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A stiffness reduction method for the in-plane design of structural steel elements

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ABSTRACT

Stiffness reduction offers a practical means of considering the detrimental influence of geometrical imperfections, residual stresses and the spread of plasticity in the analysis and design of steel structures. In this paper, a stiffness reduction approach is presented, which utilises Linear Buckling Analysis (LBA) and Geometrically Nonlinear Analysis (GNA) in conjunction with developed stiffness reduction functions for the design of columns and beam-columns in steel frames. This approach eliminates the need for modelling geometrical imperfections and requires no member buckling checks. For columns, inelastic flexural buckling loads can be obtained using LBA with appropriate stiffness reduction, while GNA with stiffness reduction is required to determine an accurate prediction of beam-column failure. The accuracy and practicality of the proposed method is shown in several examples, including regular and irregular members. For the latter case in particular, it is found that the proposed approach provides more accurate capacity predictions than traditional design methods, when compared to results generated by means of nonlinear finite element modelling.

1. Introduction

The development of plasticity within the members of a steel frame may significantly influence its overall response, generally resulting in a member force distribution different from that obtained through elastic analysis. In current design specifications [1,2], this is implicitly accounted for by employing effective length or notional load approaches in conjunction with member design equations. An alternative design strategy is based upon the use of stiffness reduction concepts [3–9]. Recent studies have focused on the enhancement of plastic hinge analysis through member stiffness reduction to take account of the spread of plasticity along the member length [10]. Orbison [11] investigated the use of a stiffness reduction function obtained from the Column Research Council (CRC) column strength curve [12], while Liew [13] proposed a refined plastic hinge approach using a smooth stiffness reduction function for plastic hinges in conjunction with a stiffness reduction function derived from the LRFD inelastic flexural buckling formulation [14] for members. Ziemian and McGuire [15] developed a stiffness reduction function considering the combined influence of minor axis bending and compression. Using the refined plastic hinge approach [13], Landesmann and Batista [16] derived stiffness reduction expressions using the European column buckling curves in lieu of the LRFD column buckling curve. Barszcz and Gizejowski [17] proposed theoretical models for compression members using the European buckling curves to determine different stiffness reduction expressions for axial and flexural stiffness as functions of member non-dimensional slenderness. Finally, Zubydan [18,19] proposed stiffness reduction functions to capture the development of plasticity at cross-sectional level.

The above studies have generally focused on the development of stiffness reduction schemes for use in plastic hinge analysis, which is not widely used in practice. More recently, Maleck [20] and Surovek-Maleck and White [21,22] proposed the use of stiffness reduction in conjunction with Geometrically Nonlinear Analysis (GNA) to account for the detrimental influence of the spread of plasticity on the response of steel frames. This method, which is included in the two most recent versions of the AISC-360 specification, including AISC-360-10 [2], can be readily applied using conventional structural analysis software, enabling the design of members without having to consider increased effective lengths associated with the sway buckling mode. Nevertheless, the stiffness reduction scheme suggested by Surovek-Maleck and White [21] does not fully capture the detrimental influence of the spread of plasticity, geometrical imperfections and residual stresses. Thus, this method still requires the use of column strength

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equations for member design, and may lead to overly conservative strength predictions when compared against accurate results from Geometrically and Materially Nonlinear Analyses with Imperfections (GMNIA).

To extend previous stiffness reduction approaches, a stiffness reduction method is presented herein that utilises more advanced stiffness reduction functions to capture fully the detrimental influence of the spread of plasticity, residual stresses and geometrical imperfections on the capacity of columns and beam-columns. According to the proposed approach, Linear Buckling Analysis (LBA) with stiffness reduction is used for the design of columns, while the use of Geometrically Nonlinear Analysis (GNA) with stiffness reduction and without considering imperfections is proposed for the design of beam-columns. Note that both LBA and GNA are elastic analysis methods. In the latter case, the section forces at the most heavily loaded cross-section are checked against the ultimate cross-section resistance. Unlike the design approach of Surovek-Maleck and White [21], the method proposed in this study does not require the use of column strength equations; instead only cross-section checks are necessary. Furthermore, the proposed approach does not require the explicit modelling of geometrical imperfections, and hence avoids the need to identify a suitable shape and direction.

In the following sections of this paper, an accurate finite element modelling approach used to analyse steel members is first described. The developed stiffness reduction equations for members under axial load, bending and combined axial load and bending are then presented. A series of design examples for regular and irregular members and a simple frame are shown to illustrate the application of the method. In all cases, the proposed approach is compared against accurate GMNIA predictions and the results obtained using current EN 1993-1-1 [1] formulations.

2. Finite element modelling

In this section, the key characteristics of the finite element modelling approach used in this study to represent the response of steel columns and beam-columns are presented. The models, which allow for geometrical and material nonlinearities and include residual stresses and geometrical imperfections, are initially validated against experimental results from the literature, and then used later in the paper for comparisons with the proposed design equations.

2.1. Development of finite element models

In the numerical simulations, the finite element analysis software Abaqus [23] was used, and the steel members were modelled utilising elastic-plastic beam elements. Such elements are suitable for analysing members which are not susceptible to local buckling, as is the case for all members considered in this study. The adopted element is named B31OS in the Abaqus element library [23] and is a Timoshenko beam element that accounts for transverse shear deformations and warping rotation. Thirty-three integration points were used for each flange and web of a generic I section to represent accurately the variation of strains and stresses within the cross-section and to capture the spread of plasticity. For numerical integration over the element length, the Simpson method [23] with one integration point located in the middle of each element was chosen. The Poisson's ratio was taken as 0.3 in the elastic range and 0.5 in the plastic range by defining the effective Poisson's ratio as 0.5 to allow for the change of cross-sectional area under load. The tri-linear elastic-plastic stress–strain relationship shown in Fig. 1 was employed, where E is the Young's modulus, E_{sh} is the strain hardening modulus, f_y and ϵ_y are the yield stress and strain

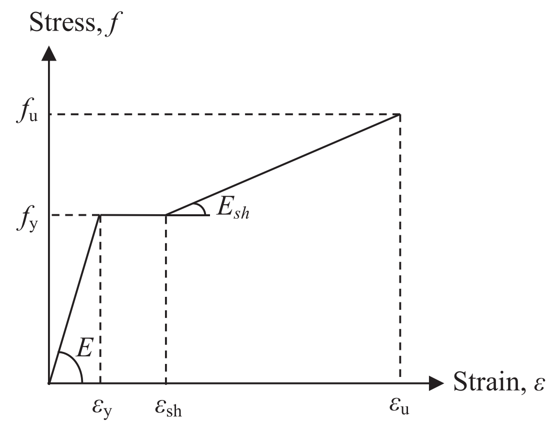

Fig. 1. Material stress–strain curves used in finite element models.

respectively, ϵ_{sh} is the strain value at the onset of strain hardening. The parameters f_u and ϵ_u correspond to the ultimate stress and strain respectively. E_{sh} was assumed to be 2% of E and ϵ_{sh} was taken as $10\epsilon_y$, conforming to the ECCS recommendations [24] for hot-rolled structural steel. Isotropic hardening and the von Mises yield criterion with associated plastic flow were assumed. The engineering stress–strain model shown in Fig. 1 was then transformed into the true stress–strain model according to the constitutive formulation used in Abaqus [23], which is based upon the Cauchy stress–strain assumption. S235 steel grade was used in all the simulations.

In the numerical models, the ECCS [24] residual stress patterns illustrated in Fig. 2 were employed to define the initial stress values at the section integration points through the SIGINI user subroutine [23]. The initial geometric member imperfections (i.e. out-of-straightness) were assumed to be half-sine waves in shape and 1/1000 of the corresponding member length in magnitude [25]. These imperfections and residual stresses are used in all GMNIA conducted throughout this paper unless otherwise stated.

2.2. Validation of finite element models

To validate the adopted finite element modelling approach, the experiments of Van Kuren and Galambos [26] on steel beam-columns were considered (Fig. 3). Additional results of the experiments are also provided in Galambos and Lay [27]. In the tests, an axial load was applied to the column first and then the bending moment, which was applied to only one end, was increased up to collapse; the specimens were restrained in the out-of-plane direction. Fig. 3 shows the experimental and numerical normalised moment-deformation curves for test specimens A5 and A7, in which N and M_y are the applied axial load and major axis bending moment, N_{pl} and M_{pl} are the yield load and plastic moment capacity and θ is the end rotation. Both specimens have a non-dimensional slenderness $\bar{\lambda} = 1.23$, where $\bar{\lambda} = \sqrt{Af_y/N_{cr}}$, in which A is the cross-sectional area, f_y is the material yield stress and N_{cr} is the elastic buckling load of the member. The numerical curves were obtained through GMNIA.

The close agreement between the experimental and numerical results shown in the figure indicates that the adopted finite element description can accurately predict the physical response of steel beam-columns. The discrepancy for the specimen A5, whose axial load level is $N/N_{pl} = 0.33$, may result from the difference between the geometrical imperfections assumed in the numerical model and the actual values, which are not reported in Van Kuren and Galambos [26]. On the other hand, owing to a relatively small axial load, geometrical imperfections are of less significance for the A7 specimen, thus resulting in a very accurate numerical prediction of the capacity of the member.