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Effect of retrofit strategies on mitigating progressive collapse of steel frame structures

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ABSTRACT

In this study, the effect of three retrofit strategies on enhancing the response of existing steel moment resisting frames designed for gravity loads is investigated using Alternate Path Methods (APM) recommended in the General Services Administration (GSA) and the Department of Defense (DoD) guidelines for resisting progressive collapse. The response is evaluated using 3-D nonlinear dynamic analysis. The studied models represent 6-bay by 3-bay 18-storey steel frames that are damaged by being subjected to six scenarios of sudden removal of one column in the ground floor. Four buildings with bay spans of 5.0 m, 6.0 m, 7.5 m, and 9.0 m were studied. The response of the damaged frames is evaluated when retrofitted using three approaches, namely, increasing the strength of the beams, increasing the stiffness of the beams, and increasing both strength and stiffness of the beams.

The objective of this paper is to assess effectiveness of the studied retrofit strategies by evaluating the enhancement in three performance indicators which are chord rotation, tie forces, and displacement ductility demand for the beams of the studied building after being retrofitted.

1. Introduction

In the past decades, there have been cases where buildings around the world have experienced partial or total progressive collapse under extreme abnormal loading conditions. In the "*Best practice for reducing the potential for progressive collapse in buildings*" published by NIST [1], the potential abnormal load hazards that can trigger progressive collapse are categorized as: aircraft impact, design/construction error, fire, gas explosions, accidental overload, hazardous materials, vehicular collision, bomb explosions, etc. As these hazards could be considered to have low probability of occurrence for structures of normal importance, thus they are either not considered in structural design or addressed indirectly by passive protective measures, yet they are seen to be important to be considered for important and susceptible structures. Most of these hazards have characteristics of acting over a relatively short period of time and result in dynamic responses. Despite the probability of the hazard occurrence, progressive collapse of a building has significant socio-economic impacts.

In progressive collapse, an initial localized damage or local failure spreads through neighbouring elements, possibly resulting in the failure of the entire structural system. The most viable approach to limiting this propagation of localized damage is to maintain the integrity and ductility of the structural system. The ASCE 7-05 commentary [2] suggests a general design guidance for improving the progressive collapse resistance of structures, but it does not provide any specific implementation rules. Recent design procedures to mitigate the potential for progressive collapse in structures can be found in two design guidelines issued by the U.S. which are the General Service Administration (GSA) [3] and the Department of Defense (DoD) [4].

In a recent investigation, Kim and Kim [5] studied the response of steel moment resisting frames using alternative load path with different damage scenarios when a corner, a first edge and internal edge column are removed. Applying static, nonlinear static and dynamic analyses, they found that nonlinear dynamic analysis is the most precise, yet the results varied more significantly depending on the variables such as applied load, location of column removal, or number of building story. Also, they found that the potential for progressive collapse was highest when a corner column was suddenly removed, and that the potential for progressive collapse decreases as the number of storey increases. Fu [6] assessed the response of a 20-storey building subjected to sudden loss of a column for different structural systems and different scenarios of column removal. One of his concluded results is that under the same general conditions, a column removal at a higher level will induce larger vertical displacement than a column removal at ground level. Also, the researcher concluded that the dynamic response of the structure is mainly related to the affected loading area after the column removal.

GSA [3] and DoD [4] guidelines recommended the use of the direct approach or the Alternate Path Method (APM). In this

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Notations

GSA	General Service Administration
UFC	Unified Facilities Criteria (DOD 2005)
ESC	Edge Short Column
CC	Corner Column
IC	Internal Column
FIC	First Internal Column
ELC	Edge Long Column
FELC	First Edge Long Column
E	Modulus of elasticity
F_y	Yield strength of steel
K_o	Initial stiffness of the beam
M_p	Plastic moment of the beam
I	Moment of inertia of the beam
$\theta_{upgr.,s}$	Upgraded chord rotation after increasing the strength only
$\theta_{upgr.,s,k}$	Upgraded chord rotation after increasing the strength and stiffness
TF	Tie Force in the beam
μ_{Δ}	Displacement ductility demand of the beam
R_s^{θ}	Reduction factor in chord rotation due to increase in strength
R_k^{θ}	Reduction factor in chord rotation due to increase in stiffness
R_s^{T}	Reduction factor in tie force due to increase in strength
R_k^{T}	Reduction factor in tie force due to increase in stiffness
R_s^{μ}	Reduction factor in displacement ductility demand due to increase in strength
R_k^{μ}	Reduction factor in displacement ductility demand due to increase in stiffness
α_s	Strength factor due to increase in strength
α_k	Stiffness factor due to increase in stiffness
$\alpha_{s,k}$	Upgrading factor due to increase in strength or stiffness

method, a single column in the ground level is typically assumed to be suddenly missing, and an analysis is conducted to determine the ability of the damaged structure to bridge across the missing column. The APM is mainly concerned with the vertical deflection or the chord rotation of the building after the sudden removal of a column. The chord rotation is equal to the vertical deflection at the location of the removed column divided by the adjacent beams' span. As such, it is a threat-independent design-oriented method for introducing further redundancy into the structure to resist propagation of collapse.

Existing buildings that were designed for gravity loads or designed according to earlier codes are expected to have inadequate resistance to progressive collapse. Steel frame structures designed to earlier codes did not behave well during extreme hazard event due to insufficient carrying capacity [7]. One of the major challenges for a structural engineer is choosing a retrofit scheme for an existing steel structure with a potential for progressive collapse. Another challenge is deciding on the level of protection against such potential event of sudden loss of a supporting column. It is not a normal practice in retrofitting to attempt to make the existing structure comply with the present code provisions, as this approach may not be economic. Alternatively, it is proposed that the retrofit objectives for a structure that is susceptible to progressive collapse should rather depend on a performance-based criterion to ensure a predefined level of damage or to prevent collapse of the building. This approach is similar to the Performance-Based Seismic Design (PBSD) recently adopted by several guides [8,9].

The retrofit strategy may involve targeted repair of deficient members, providing systems to increase stiffness and strength or providing redundant load carrying systems by a structure system such as mega truss or vierendeel trusses at the top of the building or by using bracing systems that redistribute the loads through the entire structure. In general, a combination of different strategies may be used in the retrofitting of the structure.

2. Problem definition

The ductility of steel alone cannot guaranty that the steel building will not collapse under extreme loading. Progressive failure in steel buildings occurs due to insufficient strength in the beams that are needed to bridge the load from the removed column location to the adjacent columns. Upon column removal, the vertical load is transferred to the adjacent columns, where the resulting increase in the axial load of these columns is relatively small. On the other hand, the loss of a column will result in a significant increase in the flexure and shear demand on the adjacent beams. As such, upgrading the beams by increasing their strength and/or stiffness is expected to reduce the progressive collapse of steel buildings. In case of high hazard event where more than one column is expected to be lost, upgrading both beams and columns might be needed.

The objective of this paper is to assess the effectiveness of three different retrofit strategies for beams on the dynamic response of an existing high-rise steel structure when subjected to six damage scenarios by sudden removal of one of the columns at the ground level. The three studied retrofit schemes are by increasing the strength, stiffness, and both strength and stiffness of the beams. The effectiveness of the retrofit methods of damaged buildings is evaluated by comparing three performance indicator parameters, namely, chord rotation, tie forces, and displacement ductility demand of the beams after being upgraded to those of the original existing structure. Two sets of analyses are conducted. First set is conducted on a building with bay span of 6.0 m in order to evaluate the reduction factors in the three performance indicator parameters due to the three studied retrofit strategies. Second set is conducted on three buildings with spans of 5.0 m, 7.5 m, and 9.0 m in order to assess the effect of variation of bay span.

3. Details of the analytical models

Four 3-D models of 18-storey high-rise steel moment resisting frame buildings having 3×6 bays in plan were constructed using Extreme Loading for Structures (ELS) software [10]. The buildings have the same plan throughout the whole height. For each building, the sizes of the columns were kept constant for every three stories along the height; whereas two sizes for the beams were designed and kept constant for the whole height, namely, perimeter beams and internal beams. The studied models have bay spans of 5.0 m, 6.0 m, 7.5 m, and 9.0 m in the two directions. The buildings were designed according to CISC-95 [11] for gravity loading condition. Figs. 1 and 2 show the elevation and plan of the studied buildings, respectively, along with their respective column and beam sizes.

The frame columns and beams were designed to carry a slab thickness of 200 mm. The floors are subjected to a live load of 2.4 kPa, representing a load of an office building, and a superimposed dead load of 2 kPa was taken into account for the equivalent load from interior partition, mechanical and plumbing loads. In the model, a bilinear stress–strain relationship of the steel members was taken, with Fy = 350 MPa, and strain hardening of 1% as shown in Fig. 3. Modulus of elasticity, shear modulus, and Poisson's ratio for steel were taken as 200 GPa, 81.5 GPa, and 0.2, respectively. In the model, the inherent damping due to yielding of steel was taken into account as stated in the technical manual of ELS [10], whereas the external damping was neglected.