ENHANCING THE PROGRESSIVE COLLAPSE RESISTANCE OF SEISMICALLY DESIGNED STEEL-CONCRETE COMPOSITE BUILDINGS

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1. ABSTRACT

The present work is concerned with designing steel-concrete composite buildings under requirements on: (a) safety of structural members based on provisions of Eurocodes 3 and 4, (b) structural system resistance against seismic actions without the development of extensive lateral deflection and inter-storey drifts and (c) progressive collapse resistance, which ensures that local failure in structural elements due to accidental actions does not trigger disproportionate collapse of the structure. The first two are conventional design requirements typically taken into account, while the third is an additional design requirement, therefore an increase to the total structural cost is inevitable. Thus, an engineer is required to put significant effort in the limitation of the extra cost induced, ensuring at the same time that the desirable structural performance is achieved.

Automatic optimization algorithms are valuable tools for this purpose. However, the same goal might be pursued using manual strategies and exploiting one's experience. Four manual beam upgrade strategies are proposed, in order to improve the progressive collapse resistance of steel-concrete composite buildings already designed against earthquake. The effectiveness of the aforementioned strategies is assessed in comparison with the design defined using an optimization algorithm.

2. INTRODUCTION

Progressive collapse refers to a structural failure in a form of a chain reaction, as the consequence of damage to a relatively small part (usually a column) of the structure. There is an increase of the internal forces at the remaining intact elements, especially those

adjacent to the column, which has been removed or destroyed. If the extra loads cannot be properly redistributed to the undamaged elements, then the damage extends and a broader or global failure of the structure occurs. It is an unacceptable failure type of a structure, not only because of the disproportionate propagation of the damage, comparing to its cause, but also because it can take place almost instantly after the failure of the first structural element. Thus, it is necessary to ensure the ability of the structure to receive accidental or unforeseen actions, usually as a result of local failures, without extensive damage.

In the previous decades, structural engineers have given emphasis on designing buildings, in order to resist intense seismic excitations. However, various destructive events in the past have revealed that loss of structural stability might occur even due to small scale damage. Even though the potential of progressive collapse has been identified since the collapse of the Ronan Point Building in 1965 [1] as an issue to deal with in the structural design phase, research on the particular topic intensified mainly during the past decade.

3. DESIGNING AGAINST EARTHQUAKE AND PROGRESSIVE COLLAPSE

The present work is concerned with designing structures able to resist earthquake actions and progressive collapse. Hence, conventional design requirements on safety of structural members and v are considered. Additionally, requirements on progressive collapse resistance are imposed to study their effect on structural designs. The design of structural members of an undamaged building under gravitational loads is performed using the internal forces obtained by means of an elastic static analysis, which are compared against member capacities calculated with analytical formulas provided in design codes. The assumption of elastic behaviour does not apply when designing against earthquake or progressive collapse: in these cases, inelastic deformations are allowed to occur, so the design procedure is based on the nonlinear performance of the structural model.

We consider the design of multi-storey composite buildings, which have steel-concrete columns consisting of steel members with standard I-shaped sections (HEB) fully encased in concrete; the buildings have steel beams with standard I-shaped sections (IPE) and (optional) steel bracings with standard L-shaped sections. Such composite structures are required to satisfy provisions of Eurocode 4 [2] for the composite steel-concrete columns and Eurocode 3 [3] for the pure steel members (beams and bracings). Overall seismic resistance is controlled through lateral deflection constraints evaluated using pushover analyses up to a targeted top displacement [4,5]. Moreover, interstorey drifts are constrained to achieve adequate structural performance with respect to the collapse prevention limit state [4].

Three structural analyses are conducted for each design, in order to evaluate its adequacy with respect to the aforementioned design requirements: (a) a force-controlled linear static analysis under gravitational loads, in order to perform capacity checks according to Eurocodes 3 and 4 and (b) two displacement-controlled nonlinear static pushover analyses (one for each horizontal direction), in order to assess the response of the intact structure under seismic action. All analyses are performed using the structural analysis software OpenSees [7]. Fiber section elements are utilized to represent all structural members.

Requirements for progressive collapse resistance are handled by applying the so-called alternate load path method [6], according to which a structural element, usually a column, is assumed to have experienced a destructive event and has failed. The structure is then modelled without the failed element, in order to determine its ability to redistribute the acting loads to the remaining intact members and remain stable despite the notional element loss. In order to assess the progressive collapse resistance of a building, an

equivalent to the nonlinear pushover analysis in the vertical direction is applied. Hence, the 'Nonlinear Static Analysis under Gravitational Loads', often referred to as 'Nonlinear Pushdown Analysis' in the literature, is usually performed under specific damage scenarios. Following the removal of certain element(s) from the simulated model, the structure is required to sustain the gravitational loads, which are applied to it progressively. Thus, a fourth analysis is required for each design: a force-controlled nonlinear static pushdown analysis of the 'damaged' structure under gravitational loads only. The quantitative progressive collapse resistance requirement applied in this work controls the maximum vertical drift of the steel beams above the 'damaged' area of the building. More specifically, the plastic rotation at the free end of such a beam is required to be less than 6° [6], which corresponds to a maximum allowable relative vertical displacement normalized by the beam length (vertical drift) between the beam's ends of about 10%.

4. ENHANCING PROGRESSIVE COLLAPSE RESISTANCE

When designing a steel or steel-concrete composite structure against progressive collapse, the engineer needs to focus primarily on strengthening beams and bracings, as they can horizontally transfer the loads from the damaged area to the undamaged part of the building. The determination of an effective design can take place through a trial-and-error process, according to which the engineer attempts to improve the building's performance under pre-specified damage scenarios.

Four beam upgrade strategies (Fig. 1) are proposed and assessed regarding their effectiveness in increasing the progressive collapse resistance of buildings previously designed against earthquake. The first two strategies focus on the 'local' enhancement of a particular beam group. More specifically, the first strategy, commonly regarded as an efficient technique against progressive collapse, is the creation of a strong 'bridge' over the bays affected by column(s) loss and the transfer of loads to neighbouring undamaged columns. This bridge is formed by increasing the beam sections only at very few storeys (typically at just one storey) above the location of the damage.
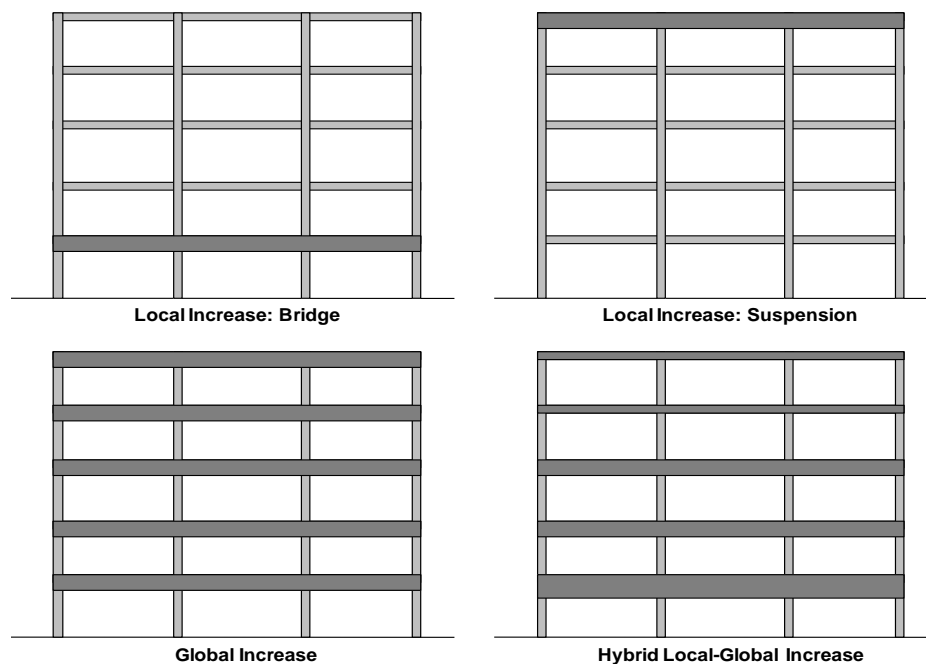

Figure 1: Illustrative representation of the proposed beam upgrade strategies

Accordingly, an alternative 'local' strategy is the 'suspension' of the damaged bays from a bridge formed at the top storey(s). A decisive difference of this strategy to the previous one is that the strengthened beam group does not depend on the location of damage, as the upgrade is always at the top of the structure. However, stronger columns are expected to be required over the whole building height, in order to safely transfer loads from the top bridge to the ground.

In a different context, the global upgrade of the structure's beams aims to invoke the whole structural system in the transfer of loads. This strategy is applied in the present work by uniformly increasing the beam sections at all storeys, e.g. by adopting the immediately larger section for each beam group of the structure. Finally, a 'hybrid local-global' strategy is investigated. In this strategy, the sizes of beam sections along the building's height are assigned in a 'pyramid-like' manner. Hence, the lowest beam group has the strongest sections; any higher group has a section of at least one size smaller than the group directly below. In particular, the first beam group upgraded is the lowest one, just like in the 'bridge' method. In the next upgrade step, the section of the lowest beam group is increased again, while the section of the beam group just above is increased by one size as well. This concept is repeated in the next steps, involving the upgrade of one extra (higher) group at each next step, until all beam groups have been modified at least once. If a feasible design has not been determined yet by this upgrade procedure, then a 'global increase' of the sections of all beam groups by one size takes place in all next steps, until the beam section database has been fully utilized. Once the sections of all beam groups have been modified at least once, the 'pyramid-like' upgrade applies the same philosophy as the global increase: the whole system is employed, in order to transfer the loads to undamaged regions of the building. However, the hybrid upgrade strategy gives emphasis on the lower beam groups, while not disregarding the upper ones, therefore this strategy may identify more cost-effective results than the 'global increase', especially for buildings with many different beam groups.

It should be noted that, in all aforementioned methods, when an increase in column sections is required, as a direct consequence of the increase in the beams, it is implemented accordingly. Since the change in the beam sections affects the overall structural response, all designs need to be assessed both for their performance under horizontal seismic action as well as for their progressive collapse resistance.

5. APPLICATION

In this section, the four beam upgrade strategies described in the previous section are implemented, in order to enhance the performance of an earthquake-resistant six-story 5x5-bay steel-concrete composite building under a specific damage scenario. In a previous investigation [8], it was noticed that earthquake-resistant designs for composite buildings satisfying the provisions of the respective parts of Eurocodes 3 and 4 for gravitational loads possess sufficient capacity to meet the progressive collapse resistance requirements under single-column removal scenarios. However, the same does not apply for more extensive damage scenarios involving multiple failed elements [9]. In the present investigation, a three-dimensional damage scenario is considered. According to this scenario, a hypothetical explosion has taken place at the building's base, near to one of its corners, significantly damaging three columns and two beams at the first story, as well as the corner column at the second story (Fig. 2). The 'damaged' elements are removed from the structural model, since they are assumed to have failed.
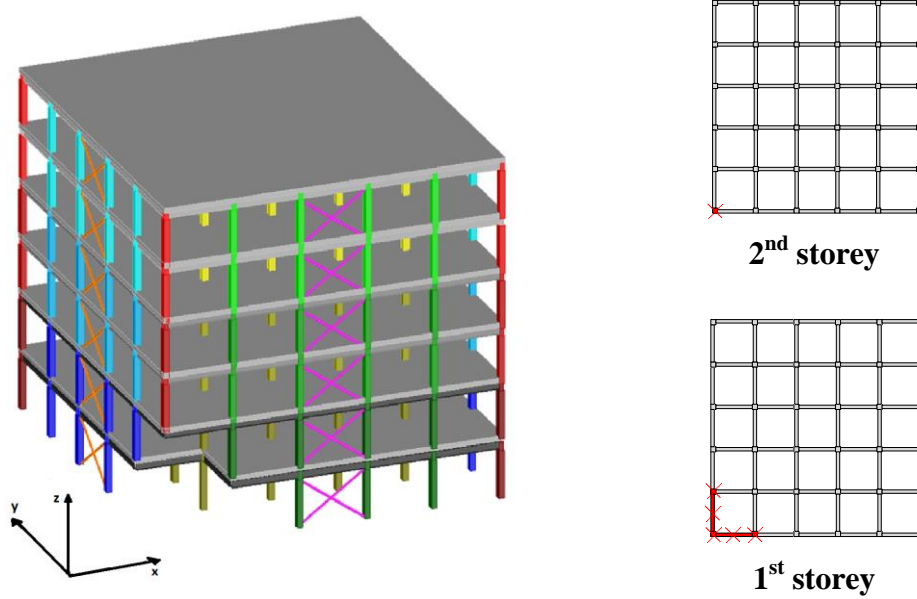
Figure 2: Investigated three-dimensional damage scenario

The investigated building includes bracings installed at the middle bays of each external side. The location of the bracings is selected in order to avoid being above the bays affected by the removed structural members. The inter-story height of the building is 3.5m and the beam length 6m in both horizontal directions.

The effectiveness of the proposed beam upgrade methods is evaluated against the design defined by an optimization procedure. The configuration of the optimization problem is as described in [10]. A total of 17 member groups, which are illustrated with different colours in Fig. 2, are considered for this building; one design variable is assigned to each member group. In particular, columns are organized every 2 storeys into 4 groups: (1) corner, (2) peripheral in *x*-direction, (3) peripheral in *y*-direction and (4) internal. Moreover, every 2 storeys, all beams belong to one group. Finally, one group of bracings is specified for each horizontal direction. Hence, 12 column-groups, 3 beam-groups and 2 bracing-groups are defined in total.

Table 1 presents the optimal member sections attained for each element group by the optimization procedure without and with design requirements against progressive collapse [10].

	Opt(E)			Opt(E&PC)		
	Storeys 1-2	**Storeys 3-4**	**Storeys 5-6**	**Storeys 1-2**	**Storeys 3-4**	**Storeys 5-6**
Corner columns	HE 220 B	HE 200 B	HE 180 B	HE 280 B	HE 200 B	HE 200 B
External columns, x-direction	HE 300 B	HE 300 B	HE 220 B	HE 320 B	HE 220 B	HE 200 B
External columns, y-direction	HE 450 B	HE 340 B	HE 180 B	HE 450 B	HE 450 B	HE 200 B
Internal columns	HE 340 B	HE 220 B	HE 180 B	HE 220 B	HE 200 B	HE 200 B
Beams	IPE 330	IPE 360	IPE 330	IPE 550	IPE 360	IPE 450
Bracings on x-direction	L 90×90×7			L 90×90×7		
Bracings on y-direction	L 90×90×7			L 90×90×7		
Equivalent steel mass (tn)	186.1			238.9		
Max vertical drift	14.6%			9.9%		

Table 1: Optimum designs for the composite building; Opt(E) refers to optimization under the Eurocodes 3 and 4 and the Earthquake-related constraints; in Opt(E&PC), the constraint on Progressive Collapse resistance is additionally included in the optimization procedure

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According to the results of Table 1, when progressive collapse resistance is not explicitly treated, the optimal seismically designed building fails to satisfy the maximum vertical drift requirement of 10% in the case of 'damage'. This requirement is satisfied when a respective constraint is incorporated in the optimization procedure; however, this is achieved with a substantial increase in the amounts of materials in the structure.

Starting from the optimally designed building Opt(E) of Table 1, the 4 beam upgrade strategies are applied, in order to enhance its progressive collapse resistance, which is not acceptable (max vertical drift 14.6%>10%). The sections of the beam groups corresponding to each strategy are increased one I-shaped size at a time, until the beam section database has been fully utilized. The results obtained for all upgraded designs produced are illustrated in Fig. 3, which presents a Pareto-type curve revealing the trade-off between the total structural cost (calculated as total equivalent steel mass of the composite building [10]) and the maximum recorded vertical drift of each design.
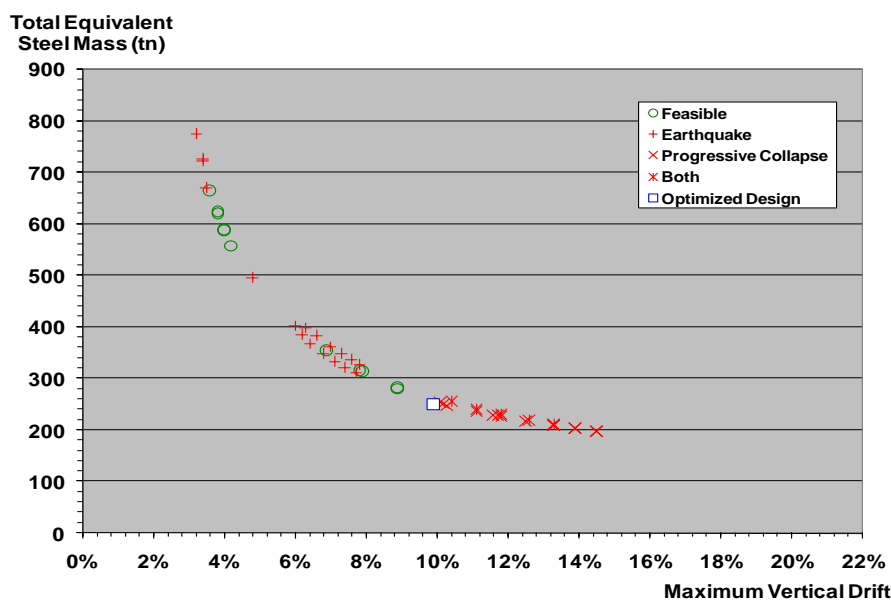

Figure 3: Total mass vs. max recorded vertical drift for the designs determined by the beam upgrade strategies (green and red points) and the optimization procedure (blue point); blue and green points correspond to designs satisfying all requirements; red points correspond to designs that violate the requirement on earthquake resistance or the requirement on progressive collapse resistance or both requirements

6. DISCUSSION AND CONCLUSIONS

In general, increasing the beam sections was found to improve the progressive collapse performance of the analyzed building. Both 'local increase' strategies are able to reduce the maximum recorded vertical drift; however, as the beam sections increase significantly, undesirable effects appear on the building's seismic performance. Therefore, when a small improvement on progressive collapse resistance is sought, the particular methods seem able to provide cost-effective solutions. For earthquake-resistant designs exhibiting large plastic rotations at beams over damaged areas, the effectiveness of 'local increase' strategies cannot be guaranteed.

The 'global increase', as well as the 'hybrid local-global increase' strategies appear to be more robust than the 'local' ones. Although the 'global' and 'hybrid' strategies result in a significant increase to the total cost for relatively small beam upgrade requirements, the

majority of the designs determined by the these strategies are feasible solutions with significantly reduced maximum recorded vertical drift. Hence, the 'global' and 'hybrid' strategies are more preferable for high requirements on progressive collapse resistance.

Of particular interest is the location of the optimized design with respect to the Pareto-type curve defined by the rest of the generated designs in Fig. 3. The point corresponding to the optimized design is lower than all other feasible ones, as it has the lowest total equivalent steel mass. Additionally, it is very close to the maximum admissible vertical drift limit without violating it though. The automatically identified optimized design has approximately 15% reduced total cost compared to the most cost-effective feasible design defined by the proposed manual beam upgrade strategies; however, the latter achieves a reduction of the maximum recorded vertical drift down to 9%, while no other design lies between those two. Hence, one can assume that, by using a richer beam section database and/or by organizing the beam sections in more groups, even more manually identified feasible designs could be generated closer to the optimized one. Consequently, the investigated beam upgrade strategies can provide cost-effective designs; however, it does not seem possible to know how far such manually obtained designs are from optimized solutions, without having performed actual design optimizations.

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ΤΡΟΠΟΙ ΒΕΛΤΙΩΣΗΣ ΤΗΣ ΙΚΑΝΟΤΗΤΑΣ ΑΝΤΙΣΤΑΣΗΣ ΣΕ ΠΡΟΟΔΕΥΤΙΚΗ ΚΑΤΑΡΡΕΥΣΗ ΓΙΑ ΣΥΜΜΙΚΤΑ ΚΤΙΡΙΑ ΣΧΕΔΙΑΣΜΕΝΑ ΕΝΑΝΤΙ ΣΕΙΣΜΟΥ

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ΠΕΡΙΛΗΨΗ

Αντικείμενο της παρούσας εργασίας είναι ο σχεδιασμός σύμμικτων κτιρίων με απαιτήσεις: (α) επαρκούς αντοχής δομικών στοιχείων βάσει ελέγχων των Ευρωκωδίκων 3 και 4, (β) επαρκούς ικανότητας αντίστασης σε οριζόντιες δράσεις λόγω σεισμού χωρίς την εμφάνιση εκτεταμένων μετατοπίσεων και (γ) επαρκούς ικανότητας αντίστασης σε προοδευτική κατάρρευση, που μπορεί να προκληθεί από την τοπική αστοχία σε δομικό στοιχείο λόγω τυχηματικής δράσης και τη δυσανάλογη διάδοση της βλάβης στον υπόλοιπο φορέα. Καθώς η τρίτη προϋπόθεση αποτελεί μια επιπλέον απαίτηση σχεδιασμού, πέραν των πρώτων δύο που συνήθως λαμβάνονται υπόψη στις μελέτες, η αύξηση του συνολικού κόστους είναι αναπόφευκτη. Ως εκ τούτου, ο μηχανικός καλείται να καταβάλει κάθε δυνατή προσπάθεια ώστε να περιορίσει την εν λόγω αύξηση, εξασφαλίζοντας ταυτόχρονα και την επιθυμητή απόκριση της κατασκευής.
Οι αυτόματοι αλγόριθμοι βελτιστοποίησης αποτελούν πολύτιμα εργαλεία στην προσπάθεια αυτή. Ωστόσο, ο στόχος αυτός μπορεί να είναι επιτεύξιμος και χωρίς βελτιστοποίηση, με την εφαρμογή μη αυτόματων στρατηγικών σχεδιασμού. Τέσσερις μη αυτόματες στρατηγικές ενίσχυσης δοκών προτείνονται για την αύξηση της αντίστασης σε προοδευτική κατάρρευση σε σύμμικτα κτίρια σχεδιασμένα έναντι σεισμού. Η αποδοτικότητα των εν λόγω στρατηγικών αξιολογείται σε σύγκριση με τα αποτελέσματα που προκύπτουν από τη χρήση βελτιστοποίησης.