top node of the damaged element (i.e., the node located at +3.20 m). The loading process can be related to an incremental "pushdown" analysis, see, e.g., [33]. In order to monitor the effective load on the structure at each loading step, the total base vertical reaction was considered as a measure of residual load bearing capacity.

The reference base reaction value in the undamaged scheme (i.e., the vertical base reaction corresponding to Eq. (1)) was determined before the removal of the element and, then, the corresponding values at each loading step were recorded. The flowchart describing the procedure accounting for this damage model is represented in Fig. 3. The reference structure, i.e., the undamaged one, is solved at the beginning of the procedure. For each damaged element, the reference structure is loaded in the software and element i is deleted. Then, a pushdown analysis consisting in the progressive increase of the external loads and the