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ON THE INFLUENCE OF SEMIRIGID JOINTS ON THE SEISMIC RESPONSE OF ONE STOREY STEEL BUILDINGS

G. A. Venneri[1], G. De Matteis[2], and G. Brando[1]

[1] Department of Engineering and Geology, University "G. d'Annunzio" of Chieti-Pescara, Viale Pindaro 42, Pescara 65125, Italy
e-mail: greta.venneri@unich.it; giuseppe.brando@unich.it

[2] Department of Architecture and Industrial Design, University of Campania "L. Vanvitelli", Aversa (CE) 81031, Italy
gianfranco.dematteis@unicampania.it

Abstract

The paper investigates the influence of the actual behavior of beam-to-column and column base joints on the seismic vulnerability of one storey steel structures. A numerical study was conducted on as-built real steel buildings whose column bases and beam-to-column joints have a semirigid and partial-strength behavior, although they were originally designed as rigid and full strength. Finite element models of the selected buildings were developed, assuming both the theoretical and the actual joint behavior and their seismic performance was investigated through nonlinear time history analyses. Kinematic hysteresis models were assigned to semirigid joints to account for their actual stiffness, nonlinear behavior, and energy dissipation capacity. Then, fragility curves were derived, on both the theoretical and the as-built buildings, for various damage levels, with the aim of comparing the probabilities of exceedance of the selected limit states.

Keywords: Semirigid Joints, Steel Buildings, Nonlinear Dynamic Analysis, Seismic Performances, Fragility Curves.

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1 INTRODUCTION

When modeling steel structures, it is essential to carry out reliable predictions concerning their joint behavior, so to avoid possible errors in evaluating the performance of the whole structure in terms of strength and deformability. Conventional analyses of steel frames usually assume either perfectly rigid or nominally pinned connections, also because this simplification facilitates the analysis implementation and the application of manageable design procedures. However, the response of the analyzed structures that can be obtained under one of these hypotheses may need to be revised because most joints used in real practice transmit a limited amount of moment and/or undergo significant deformations under load [1], [2]. Therefore, to assess the real behavior of the frame, it becomes necessary to incorporate the effect of joints flexibility in the numerical analyses [3].

While it is recognized that as-built joints provide some rigidity, the evaluation of the actual restraining action of semirigid connections is a challenging task. In this regard, the scientific literature provides several methos for estimating the real behavior of joints, including experimental characterization, numerical, analytical or mechanical models, Among the analytical methods, in order to reproduce the joint moment-rotation curve, the Component Method, a mechanical modelling approach included in Eurocode 3 Part-1-8 [4]–[6], is definitely one of the most reliable [7].

The evaluation of the real joint behavior is of particular importance for one store steel buildings, whose structural conception usually includes the use of column bases, at least in one of the main directions of the building, and, when portal frames are present, of beam-to-column joints. In this framing, Italian industrial buildings are mostly made of one storey steel structures, frequently characterized by semirigid and partial-strength joints. In most cases, old steel buildings were designed under gravity and wind loads only, neglecting or underestimating the effects of seismic actions [8], [9]. The evaluation of their vulnerability is a task of paramount importance, in the light of the fact that these buildings usually contain valuable assets whose lost could mean relevant economic consequences.

Based on the remarks reported above, this paper aims at evaluating the influence of the presence of semi-rigid joints on the seismic performance of single-story steel buildings. The investigation is conducted by considering some case studies consisting in some Italian real industrial buildings whose constructional details (columns, connections, roof members, roof bracing) have been deduced from the original blueprints. The considered stock of structures is characterized by joints designed to be rigid and full-strength, but that, through the application of the Component Method, proved to be semi-rigid and partial strength. Finite element models were developed for each building, in which both the theoretical and actual joint behavior were reproduced. This allowed to compare the response for different earthquake intensities. In Section 2 a short description of the buildings and of the modeling are provided. Then (Section 3), the nonlinear dynamic analysis setup is described. Therefore, the results of the analyses are given in terms of fragility curves and a discussion about the observed differences is proposed. The seismic vulnerability was assessed to determine whether reinforcement interventions on the joints are necessary or not to increase the overall safety level of the considered buildings.

2 THE CONSIDERED CASE STUDIES

2.1 Description of structures and joints

The considered stock of of structures includes eight existing steel buildings, designed and built between the 1970 and 2018 in Parma (Italy), according to the codes and standards of that period. All the structures have industrial destinations and are characterized by construction elements that are very common for this structural typology: columns, truss beams, crane runway beams, connections, roof members, roof bracing. The buildings considered as case studies, numbered from 1 to 8, are shown in Figs. 1, 2, 3. Four buildings (Case Study 1 to Case Study 4) have structures with single or multi-bay portals in the transverse (X) direction, whereas concentrically braced frames are present in the longitudinal (Y) direction. Two buildings (Case Studies 5 and 6) have a portal structural scheme along both the longitudinal and transversal directions, thus resulting a spatial moment resisting frame. Case Studies 7 and 8 have masonry infills effectively connected to steel elements in the longitudinal direction. These infills have been schematized through the equivalent strut model, according to the current literature [10]–[12]. The roof of all buildings features purlins and braces to stabilize the scheme.

The numerical models accounted for the geometric dimensions and the boundary conditions declared by the designers in the original projects. In detail, column bases were assumed as fixed supports in the portal direction and pinned in the braced direction. Beam-to-column joints were assumed as pinned constraints in all but one building (Case Studies 1), in which they were designed to be moment-resisting joints. Members and plates featured a steel grades Fe360 (nominal yield stress f_y = 235 MPa; nominal strength f_u = 360 MPa; Young's modulus E = 210000 N/mm^2) or Fe430 (nominal yield stress f_y = 275 MPa; nominal strength f_u = 430 MPa; Young's modulus E = 210000 N/mm^2). For beam-to-column joints and column bases, bolts and anchor bolts of classes 5.6, 6.8, 8.8 were used. Based on the available information, it has been inferred that the foundations are made of isolated reinforced concrete blocks.

The examined column bases, designed to be rigid and full-strength, consist of the column welded to the base plate, which is connected, by a minimum number of four anchor bolts, to the concrete block of the foundation plinth. In addition, these joints, except for those in buildings 5 and 8, have vertical stiffeners welded to the base plate and the column.

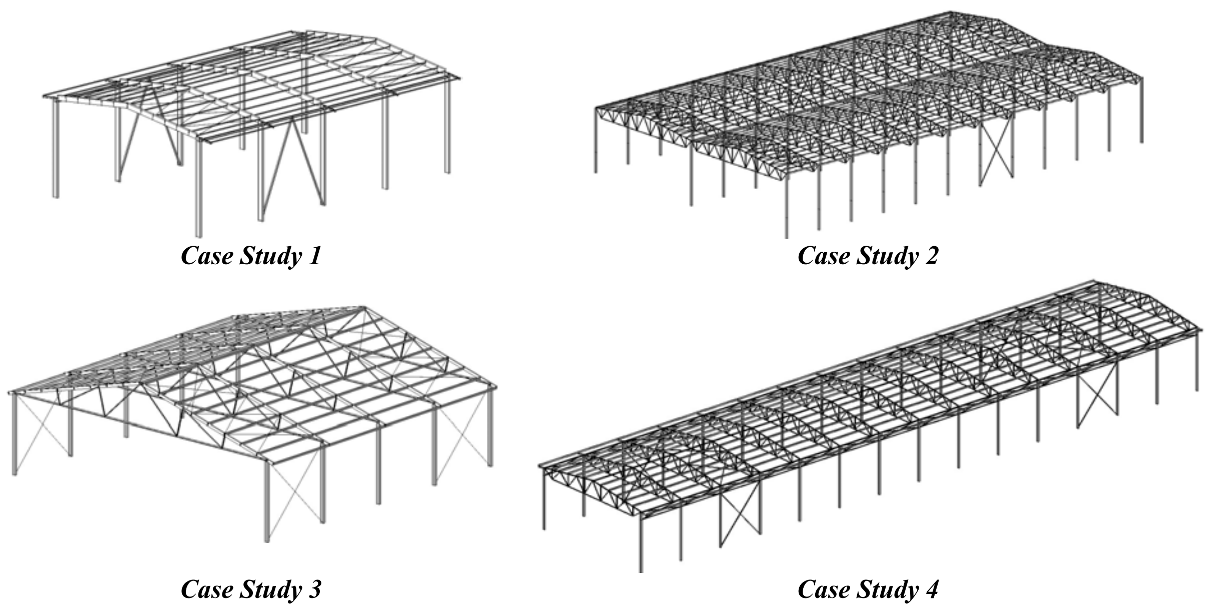
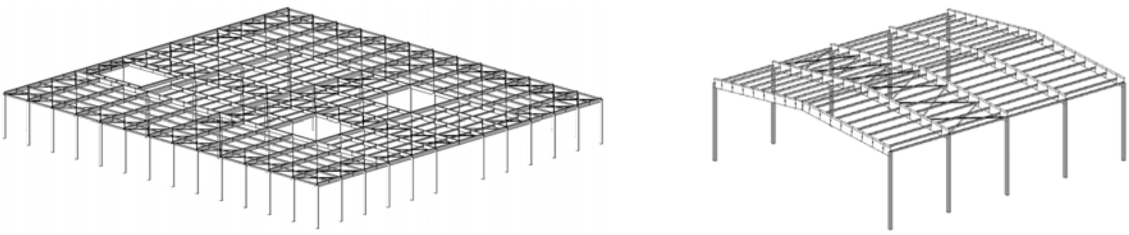
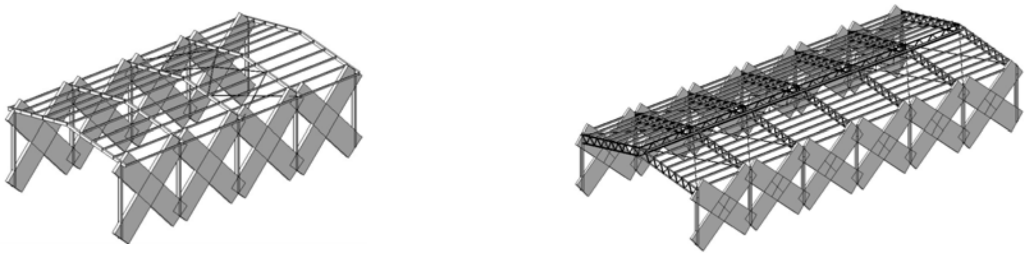


Figure 1: Finite element model of the CBF structures.



Case Study 5 *Case Study 6*

Figure 2: Finite element model of the MRF structures.



Case Study 7 *Case Study 8*

Figure 3: Finite element model of the structures with masonry infills.

2.2 Classification of joints

Beam-to-column joints and column bases of the buildings were classified by applying the Component Method [5], a reliable procedure for the mechanical characterization of connections adopted by Eurocode 3 (Part 1-8), which allows to evaluate the joint stiffness and the strength, as well as to approximate a moment-rotation curve [13], [14]. The analyses proved that all the joints of the studied frames are semirigid and partial-strength. Therefore, joints originally assumed to be continuous actually exhibits stiffness and strength characteristics that are lower than those predicted in the calculation process. Figure 4 shows the column base configuration of Case Study 4 and its M-θ diagram, which also shows the stiffness and strength limits that define the different classes of joints according to EC3.

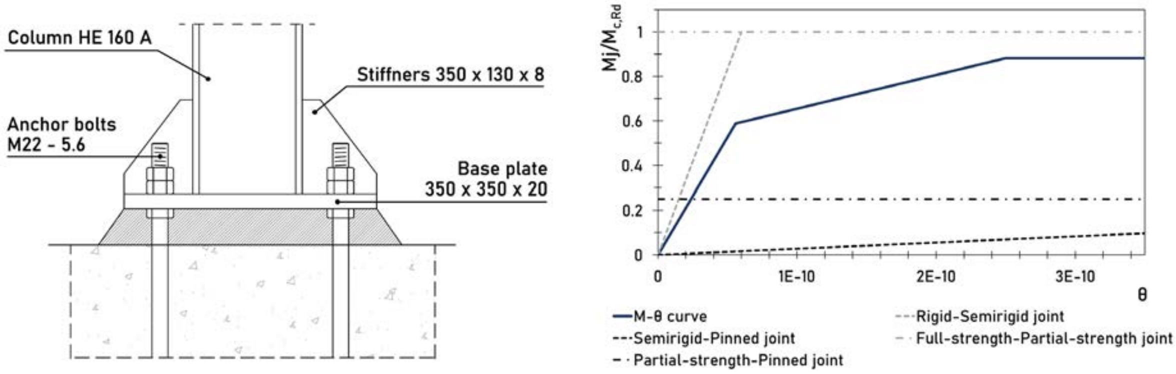


Figure 4: Column base of Case Study 4 and corresponding moment-rotation curve.

2.3 Numerical modeling of one storey steel buildings

Three-dimensional finite element models of the structures (Figs. 1, 2, 3) were developed within the software MIDAS Gen [15]. Beams and columns were modeled as one-dimensional linear elements. FEMA-type plastic hinges [16] were assigned to account for the nonlinear behavior of the most stressed sections, also considering the potential trigger of buckling phenomena. Thus, concentrated shear and bending hinges were implemented at the mid-length and ends of the elements, respectively. Members subjected to axial forces only, such as diagonal braces and trusses, were modeled as beam elements with moment releases at the ends, so to guarantee the uniaxial tension-compression behavior. Two models were developed for each building, differing in terms of joint modeling: “the theoretical model” with rigid and full-strength joints assumed consistently with the original project, and “the actual model” with semirigid and partial-strength joints, as resulted from the Component Method application.

In the theoretical model, the columns were modeled as cantilever elements in the portal direction (X) and as pinned elements in the braced direction (Y). In the actual model, nonlinear rotational springs were implemented at the base of the columns to model the rotational degrees of freedom of semirigid joints. A Bouc-Wen type hysteretic model [17]–[19] was adopted to characterize the rotational springs, whose stiffness and strength values were imposed based on the M-θ diagrams provided by the joint classification (Figure 4). Rigid link elements were used to model translational degrees of freedom, constraining all relative translational movements of the connected nodes [20]. In both theoretical and actual models, roofing members were modeled as pinned at the ends, except for Case Study 1, where fixed constraints were implemented in the theoretical model to reproduce the design assumption of moment-resisting joints, and rotational springs in the real mode to simulate the actual semirigid behavior.

3 ANALYSES AND RESULTS

Modal analyses and linear were initially performed on the finite element models. The results obtained for the theoretical and actual models were compared to evaluate the influence of the lower stiffness of the connections on the overall response of buildings.

For the seismic vulnerability assessment, nonlinear dynamic time history analyses were carried out. A set of 125 natural records grouped into families, each representative of a PGA value, were applied to the structures. Thus, the main uncertainty considered in the problem is related to ground motions (record-to-record variability) [21]. The records used in the present study were selected by a ReLUIS project conducted in past years (WP4 MARS Risk Map - 2019/2021) [20].

3.1 Multiple-stripes analyses

Multiple-stripes analyses were carried out based on the results of the nonlinear time history analyses to calculate the probability of exceeding the selected limit states for each group of records. The probabilities were estimated by assessing the percentages of cases in which demand exceeded the capacity. Demand corresponds to the peak roof-drift ratio produced by each accelerogram. Structural capacity values correspond to the performance levels specified in FEMA 356 [22]: 0.7% for Immediate Occupancy (IO), 2.5% for Life Safety (LS), and 5% for Collapse Prevention (CP). The three limit states have been associated with the damage levels D1, D2, and D3, respectively. In addition, the average PGA of each record family was considered as the measure of the seismic intensity (IM) of the problem.

3.2 Fragility curves

The probabilities resulting from the multiple stripes analyses were gathered and plotted in terms of fragility curves which are a useful way for representing, with a probabilistic meaning, the seismic performance of existing buildings [23]. Figs. 5, 6, 7 show the fragility curves obtained for braced structures, moment resisting structures, and structures with masonry elements, respectively, for the three limit states IO, LS, and CP. The suffix “R” and “SR” indicate the curved plotted for the model characterized by joints with theoretical behavior and for the as-built model with semirigid joints, respectively.

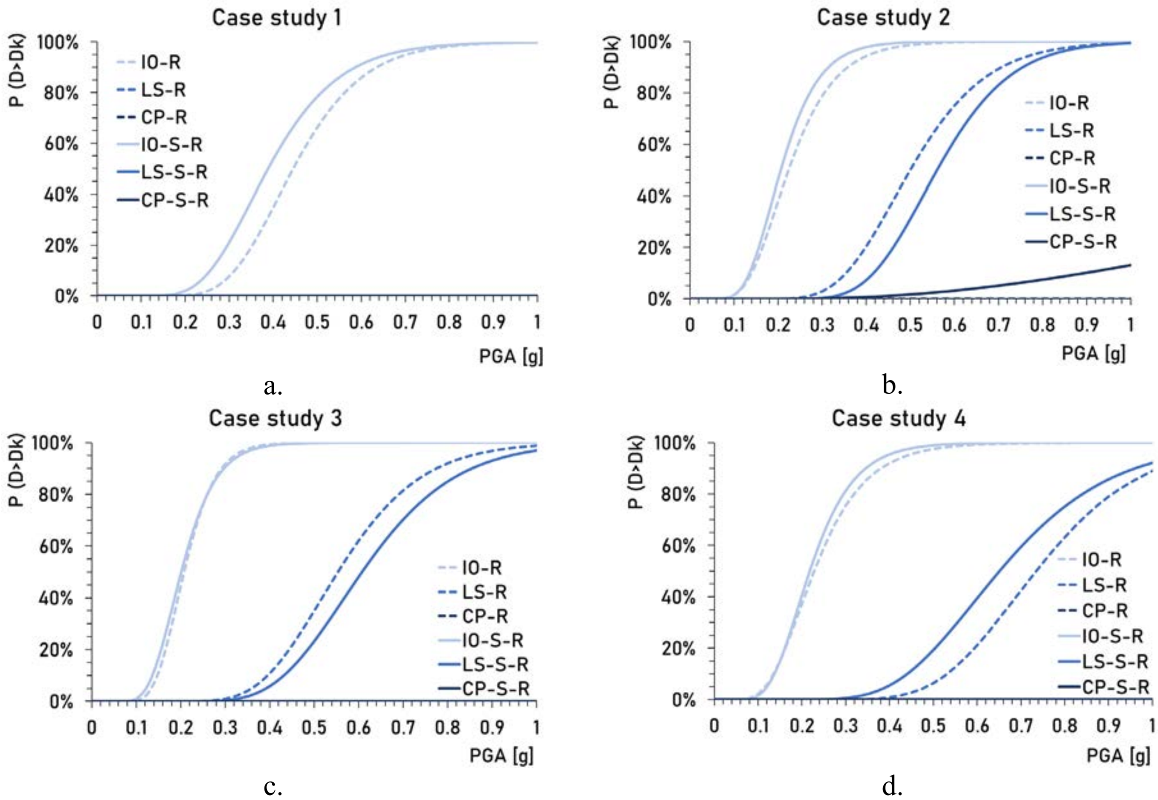


Figure 5: Fragility curves of CBF buildings: a) Case Study 1; b) Case Study 2; c) Case Study 3; d) Case Study 4.

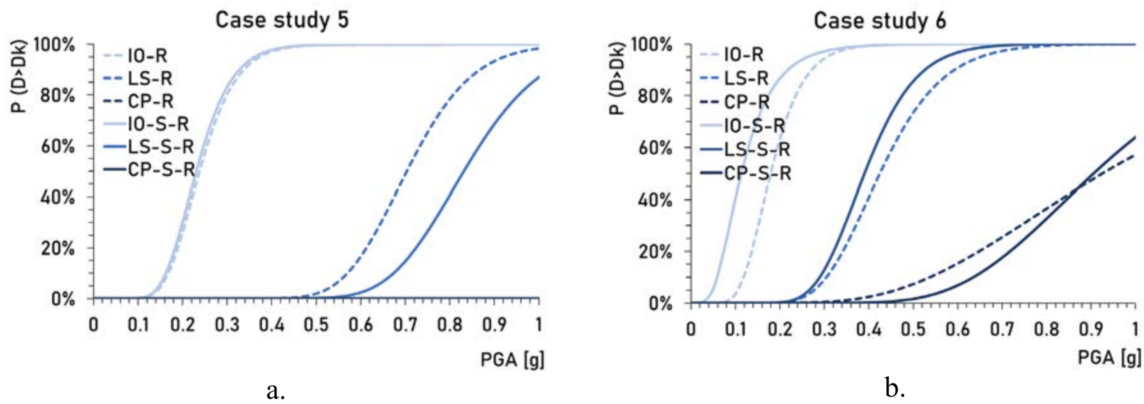


Figure 6: Fragility curves of MRF buildings: a) Case Study 5; b) Case Study 6.

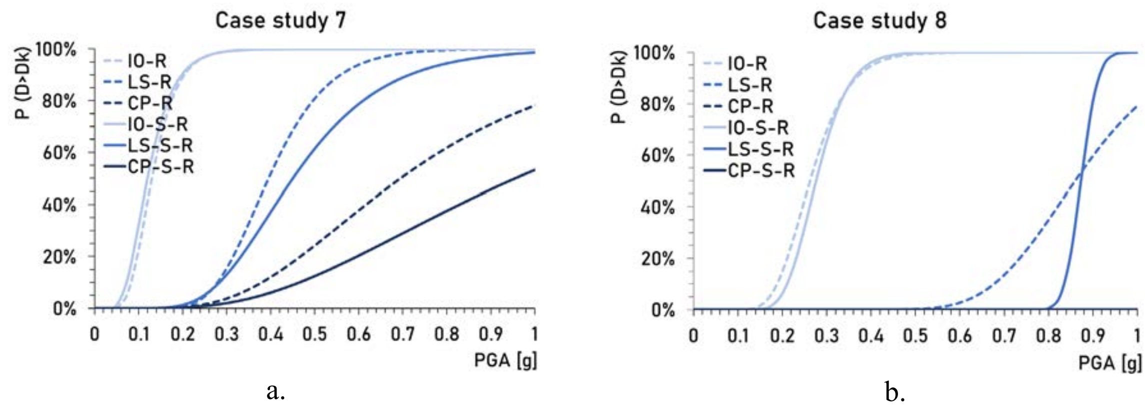


Figure 7: Fragility curves of buildings with masonry infills: a) Case Study 7; b) Case Study 8.

Almost all the buildings, except for Case Studies 6 and 7 (Figure 6(b) and Figure 7(a)) and the rigid model of Case Study 2 (Figure 5(b)), do not reach the CP limit state for the records used as seismic input. Even worse, Case Studies 1 and 5 do not reach the LS limit state (Figure 5(a) and Figure 6(a)). As it is possible to observe, the presence of semirigid joints affects the performance of the buildings, inducing a higher vulnerability to seismic actions, as it was found for Case Studies 1 and 4 (Figs. 5(a), 5(d)). Case Study 6 exhibits the same behavior, except at the limit state CP and for PGA value lower than 0.85 g (Figure 6(b)).

Conversely, for the Case Studies 2, 3, and 5 it was observed that the model characterized by rigid joints is more vulnerable at the SL limit state (Figs. 5(b), 5(c) and Figure 6(a)), but rigid and semirigid models behave almost similarly at the IO and CP limit states. Case Study 7, at the LS and CP limit states, is safer in the as-built configuration (Figure 7(a)). The fragility curves of Case Study 8 intersect and express a greater vulnerability of the semirigid model only from certain PGA values (Figure 7(b)).

4 CONCLUSIONS

This study deepened one of the main issues concerning the seismic behavior or existing steel structures, which are frequently characterized by semicontinuous joints, whose contribution to the inelastic response of the structural system cannot be neglected. With particular attention to existing single-story steel buildings, this paper provided a first attempt of evaluation of the safety gap due to a wrong interpretation of the joint behavior in the as-built configuration with respect to the theoretical one, the last being commonly associated to a pinned or a rigid behavior excluding the possibility of intermediate responses.

To this purpose, eight case studies of existing buildings were considered and fragility curves for three limit states (IO, LS, CP) were plotted and compared considering both the real and the theoretical behavior of the joints. From the comparison of fragility curves, it has been observed that the probability of exceeding the limit states is sometimes higher for structures characterized by the real behavior (semirigid joints). In such situations, it would be appropriate to design reinforcement interventions on the joints to increase the safety level of the whole structure. However, for some of the considered case studies, the reported fragility curves, at least for certain domains of seismic intensities, presented an inverse behavior, resulting safer in the as built configuration because of the beneficial effect due to the change of dynamic features. As such, further studies are necessary. These have to be oriented so to obtain information on the possibility of implementing retrofitting interventions on the basis of the knowledge of the variations of the whole stiffness of the building on the basis of the variation of the joint stiffness passing from the theoretical to the real behavior.

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