Structural robustness of reinforced concrete moment resisting frames: deterministic and reliability assessment

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Abstract

This work describes the robustness evaluation of a 2D reinforced concrete moment resisting frame designed in a highly seismic area. In detail, the loss of the central column at the ground-storey level is assumed to trigger the progressive collapse of the frame. At first, a deterministic analysis is elaborated by modelling the frame in the finite element software (FEM) ATENA 2D and performing non-linear pushdown analyses. These analyses consist in applying an increasing vertical displacement in the point of column removal and registering the corresponding reaction. By means of the derived displacement-reaction curves, the capacity of the structure against the removal of the supporting column is studied. Then, a full probabilistic approach is adopted, by sampling 100 different realizations from both material and load properties to evaluate the reliability of the frame. By performing static equivalent non-linear analyses (i.e., amplifying the gravity loads according to properly calibrated dynamic amplification factors) the probability of failure in different cross-sections of the structure is computed. The results demonstrate the lacking safety level associated to the frame under a progressive collapse scenario.

1 Introduction

Dramatic events like the terrorist attack of 2001, which caused the total collapse of the Twin Towers and resulted in thousands of fatalities, represented a shock for the entire community. In recent decades, such catastrophic events impacting strategic structures have prompted architecture, engineering and construction experts to focus increasingly on structural robustness. European codes [1]-[5] have incorporated specific sections addressing structural robustness with guidelines and general requirements. International guidelines [6]-[9] have further explored the concept of extreme actions and their consequences on structures, making it clear that risk analysis should be part of strategies for preventing collapses from low-probability high-consequence (LPHC) events, aiming to find socially acceptable and technically feasible solutions [10]-[13].

Quantitative risk analysis in probabilistic terms can reliably assess the safety level associated with LPHC events by incorporating uncertainties in engineering issues [14]-[17]. For instance, [18] details a sensitivity analysis to compute the bearing capacity of various reinforced concrete (RC) structural members when a central supporting element is removed. Fragility analyses in [19] compute the exceedance probability of different damage states for low-rise RC buildings in a column loss scenario. Global variance-based sensitivity analysis is used in [20] to identify major sources of uncertainty in the response of RC structures to sudden column loss. A reliability-based index of structural collapse for 2D linear elastic truss systems is computed in [21], using random sampling of loads and strengths. The reliability of RC frames under different column-loss scenarios is investigated in [22] identifying side column-loss scenarios as the worst case if infill walls are not considered.

This study evaluates the structural reliability of 2D RC MR frames, focusing on the accidental loss of the central supporting column. Initially, a deterministic approach is adopted to assess the frame's bearing capacity. The frame is modeled in nonlinear finite element (NLFE) software, using mean values of mechanical properties, and a displacement-controlled pushdown analysis is conducted by removing the supporting column and applying an increasing vertical displacement.

Following this, a full probabilistic approach is applied, sampling 100 realizations for various basic random variables related to both material properties and external loads, accounting for their statistical correlations. Preliminary simulations are finalized to compute Dynamic Amplification Factors (DAFs) which are then used in probabilistic equivalent static NLFE analyses to simulate the column removal and to amplify loads on adjacent spans due to dynamic effects. The output of these analyses were strains at sections close to beam-column nodes, distinguishing between confined concrete and reinforcement. Convolution integrals between these strains and ultimate strain distributions are used to determine failure probabilities concerning the ultimate limit state (ULS). Results highlight the need for improved design strategies to enhance the robustness and reliability of structures against such critical scenarios.

2 Case study and finite element modelling

The basis of this work is the design of a 2D multi-story RC MR frame that is regular in elevation and symmetrical. This structure consists of four floors and a roof, each with an inter-story height of 3 meters, and four spans, each having 5 meters in length. The width of the spans in transverse direction is also 5 meters, as shown in Figure 1 (left). Located in L'Aquila, Italy, the structure is designed to meet a high ductility class according to [3]-[4]. The materials specified are B450C steel for the reinforcing bars and C25/30 concrete with a clear concrete cover of 3.5 cm for all structural elements [3]-[4].

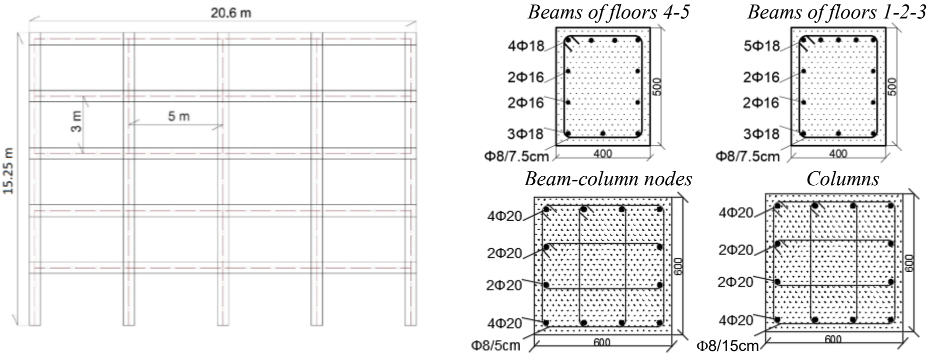

Fig. 1 Characteristics of the frame under investigation: front view (left), reinforcement detailing of the structural elements in the most stressed sections (right).

To define the geometry and reinforcement, both static and modal analyses were conducted, adhering to ultimate limit states (ULSs), serviceability limit states (SLSs), and the capacity design principles for seismic verifications in all structural elements. Figure 2 (right) shows the reinforcement details: ϕ18 longitudinal bars and ϕ8 stirrups for all structural components. All the beams have cross-sections measuring 40x50 cm². The transverse reinforcement in the beams includes 2-leg ϕ8 stirrups, spaced at 10 cm in the dissipative zones and at 15 cm in the central zones of the spans, for all floors, with the dissipative zones extending 100 cm from the beam-column nodes. Columns have cross-sections measuring 60x60 cm². The longitudinal reinforcement consists of 12ϕ20 bars, symmetrically arranged in both directions, and the shear reinforcement is composed of 4-leg ϕ8 stirrups, spaced at 10 cm throughout the column length, except at the beam-column joints where the spacing is reduced to 5 cm.

The FE models of the three frames were developed using the FEM software ATENA 2D [23], employing four-node quadrilateral iso-parametric plane stress finite elements based on linear polynomial interpolation. The element sizes ranged from 0.05 to 0.1 meters, determined through an iterative process for numerical accuracy. The nonlinear system of equations was solved using the standard Newton-Raphson iterative procedure, assuming a linear approximation, with a maximum of 2000 iterations.

For the behavior of concrete in compression, the pre-peak stress behavior is based on the formula from CEB-FIP Model Code 90 [24], while the post-peak response is modeled with a linearly descending branch, incorporating local strain softening. The characteristics of the response in compression in terms of elastic modulus, the peak stress and strain as well as the ultimate strain were assumed including the confinement effects according to the Saatcioglu and Razvi model [25]. The reduction in compressive strength due to cracking is taken as 0.4, consistent with [26]. The compressive behavior of concrete is depicted in Figure 2 (left), using the mean values of the mechanical properties. The tensile behavior of the concrete has been modeled using a local strain tension softening approach. This involves a linear post-peak branch extending to zero strength at a strain equal to $2f_{ct}/E_c$, where f_{ct} is the tensile strength of concrete and E_c is the secant elastic modulus. Additionally, the cracking process has been modeled using the "Smeared cracking" approach with a fixed crack direction model [27].

For the reinforcement steel, a bi-linear constitutive law has been applied for both tension and compression, incorporating a hardening law. The reinforcement was modeled using discrete bar elements, assuming a perfect bond with the surrounding concrete. Figure 2 (right) illustrates the compressive and tensile behavior of steel, based on the mean values of the mechanical properties.

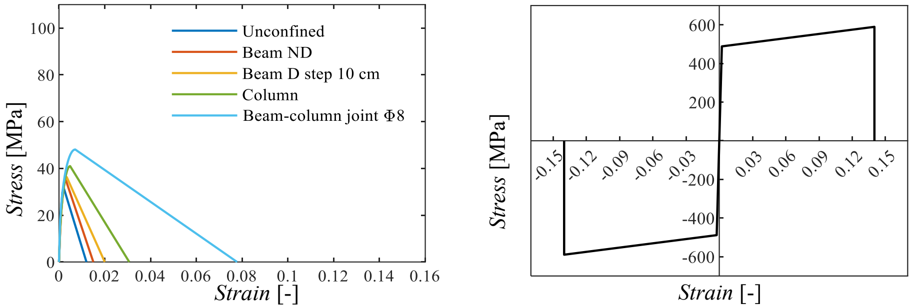

Fig. 2 Constitutive models assuming mean values of the mechanical properties for: concrete in compression (left), reinforcement steel both in compression and in tension (right).

3 Deterministic assessment

A displacement-controlled pushdown analysis has been performed in order to compute the response of the structure against the central supporting column removal. Mean values of the mechanical properties are adopted in this stage. In detail, an increasing vertical displacement is applied where the column is removed and the corresponding reaction given by the structure is registered, allowing to define the capacity curve (i.e., displacement-reaction curve).

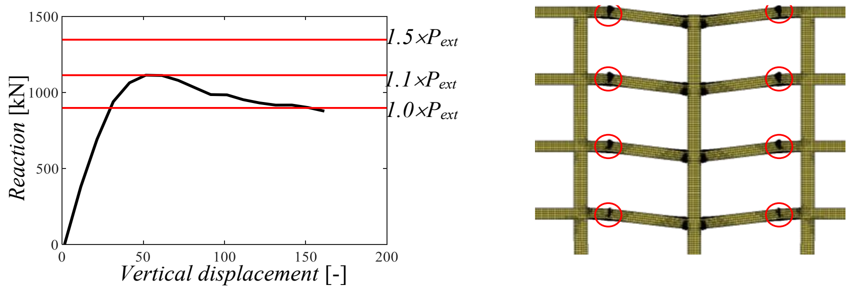

Fig. 3 Results of the pushdown analysis: capacity curve and comparison with the gravity loads (left), deformed shapes and crack formation before failure (right).

Figure 3 (left) shows the results in terms of capacity curve, demonstrating how the structure fails without showing any catenary behavior (i.e., the resistance is not recovered after the softening stage). In addition, by considering the resultant of the gravity loads P_{ext}, obtained considering the accidental combination of the gravitational loads according to [2], it is possible to observe that for a dynamic amplification of the external loads larger than 1.1, the structure is not able to sustain the phenomena of progressive collapse. Furthermore, Figure 3 (right) shows that concentrated curvatures are localized in critical areas of the beams where there is the transition between dissipative and non dissipative zone, where also the longitudinal reinforcement in the upper chord changes. This result suggests that the longitudinal reinforcement should be placed continuously over the beam-column nodes, as widely recognized in many standards [6]-[9], but guaranteeing a continuity over at least 1/3 of the length of the beam, as underlined in [28].

4 Probabilistic assessment

To assess the structural robustness of the frame, a full probabilistic approach is adopted by following the next steps:

- sampling the random properties of both materials and loads;
- conducting displacement-controlled pushdown NLFE simulations on the structure in order to define the probabilistic capacity curves (i.e., displacement-reaction curves);
- using the energy equivalence approach, as proposed by Izzuddin [29], compute the dynamic displacement and the corresponding dynamic gravitational load P_d that causes that displacement;
- evaluating the dynamic amplification factors (DAFs) as the ratios between the dynamic and static gravitational loads P_{ext};
- performing equivalent static non-linear analysis to simulate the removal of the base column and amplify the loads of the adjacent spans on each floor using the energy-based DAFs, while maintaining non-amplified gravity loads on the other spans
- analyzing the aleatory results in terms of strains for different materials to assess the structural reliability at the ULS.

4.1.1 Sampling of the basic random variables

A full-probabilistic approach was considered by sampling both materials and loads as basic random variables, using the Latin Hypercube Sampling (LHS) technique [30] and including the following statistical characteristics:

- permanent structural load: G_1 ~ Normal (16 kN/m ; 0.05);
- permanent non-structural load G_2 ~ Normal (13 kN/m ; 0.05);
- floor variable loads: Q_f ~ Gumbel (7.3 kN/m ; 0.20);
- roofing variable loads: Q_r ~ Gumbel (1.8 kN/m ; 0.20);
- and snow load: Q_s ~ Gumbel (4.7 kN/m ; 0.20);
- concrete compressive strength: f_c ~ Lognormal (31.9 MPa ; 0.15);
- reinforced concrete specific weight: ρ ~ Normal (25 KN/m^3 ; 0.05);
- reinforcing steel elastic modulus: E_s ~ Lognormal (210000 MPa ; 0.03);
- reinforcing steel yielding strength: f_y ~ Lognormal (488.6 MPa ; 0.05);
- reinforcing steel ultimate strength: f_u ~ Lognormal (589.8 MPa ; 0.05);
- reinforcing steel ultimate strain: ε_{su} ~ Lognormal (0.14 ; 0.09);

where the symbol ~ indicates "distributed as", the first number in the parenthesis indicates the mean value and the second one denotes the coefficient of variation of the basic random variable. In detail, 100 realizations were sampled for each basic variable. In addition, the following correlation coefficients between reinforcement parameters are considered:

- 0.85 between f_y and f_u;
- -0.5 between f_y and ε_{su} ;
- -0.55 between f_u and ε_{su}.

4.2 Probabilistic pushdown analysis and DAFs

This section focuses on computing the DAFs necessary for probabilistic equivalent static NLFE analyses to address the dynamic nature of a column removal scenario. The DAFs are calculated using the energy equivalence method by [29]. According to this method, the maximum dynamic displacement occurs when the external work (work done by gravity loads) equals the internal energy (energy absorbed by the structure). Subsequently, the DAF is computed as the ratio between the dynamic load P_d and static load P_{ext}. Specifically, P_d is evaluated at the maximum dynamic displacement, whereas P_{ext} represents the static gravity load concentrated at the top of the removed column, evaluated according to the accidental combination of gravity loads (i.e., permanent and variable loads), as defined in the previous section adopting the mean values.

To evaluate the dynamic response P_d, static displacement-controlled pushdown NLFEAs were performed following the previously described procedure. Specifically, one set composed of 100 random NLFE models was defined, varying material properties while keeping geometry and constraints consistent. This process generated aleatory force-displacement curves (Figure 4), referred to as aleatory pushdown or capacity curves. The results in Figure 4 confirm that the structure does not show any catenary behavior even for the most ductile mechanisms. In addition, there is a large variability of the ultimate response of the structure due to both the uncertainty in the concrete and steel mechanical characteristics.

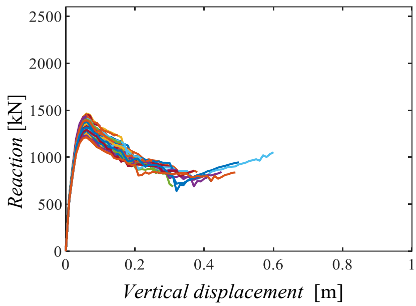

Fig. 4 Probabilistic capacity curves.

By applying the energetic equivalence approach, the DAFs were determined for only about 25% of the cases, as only those combinations of load and material properties allowed achieving a performance point (i.e., equality between internal energy and external work). The mean value of the DAFs where a performance point was reached is equal to 1.16. The number of collapse cases are taken into account in the computation of the failure probability in the following section.

In Table 1, the statistical properties of the dynamic load and external load are shown.

Table 1 Maximum, minimum and mean values of the external and dynamic load.

P_o [kN]			P_d [kN]		
Max	Min	Mean	Max	Min	Mean
1228	1009	1127	1433*	1168*	1267*

*Evaluated only for the cases where the performance point was obtained between the internal and external energies.

4.3 Probabilistic equivalent static analysis and probability of failure

This section focuses on the equivalent static NLFE analyses to simulate the progressive collapse and compute the corresponding failure probability. For the cases where the energetic equilibrium was obtained, the simulations consisted in applying gravity loads, removing the central supporting column, and amplifying the gravity loads on the spans of the five floors adjacent to the central column based on the previously computed DAFs. The load amplification was limited to these spans because the damping properties of the structural RC elements prevented other spans from being affected. In cases without DAFs, simulations were run until collapse occurred, as predicted by the energetic approach, by slightly amplifying the loads in the spans adjacent to the removed column.

Next, the principal total strains at significant points of the RC frames were extracted for each set of simulations. Specifically, in the cross-sections of both beams and columns near each beam-column joint, various points were considered, distinguishing between points in the confined concrete core and those related to reinforcement, including ordinary reinforcement and stirrups.

The failure probability was then determined for each material in each cross-section, calculated as the probability of the demand exceeding the capacity concerning the ULS. Specifically, the failure probability was computed by performing a convolution integral between the Probability Density Function (PDF) of the demand, derived from the 100 aleatory principal total strains, and the Cumulative Density Function (CDF) of the capacity, obtained from the sampled ultimate strain thresholds. All ultimate strains were associated with lognormal distributions according to statistical inference analysis with a 5% significance level. The failure probability in the central spans also accounted for the collapse cases applying the total probability theorem.

In figure 5 the sections where failure may occur and the probability of their failure are shown. The largest failure probabilities were generally observed at the concrete level. In detail, the maximum failure probability is equal to 0.82 and it is reached in the beams of the first floor adjacent to the removed column, but also the beams of the upper floors are subjected to values much larger than 10^{-1}. In addition, the lateral spans are characterised by failure probabilities between 10^{-2} and 10^{-1}, indicating a damage spread from the area directly affected by the collapse and the one indirectly affected. As already anticipated within the deterministic analysis, this result indicates that the structure designed according to actual code rules is not able to sustain the accidental removal of the supporting column. In fact, the largest failure probability corresponds to a safety level in terms of reliability coefficient much lower than 1.

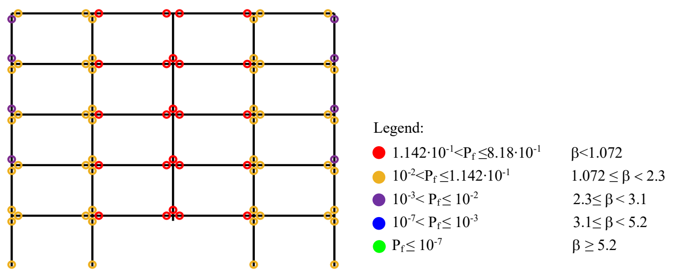

Fig. 5 Maximum probability of failure between concrete and steel reinforcement.

5 Conclusions

This study evaluates the structural reliability of planar seismically designed frames, particularly focusing on the accidental loss of the central supporting column. At first, a deterministic approach is adopted to evaluate the bearing capacity of the frame. In detail, the frame is modelled in a nonlinear finite element software, considering mean values of mechanical properties and carrying out a displacement-controlled pushdown analysis, by removing the supporting column and applying an increasing vertical displacement. Results in terms of displacement-reaction demonstrated that, for a dynamic amplification of the external gravity loads larger than 1.10, the structure is not able to sustain the accidental removal of the supporting column. Then, a full probabilistic approach is applied, sampling 100 realizations for various basic random variables related to both material properties and external loads, accounting for their statistical correlations. Preliminary analyses are carried out to compute the dynamic amplification factors (DAFs) for all the realizations, demonstrating that the structure failed to find an energetic equivalence at the performance point for the majority of cases. These DAFs were used to conduct probabilistic equivalent static nonlinear finite element analyses to simulate the column removal and amplifying loads on adjacent spans due to the dynamic effects. Subsequently, these analyses evaluated strains at beam-column joints, distinguishing between confined concrete and reinforcement. Convolution integrals between these strains and ultimate strain distributions were used to determine failure probabilities with respect to the ULS. The maximum failure probability, equal to 0.82, was reached in the beams of the first floor adjacent to the removed column. Additionally, the beams of the upper floors exhibited failure probabilities significantly larger than 0.1. The lateral spans also showed failure probabilities ranging between 0.01 and 0.1, indicating that damage spread from the area directly affected by the collapse to areas indirectly affected. This result confirms again that the structure designed according to current code rules is not capable of sustaining the accidental removal of the supporting column. In fact, safety levels in terms of the reliability coefficient are much lower than 1. This underscores the need for improved design strategies to enhance the robustness and reliability of structures against such critical scenarios.

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