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Storey-based stability of unbraced structural steel frames subjected to variable fire loading

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ABSTRACT

Structural steel has poor fire resistance properties and often requires thermal protection. Passive thermal protection systems such as insulation for steel members by application of spray-applied fire-resistance materials (SFRM) on surfaces of structural steel members are expensive and represent a significant portion of building costs for steel structures. Also, design codes around the world are progressing toward performance-based approaches rather than prescriptive-based approaches in design for fire safety. However, it is difficult to account realistically for all probable fire scenarios, and the actual fire resistance of a structure may vary significantly depending on the nature of the fire, location of origin and characteristics of the building. A means to define and identify the maximum and minimum fire loading scenarios causing instability of a given structure is therefore desirable. Presented in this paper is a novel global optimization approach for determining the highest temperature, lowest temperature, most localized, and most distributed fire scenarios causing instability for an unbraced structural steel frame. The investigation assumes that the columns are fire protected but the beams are unprotected, and may be extended to apply in other configurations and framing materials.

1. Introduction

Structural steel frames must be thermally protected from fire loadings, due to the inherent ability for steel to conduct heat and degrade quickly under high temperatures. However, the cost of insulation represents a significant portion of building costs and it may be worth considering the removal of fire protection where it is deemed excessive. In particular, the Cardington tests [1] demonstrated that steel frames perform better when considered as a whole structure than that predicted from the testing of individual components. Since then, researchers have been developing methods for evaluating the stability of a steel frame under fire conditions [2–5]. This paper proposes a novel model for investigating the stability conditions of single-storey steel frames with fire-protected columns and unprotected beams when subjected to variable fire loading. It is unique in that the methodology for evaluation is in the form of an optimization problem that determines the extreme fire scenarios causing instability for a single-storey structural steel frame. The formulation of the optimization problem is based on that of the stability of unbraced steel frames subjected to non-proportional/variable loading in ambient temperature originally devised by Xu [6], and is an extension to account for the effects of elevated temperature proposed by Zhuang [7]. The variables in the optimization problem are the temperatures assigned to each of the beams in the bays of the model. The formulation is used to identify the highest and lowest temperature scenarios causing instability of the frame, as well as the most localized or distributed fire scenarios causing instability. Additionally, a method for determining the damage contribution of the columns is presented, which indicates the most vulnerable regions of the model when the frame is subjected to fire. Finally, two numerical examples are provided to demonstrate the efficiency of the proposed approach.

A brief review of the theory on storey-based frame stability and structural design for fire safety is presented in Sections 2 and 3, respectively. The proposed formulation is presented in Section 4, and the numerical examples are demonstrated in Section 5.

2. Storey-based frame stability

The overall stability of steel frames subjected to proportional loading was first addressed by Yura [8], who noted that in determining the stability of unbraced frames the stiffness contribution and interaction of all members in the frame must be considered, and that storey buckling must occur with all of the columns buckling simultaneously. This idea was extended by LeMessurier [9] and Lui [10], who each proposed new methodologies for evaluating the storey stability of frames.

Xu [6] proposed a global optimization formulation for determining the minimum and maximum variable loading scenarios that would

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result in buckling of unbraced frames. The overall lateral frame stiffness of the frame is evaluated based on the applied load at each column which is the variable of the optimization problem. The formulation abandons the traditional approach of assuming proportional loading, but instead determines the minimum and maximum loading cases that result in storey-based instability of a frame. The basis behind the formulation is that if the overall lateral stiffness of the frame is less than or equal to zero, then the frame is laterally unstable. The overall lateral stiffness of the frame, S_T, is the sum of the stiffness of the individual columns, $S_{c,i}$, defined by Eq. (1):

$$S_T = \sum_{i=1}^{n} S_{c,i} = \sum_{i=1}^{n} \frac{12E_{c,i}I_{c,i}}{L^3_{c,i}} \beta_{c,i}(\phi_{c,i}, r_{l,c,i}, r_{u,c,i}) \tag{1}$$

where $E_{c,i}$ is the modulus of elasticity of an individual steel column, $I_{c,i}$ is the moment of inertia of the column, $L_{c,i}$ is the length of the column, and $\beta_{c,i}(\phi_{c,i},r_{l,c,i},r_{u,c,i})$ is a stiffness modification factor of the column which accounts for stiffness degradation associated with column axial load and is defined in Eq. (2) [6]. Note that the subscript c corresponds to properties of the columns, and n is the number of columns.

$$\begin{gathered}\beta_{c,i}(\phi_{c,i}, r_{l,c,i}, r_{u,c,i}) = \frac{{\phi_{c,i}}^3}{12} \frac{a_1\phi_{c,i}\cos\phi_{c,i} + a_2\sin\phi_{c,i}}{18r_{l,c,i}r_{u,c,i} - a_3\cos\phi_{c,i} + (a_1-a_2)\phi_{c,i}\sin\phi_{c,i}} \\ a_1 = 3\left[r_{l,c,i}(1-r_{u,c,i}) + r_{u,c,i}(1-r_{l,c,i})\right] \\ a_2 = 9r_{l,c,i}r_{u,c,i} - (1-r_{l,c,i})(1-r_{u,c,i}){\phi_{c,i}}^2 \\ a_3 = 18r_{l,c,i}r_{u,c,i} + a_1{\phi_{c,i}}^2\end{gathered} \tag{2}$$

where $\phi_{c,i}$ is the load parameter equal to $\pi\sqrt{P_i/P_{ei}}$ in which P_i is the applied axial load, P_{ei} is the Euler buckling load, and $r_{l,c,i}$ and $r_{u,c,i}$ are the lower and upper end rotational fixity factors of the column connections, respectively. The end fixity factors vary from 0 (pinned) to 1 (fixed) depending on the end conditions of the column, and are functions of the rotational stiffness, $R_{u,c,i}$ or $R_{l,c,i}$, at the corresponding end connections [6]:

$$r_{u,c,i} = \frac{1}{1 + 3E_{c,i}I_{c,i}/R_{u,c,i}L_{c,i}}; \quad r_{l,c,i} = \frac{1}{1 + 3E_{c,i}I_{c,i}/R_{l,c,i}L_{c,i}} \tag{3}$$

By constraining the overall lateral stiffness of the frame to zero and varying the magnitudes of the loads at the columns, different combinations of loads resulting in lateral instability of the frame can be identified. The formulation can then identify the maximum and minimum total frame loads causing instability by using the objective function, Z, in Eq. (4a) [6]:

$$Z = \sum_{i=1}^{n} P_i \tag{4a}$$

The objective function is subject to the zero lateral stiffness and individual column buckling constraints in Eqs. (4b)–(4c), respectively:

$$S_T = \sum_{i=1}^{n} S_{c,i} = \sum_{i=1}^{n} \frac{12E_{c,i}I_{c,i}}{L^3_{c,i}} \beta_{c,i}(\phi_{c,i}, r_{l,c,i}, r_{u,c,i}) = 0 \tag{4b}$$

$$P_{1i} \le P_i \le P_{ui} = \frac{\pi^2 E_{c,i}I_{c,i}}{(K_{c,i}L_{c,i})^2}; \quad (i = 1, 2, \ldots, n) \tag{4c}$$

where P_{li} and P_{ui} are the lower and upper limits of the axial load P_i on column i, and n is the number of columns in the frame. The lower bound, P_{li}, can be taken as either zero or the compressive axial load associated with the service dead load. The upper bound, P_{ui} is required to prevent non-sway buckling of individual weak columns. The effective length factor $K_{c,i}$ is related to the end fixity factors of the column derived in [11], as follows:

$$K_{c,i} = \sqrt{\frac{(\pi^2 + (6-\pi^2)r_{u,c,i}) \times (\pi^2 + (6-\pi^2)r_{l,c,i})}{(\pi^2 + (12-\pi^2)r_{u,c,i}) \times (\pi^2 + (12-\pi^2)r_{l,c,i})}} \tag{5}$$

The minimum solution of the formulation stated in Eqs. (4a)–(4c) can be defined as the "worst-case scenario" since it represents the least amount of loading required in order for the frame to become unstable. In contrast, the maximum loading case can be defined as a "best-case scenario" since it represents the highest possible load capacity of the frame.

3. Structural design for fire safety

The fire resistance of a structure is a broad term that can be defined as the duration of a fire, temperature of a member, or loading capacity that, if exceeded, results in failure of the structure, where failure can be defined in various ways [12]. Fire resistance can be evaluated using standard testing such as the ASTM E119 [13], ISO-834 [14], and Eurocode [15], or by analytical methods involving calculations. Standard fire curves are used in the tests because "the number of possible fire scenarios is usually too large and the analysis of each one is not practicable" [16]. For this reason, the testing methods for evaluating fire resistance do not always provide reliable estimates of the actual fire resistance of a structure during fire events. Also, standard testing is performed on individual structural elements without considering their interactions with the entire structural system as a whole. Furthermore, standard fire testing is expensive and time consuming. As such, reliable and simple analytical methods are preferred over the testing methods for determining the fire resistance of structures.

Analytical models for evaluating fire performance of steel structures have been developed over the past few decades. Of particular relevance is the definition of a relationship between the temperature of steel and its load carrying capacity. The evaluation of stability for steel members subjected to elevated temperatures involves reducing the elastic modulus using a degradation factor, such as the one presented in [17] and expressed in Eq. (6).

$$E_T = \lambda_T E_{20}, \quad \lambda_T = \begin{cases} 1.0 + \dfrac{T}{2000\ \ln(T/1100)}, & 0^\circ C < T \le 600^\circ C \\ \dfrac{690 - 0.69T}{T - 53.5}, & 600^\circ C < T \le 1000^\circ C \end{cases} \tag{6}$$

where E_T is the modulus of elasticity at elevated temperature T, λ_T is a material degradation factor accounting for the reduced properties at elevated temperature, and E_{20} is the modulus of elasticity at ambient temperature. A plot of the material degradation factor versus temperature by this relation is shown in Fig. 1.

In combining the relations for reduced properties of steel with the storey-based stability optimization program results, the formulation stated in Eqs. (4a)–(4c) can be extended to account for variable fire loading.

3.1. Thermal insulation

Due to their high thermal conductivity and degrading strength properties at elevated temperatures, the use of steel structural members most often requires for the steel to be protected with thermal insulation such as SFRM, which introduces additional costs to steel construction. The British Research Establishment [1] conducted a series of fire tests on a full-scale eight-storey steel frame in its laboratory in Cardington, UK. The tests varied the amount of fire protection applied on the beams and columns in the model, and assessed the damage to the structure under various fire scenarios. Based on the results, Wang and Kodur [18] concluded that while fire protection is essential for the columns in order to limit widespread damage to the structure due to buckling, the