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Weak coupling of shell and beam computational models for failure analysis of steel frames ☆

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ABSTRACT

A hierarchical modeling of planar steel frames that is based on a weak coupling of shell and beam computational models is studied in detail. A shell finite element is used to compute failure response of a representative part of each column and each beam of a frame under consideration; elasto-plasticity, geometrical nonlinearity and material softening (along with a localization limiter) are taken into account. The computed results, which naturally include local buckling and/or localized material failure effects, are further incorporated into a beam inelastic stress-resultant constitutive model; the embedded-discontinuity-in-rotation Euler–Bernoulli finite element is considered in this work. The designed beam finite element formulation can be effectively used for the failure analysis of the steel frame. Important issues of such weak coupling concept are presented and results of several numerical simulations are discussed.

1. Introduction

A hierarchical approach to structural modeling is advantageous in many engineering applications. In general, it involves coupling of a higher-level computational model with a lower-level one. In the hierarchical modeling of planar steel frames the shell and the beam finite element formulations can represent the former and the latter, respectively. The shell model can capture localized instabilities (of both material and geometrical types) that are completely unseen by the beam model; however, the modeling effort and the computational cost could be very large if only shell elements are used to model the entire structure. The beam model offers effective and robust analysis; the beam analysis results are also more suitable for the design purposes. By coupling the models, the best of both worlds can be combined. The coupling can be done in a strong way; in that case, the finite element model of a frame consists of beam and shell elements, as well as of constraints connecting both element types in a consistent manner (e.g. [39,9]). The coupling of the computational models can be also weak (e.g. [2,14]). In that case, the shell element is used to compute the failure response of a representative part of each subsystem of a frame under consideration; this information is further incorporated into the beam element that is used for the analysis of the entire frame. Thus, the weak coupling concept involves carrying out sequential shell and beam computations.

In this work, we present, illustrate, and make an assessment of the weak coupling concept for the analysis of planar steel frames made of I-cross-section beams and columns. We note that the concept can be also applied for other frame member geometries, e.g. with I-cross-section and transverse (and longitudinal) reinforcements (e.g. [34]). We aim to compute the response of a steel frame under static load beyond its limit point. In the earthquake engineering, this kind of computation is called the push-over analysis; we will call it here the failure analysis. An accurate yet fairly simple failure analysis of steel frames is of great practical interest. The failure analysis is an indispensable tool in order to obtain important design parameters, such as the limit load, the limit ductility, and the failure mode.

Under large loads, a frame member may undergo plastic deformations. Moreover, plastic strain localization (i.e. material softening) and local buckling may occur. Inelastic geometrically nonlinear beam finite elements cannot capture automatically those complex effects. However, they can be implicitly included into the beam finite element formulation by using the weak coupling concept suggested in [14] that can be summarized as:

(a) Use a geometrically and materially nonlinear (i.e. elasto-plastic) shell finite element formulation (see e.g. [8,31,29,7,16,40,36,11,15]) in order to study response of the representative part of a frame member (RPFM) up to its complete collapse. Such a response naturally includes local

☆ This paper is dedicated to the memory of Darko Beg, professor of steel structures at the University of Ljubljana.

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buckling and/or steel localized failure, since the RPFM fails due to a subtle combination of localized material failure and local buckling. The shell element should be able to model branching shells: either by using a shell formulation with the drilling rotation (see e.g. [18,19,25,42]) or by using a shell formulation with rotations that transforms the two local rotations into the three global ones at the nodes where different branches of the shell meet (e.g. [32]) or yet by using a rotation-less solid-shell formulation (e.g. [38,13]). Also, the effect of the mesh-dependency at steel softening (i.e. at localized steel failure) has to be minimized. For that purpose a localization limiter (see e.g. [20]), or a more rigorous and complex approach, like the extended finite element method (X-FEM) (see e.g. [3,4]), is to be used.

(b) Incorporate the results obtained by the shell finite element into the stress-resultant beam inelastic constitutive model. In this work, we make use of the embedded-discontinuity-in-rotation Euler–Bernoulli beam finite element described in [5,14,21], which can model perfectly well, without mesh-dependency, the cross-section softening as well as the frame softening. The geometric nonlinearity has to be taken into account in the beam formulation, at least in its simplest form of the von Karman type (see e.g. [27,14]).

(c) Use the beam finite element, designed under (b) above, for an effective failure analysis of the entire steel frame; such an analysis is fast and implicitly takes into account localized

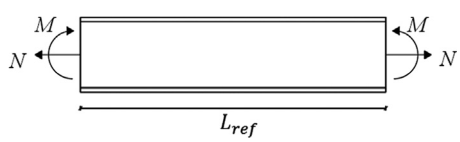

Fig. 1. Representative part of a frame member (RPFM) that is analyzed by shell finite elements.

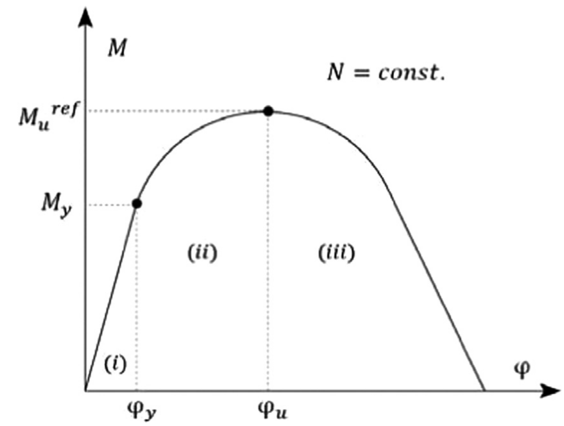

Fig. 3. Applied end-moment versus end-rotation ($M-\varphi$) curve obtained for the RPFM by shell computation. Rotation φ_u defines the border-point between the hardening and the softening.

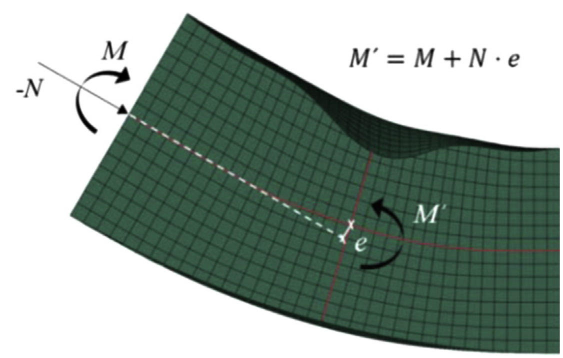

Fig. 4. Moment at a RPFM cross-section that is under failure for boundary conditions (a2).

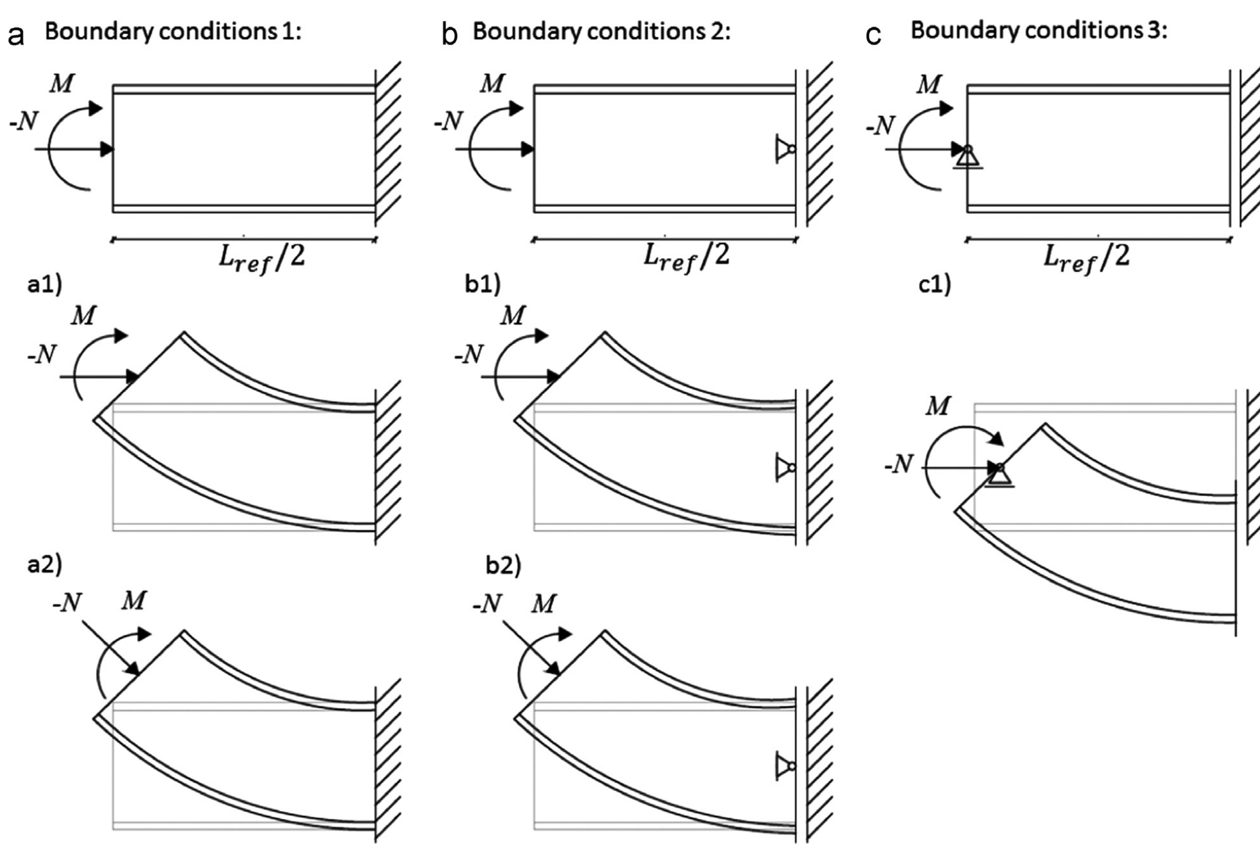

Fig. 2. Possible boundary conditions for the analysis of one half of the RPFM: (a2) is applied in this work and (c1) is used in [14].