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Plastic collapse load numerical evaluation of welded beam-to-column steel joints

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ABSTRACT

A finite element based numerical procedure for predicting the plastic collapse load as well as the plastic collapse mechanism of beam-to-column steel joints is presented. The promoted procedure is based on two methods following the static and the kinematic approach of limit analysis. Both methods have been rephrased for a von Mises type material in the deviatoric plane and in terms of deviatoric stress invariants. The key concepts are: i) in the static formulation, to mimic the stress redistribution arising within a structure approaching its critical (collapse) state, such stresses being in equilibrium with the maximum redistributable loads; ii) in the kinematic formulation, to build a plastic collapse mechanism characterized by compatible strain and displacement rates corresponding to a minimum value of loads doing positive work equal to the total plastic dissipation. A validation of the numerical results is pursued by comparison with experimental findings on real scale prototypes of the tackled steel joints. Future developments are outlined at closure.

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1. Research context, motivations and main goals

The structural analysis of steel joints is nowadays a matter solved by any commercial numerical code in the engineering fields. Both the constitutive behavior of steel and the post-elastic behavior of steel structural components are successfully described by commercial finite element (FE) codes in statics or in dynamics also in the presence of damaging processes. Sophisticated step-by-step and/or time-stepping algorithms are available robust tools to handle the analysis of steel structures. Also the accuracy and computational performance of such algorithms, when dealing with only-steel elements, is certainly the more competitive. So that, the here claimed evaluation of the plastic collapse load for welded beam-to-columns steel joints, avoiding the description of the post-elastic behavior of the addressed steel structural elements, might appear even outdated.

The problem here in mind is however related to the use of those steel structural components as parts of more complex structural systems characterized by the presence of other structural elements made of materials whose constitutive as well as post-elastic behavior is not easy to handle or it is described by criteria not possessing the necessary general applicability. This is actually a recurrent circumstance when dealing, for example, with seismic retrofitting of masonry or reinforced concrete existing structures often pursued by strengthening techniques which insert moment resistant steel frames or steel bracing systems, see e.g. [1], [2], [3], [4] and references therein. A similar circumstance appears also in composite steel-concrete structures, [5], [6], or in new composite steel-concrete structural elements as, for example, in concrete filled welded steel columns [7], [8], [9], [10] or in steel ribs for strengthening of steel concrete joints [11], or in composite beams [12], also in the presence of other materials of common use nowadays as fiber reinforced polymers [13].

The above list of papers, far to be exhaustive, gives the idea of the research context hereafter referred in which a conflict in the adoptable design methodologies arises. From one side, steel members and their mutual joints are described by well known constitutive criteria, as von Mises for example, and can be handled by plasticity or damage theories fully implemented in nonlinear FE codes. From the other side, structural elements made of masonry or concrete, whose constitutive as well as post-elastic behavior is not uniquely defined being also affected by the constructive techniques, are often treated with FE codes whose results are valid only for very particular cases or structural typologies. In this context a direct method, as Limit Analysis, with all its congenital limitations, can result more reliably to predict a limit load of plastic collapse for steel members and of rupture for the other ones giving rise to a complete and effective limit states design approach.

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The present work finds and tries motivations on the above considerations and, as a first step of the study, presents a limit analysis FE based procedure applied to steel joints. It is worth noting that, looking at a real engineering practical context, limit analysis has to be performed numerically, indeed a FE based friendly procedure is presented in the paper while the joints are focused being the weak points of a steel elements system. The promoted method, already experienced by the authors in different contexts [14], [15], [16], [17], [18], is here applied to welded beam-to-column joints to predict their plastic collapse load. Perfect associative plasticity is postulated and von Mises yield criterion in deviatoric plane is used throughout. Two different numerical techniques, based on the static and the kinematic approach of limit analysis respectively, are simultaneously applied to detect the plastic collapse limit load of the analyzed joints. The procedure and the related numerical findings are validated by comparison with experimental outputs on real scale prototypes, [19], to show the robustness and reliability of the numerical plastic limit state design when facing a real engineering problem.

2. Limit analysis via a FE-based procedure

2.1. Theoretical bases

In the realm of perfect plasticity, limit analysis gives the theoretical tools to determine the plastic collapse loads, i.e. the loads under which the structure, modeled as elastic-perfectly plastic, reaches a *critical state* in which large increases in plastic deformation become possible with little, if any, increase in loads.

For simplicity, but without loss of analytical rigor and practical effectiveness, in the following the loads are only the external surface actions applied to the structure, body forces are assumed negligible with respect to the formers. Moreover, as usual in this context, all the acting loads are expressed in terms of assigned reference loads, say $\bar{\mathbf{p}}$, multiplied by a single load multiplier, say P. To set the problem from an analytical point of view, let us now denote with V the volume, referred to a 3D Euclidean space, occupied by the analyzed structure whose external surface is $S = S_t \cup S_u$ where S_t is the portion where loads $P\,\bar{\mathbf{p}}$ act and S_u the portion where boundary kinematic conditions, say $\mathbf{u} = \bar{\mathbf{u}}$, are specified.

The cited theoretical tools are the well known *theorems of limit analysis*, based on the principle of maximum plastic dissipation valid only for standard materials as the one here assumed, and furnishing a lower bound, P_{LB}, and an upper bound, P_{UB}, to the plastic collapse load multiplier, say P_U. Borrowing from a classic textbook on plasticity, [20], the two theorems may be stated as follows:

Static or Lower bound theorem: the loads that are in equilibrium with a stress field that nowhere violates the yield criterion do not exceed the collapse loads. That is, if at every point within V exists a stress field $\tilde{\sigma}_j$ with $f(\tilde{\sigma}_j) \leq 0$, with j ranging over the components of the stress vector in principal stress space and $f(\sigma_j)$ denotes the yield function in the same space, and also the stress field $\tilde{\sigma}_j$ is in equilibrium with the applied loads $P_{LB}\,\bar{\mathbf{p}}$, then P_{LB} is a lower bound to the plastic collapse load multiplier P_U, i.e. $P_{LB} \leq P_U$.

Kinematic or Upper bound theorem: the loads that do positive work on a kinematically admissible velocity field at a rate equal to the total plastic dissipation are at least equal to the collapse loads. That is, if the acting loads are $P_{UB}\,\bar{\mathbf{p}}$; $\dot{\mathbf{u}}^c$ is a kinematically admissible velocity field whose related compatible strain rates, say $\dot{\varepsilon}_j^c$, have the direction of the outward normal to the yield surface $f(\sigma_j) = 0$ at $\sigma_j = \sigma_j^Y$, which means that $\dot{\varepsilon}_j^c = \dot{\lambda}\,(\partial f/\partial\sigma_j)$ with $\dot{\lambda} > 0$ being a scalar multiplier; σ_j^Y denotes the stresses at yield associated to $\dot{\varepsilon}_j^c$, then P_{UB} given by

$$P_{UB} = \frac{\int_V \sigma_j^Y\ \dot{\varepsilon}_j^c\ dV}{\int_{S_t} \bar{p}_i\ \dot{u}_i^c\ dS_t} \tag{1}$$

is an upper bound to the plastic collapse load multiplier P_U, i.e. $P_{UB} \geq P_U$.

It is well known that for standard materials the maximum value of P_{LB} and the minimum value of P_{UB} produced by the application of the two theorems are equal to each other and they also equal the collapse load multiplier P_U. The static and the kinematic approaches of limit analysis pursued to detect P_U are essentially techniques to maximize P_{LB} and to minimize P_{UB}, respectively. The above theorems, as well as the limit analysis approaches they generate, are well known and have been here recalled only for a better understanding of the numerical procedures applied throughout the present analysis both illustrated in the next section with the aid of a geometric, more intuitive, interpretation.

2.2. The FE-based limit analysis

The promoted FE-based procedure arises from the application of two different methods, namely the Elastic Compensation Method (ECM) and the Linear Matching Method (LMM), see e.g. [18], and references therein.

The ECM is aimed at determining the maximum value of loads, say $P_{LB}\,\bar{\mathbf{p}}$, in equilibrium with a plastically admissible stress field, at which the structure finds itself at a state of incipient collapse. It then operates in the spirit of the static approach. The key-concept of the ECM is to *mimic the stress redistribution* arising within a structure approaching its critical (collapse) state when subjected to loads increasing up to collapse. Indeed the greater are the acting loads the wider are the structure portions where the elastic (plastically admissible) stresses attain an admissible threshold given by the assumed yield condition. When such redistribution cannot take place anymore the structure enters its post-elastic (plastic in this context) phase and plastic collapse is readily manifested. Precisely, the load increase is achieved by the ECM performing *many sequences* of elastic FE analyses. At the end of each sequence the applied loads, say $P^{(s)}\,\bar{\mathbf{p}}$, with $P^{(s)}$ = load multiplier of the current sequence (s), is increased of a fixed increment. On the other hand, the stress redistribution is achieved by the ECM performing, for the current fixed loads $P^{(s)}\,\bar{\mathbf{p}}$ of the sequence, a number of FE analyses on the discretized structure in which a reduction of the elastic modulus is applied to the portions where the stress has attained the yield threshold.

The redistribution of the stresses associated to $P^{(s)}\,\bar{\mathbf{p}}$ is pursued iteratively and can be easily understood with reference to the sketch of Fig. 1 where the assumed von Mises yield surface in the deviatoric $\pi-$plane is given by a circle of equation $\rho^2 - \rho_y^2 = 0$, with $\rho := \sqrt{2J_2}$ (being J_2 the second deviatoric stress invariant) and $\rho_y := \sqrt{\frac{2}{3}}\sigma_y$ (being σ_y the uniaxial yield stress) is the circle's radius.

The sketch of Fig. 1 depicts the scalar value stresses computed at the $(k-1)$th iteration (or, equivalently, at the $(k-1)$th FE analysis within the current sequence) at each element in the FE mesh. Such value, say $\rho_{\#e}^{(k-1)}$ for the generic element $\#e$, is the average of the stress values computed at each Gauss point within the element. Among all the $\rho_{\#e}^{(k-1)}$ (with $\#e = 1, 2, \ldots$ total number of elements), the "maximum stress" in the whole mesh, named $\rho_{max}^{(k-1)}$ in Fig. 1, is detected. Such a stress point is, in practice, the stress point "farthest away" from the von Mises circle. If such maximum value is greater than ρ_y, as hypothesized in the sketch, the method tries to redistribute the current loads $P^{(s)}\,\bar{\mathbf{p}}$ performing a new (kth) FE analysis of the discretized structure where within the elements with a $\rho_{\#e}^{(k-1)}$ greater than ρ_y (like, for example, in elements $\#1, \#2, \#e, \#n$, in Fig. 1) the elastic modulus is *reduced*, *to bring* the not admissible stress onto the yield surface, according to the formula:

$$E_{\#e}^{(k)} = E_{\#e}^{(k-1)}\left[\frac{\rho_y}{\rho_{\#\,e}^{(k-1)}}\right]^2. \tag{2}$$

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