NON-LINEAR BEHAVIOUR OF STEEL-CONCRETE COMPOSITE MOMENT RESISTING FRAMES

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SUMMARY

Steel-concrete composite framed buildings are highly efficient structural systems due to their stiffness, strength and ductility. Such systems allow also adequate seismic performance nevertheless their application in seismic area is prevented by the lack of experimental information and design rules, especially about joints. The present work focuses on the seismic design and assessment of a typical steel and concrete composite multi-story moment-resisting frame, with 4 stories. The provisions implemented in European standards and guidelines are applied and discussed especially when uncertainties about their application come out; in fact a number of such provisions are not straightforward and/or reliable and further improvements are deemed necessary. After the frame has been designed by a linear analysis, a series of non-linear pushover analyses are performed by a lumped plasticity model introducing different models for the beam-column joint and plastic hinge length. In particular, since the definition of the plastic rotation capacity is not univocally defined, the formulations available for steel or reinforced concrete structures are discussed and evaluated also. The results are synthesized in terms of q-factor in order to assess the values suggests by the international codes.

Keywords: steel-concrete composite constructions, seismic design, composite frames, non linear analysis.

1. INTRODUCTION

The static nonlinear analysis is actually a very important tool for evaluating the performance of structures in seismic areas. However, the application of the method is not equally established for all type of structures. In the case of steel and concrete composite framed constructions, there is little information on the ductility (plastic rotational capacity) that can be assigned to the elements. This problem is particularly complex in the case of composite beams that have a strongly asymmetric behavior at sagging or hogging regions, furthermore are affected by uncertainty on the determination of the effective width, b_{eff} , and behavior of the beam-slab connection.

The experimental tests carried on the full-scale composite MRFs (Braconi et al. 2008) or parts of them (Bursi et al. 2000; Nakashima et al. 2007), even if limited, indicate how the performance under seismic actions of such systems could be compared with the steel or concrete ones.

Anyway, in the capacity design procedure of the composite frames, it is possible to identify different strategies for dealing with the problem of energy dissipation, which in some cases are typical of reinforced concrete structures and in other cases of steel ones. Aligned with the international standards guidelines, a strategy to exploit the dissipative capacity of the frames is to develop the plasticity at the ends of all beams instead of in the columns (strong columns / weak beams), and finally at the base of the columns. To achieve this result, the use of full-strength beam-column joints ensures that the plasticization is realized only in the beams and not in the joints. Numerical studies (Broderick et al.) focused on this approach, have shown that the ductility and dissipation capacity of the composite frames are greater than those expressed by the behaviour factors of the standard rules, proving that it could get a significant plastic rotation in beams and at the base column connection. However, on the basis of experimental studies (Braconi et al. 2008), it has also been verified the effectiveness of concentrating the plasticization in the elements of the connection, for example in the beam-column joint connection (end-plate), in the web panel of the column, and at the base column relying on the plasticity of the connection to the foundation. In this case, the use of semi-rigid and partial strength beam-column connections could be realized with the use of T-stub elements or angle, at least in the lower part of the beams (Green et al. 2004). Therefore, the design approaches could be varied according to appropriate capacity design criteria.

The columns can be realized with various solutions: fully encased, concrete filled (Shanmugam et al. 2001) and partially encased; moreover the base connection type could play an important role for the global ductility (Di Sarno et al. 2007) too.

The beams are generally made as a coupling between a steel profile in the lower part and a slab in the upper side, connected through stud connectors, which introduce a further element of variability in the structural behavior of the element. Moreover, if the rotational capacity of the composite beam in sagging region could be limited by the low ductility of the concrete, in hogging moment region can be limited by the local buckling of the steel profile; this limit can be assessed by defining a threshold value of plastic deformation (Kemp, 1985) or a critical stress (Kato, 1989) beyond which the instability occurs.

On the base of the high number of parameters that characterize the steel-concrete structures, the paper presents the results of a nonlinear static analysis of composite frames designed according to Eurocode 8 (2004), carrying out a detailed evaluation of the influence of the beams rotational capacity and beam-column joints deformability. The results are principally synthesized in terms of q-factor in order to evaluate the variability of this parameter and compare the obtained values with those suggests by international codes.

2. THE DESIGN OF THE COMPOSITE FRAME

The analyzed frame is extracted from the design of a multi-storey building used as offices. The building is regular both in elevation and plan. The plan dimensions of the building are: 31m in longitudinal direction and 24m in transverse direction, with a total covered area of 744 m^2. The height from the ground plan is 14.50 m for a total of 4 stories. The height of the first floor is 4 m, while all others are characterized by an interstory height of 3.5 m (Fig. 2.1).

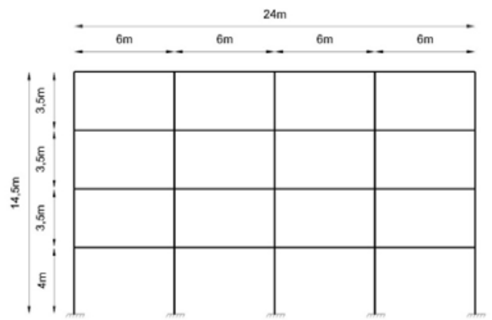


Figure 2.1. The frame analyzed.

Material	Class	Mechanical features				
		f_{ck}	f_{yk}	f_u	γ_m	E
		MPa	MPa	MPa	(-)	MPa
Conc	C20/25	20			1.50	29962
Steel bar	B450C		450	540	1.15	210000
Steel	S275		275	430	1.1	210000

Table 2.1: Mechanical characteristics of the materials.

Partially encased columns were used; the beams were composed with IPE profiles and reinforced concrete solid slabs with a height of 120mm connected by shear studs. The materials used in the design of the structure, and their mechanical features are given in Table 2.1. The design was developed according to the Eurocode 8 rules, applying the concept of capacity design by the column-beam strength hierarchy in bending and shear criteria for beams and columns. For the seismic actions characterization, reference was made to a site characterized by a medium-high level of seismicity with a PGA value of 0,08g for service limit state (in particular SLD) and 0,25g the ultimate limit state (ULS) (in particular SLV). The calculation of the stresses in the elements was arisen with the aid of the software SAP2000 (2011); both the performances at the ultimate limit state (ULS) and service limit state (SLS) were checked. For ULS,

verifications of the flexural and shear strength were considered for the static (vertical loads) and seismic (vertical and seismic loads) combination loads. For the SLS the stress and deflections were checked.
The definition of the effective widths of the beams was carried out according Annex C of the Eurocode 8 in all linear and nonlinear seismic analyses and the par. 5.4.1.2 of Eurocode 4 for vertical loading effects.
The characterization of the response spectra for SLS and ULS, was referred to the seismic hazard Italian classification. Particularly it was assumed a reference life of 50 years, a class of use II (Importance class II), a ground type C and a normal topographical condition.
Assuming in the design a high ductility class (DCH) and that the structure is regular, the adopted behavior factor is:

$$q_0 = 5\frac{\alpha_u}{\alpha_1}$$

$$q = q_o \cdot k_R = 5 \cdot 1{,}3 \cdot 1 = 6{,}5$$

By means of a first predimensioning step and an iterative design process, the final solution was identified.
On the base of seismic forces a satisfactory solution for the structural elements has been identified adopting beams IPE270 and columns HE320B for the first two stories and beams IPE240 and columns HE280B for the other ones. The effective width, evaluated according to the standard code, for beams vary along the longitudinal length and with the loading condition (vertical or horizontal loads). In the seismic condition, for beams under sagging moment has been assumed an effective width of 900mm that begin of 1200mm for those ones subject to hogging moment, both for internal and external joint.
The slab reinforcement was characterized by bars of 12mm diameter, spaced at 150mm. The partially encased columns are reinforced with 4 bars of 12mm diameter.
The shear connection of the beams was calculated according to the plastic theory with ductile shear studs (19mm diameter - class S275), with a spacing of 140mm over the entire length of the beams.
The modal elastic period of the 3D structure is 1.1s and the design spectral acceleration is 0.065g at ULS and 0.125 at SLS.

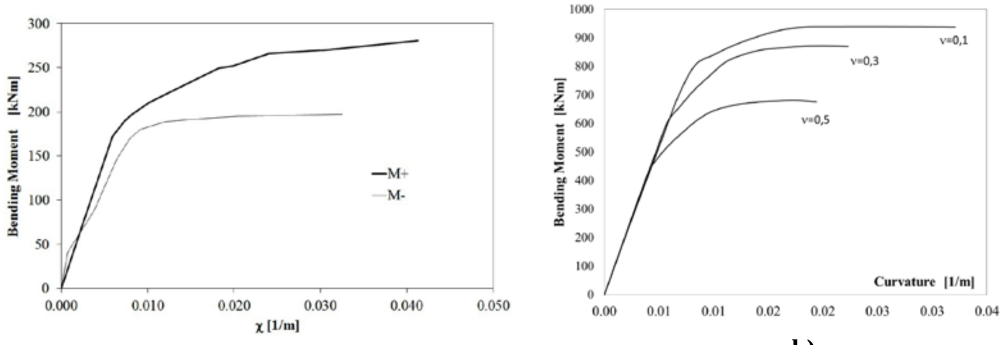
2.1 The non-linear analysis of composite MRF

After the elastic frame design, a nonlinear static analysis was performed to assess the actual resources of ductility of the adopted structural system. This analysis was carried out assembling a lumped plasticity model. Elastic frame elements were adopted for columns and beams with plastic hinges at the ends. The advantage of this modeling is that it allows working primarily with elastic elements which are less expensive from the computational point of view, leaving into few points the concentration of the material non-linearity. The limit of this modeling approach is that it requires some experience of the operator to determine where to place the plastic hinges and choose adequate moment-rotation relationships for these hinges, but for composite elements there are not well assessed formulations directly calibrated by experimental results as for RC elements (Paulay et al., 1992, see also the formulation of Eurocode 8 based on the study of Panagiotakos and Fardis, 2001). Therefore it would be interesting to analyze the rotational capacity evaluated by means the product of the plastic curvature of the section and the plastic hinge length (L_p); this last value is defined in the literature and codes for steel and RC elements basing on a lot of studies, but for composite beams the scientific resources are lack (Chen et al., 2008). Anyway this approach allows to take into account of the type of section, in this case composite beams (under sagging and hogging moment) and columns.
It is clear that the global response accuracy may be compromised if a wrong calibration of the moment-curvature relationship or plastic hinge length is done. Therefore, particular attention must be reserved for the estimation of the moment-curvature diagrams, even in the presence of axial load and in the prediction of an equivalent plastic hinge length so as to define a rotational ductility close to the real one. Such an approach is, however, inconsistent with the fact that the length L_p is not updated with the level of actual damage but it is a fixed size (Berry et al., 2008). However, in the current state of scientific knowledge, it seems to be the only viable for the steel-concrete composite structures. In the next paragraph (2.2.1), an analysis of the various L_p expressions and their impact on the overall response of the designed MRF is conducted; the expressions considered for the determination of the plastic hinge length are the ones

available for reinforced concrete structures, steel and steel-concrete composite structures in the technical literature. The formulations for the direct evaluation of the plastic rotation capacity of RC elements was not considered since are based only on experimental tests of RC elements conversely the introduction of the moment-curvature of the composite section could give surely a better approximation albeit the use of a plastic hinge length is referred to RC elements.
The designed frame is a case study thus the non-linear analysis is developed applying the modal and constant distribution of horizontal forces and give, as well-known, different results.

2.1.1 The moment-curvature relationships of beams and columns composite sections

The moment-curvature relationships of composite sections were tailored for the various sections of beams and columns assuming the Bernoulli hypothesis (the plane section remains plane) between the various components (concrete, structural steel, reinforced steel) and nonlinear constitutive relationships for the materials. The mechanical characteristics of the materials are the same used for the design and already summarized in Table 2.1.
For concrete, the Mander non-linear model (Mander et al, 1988), was implemented since it could take into account the confinement degree of concrete and in particular that of the composite columns between the flanges of steel profile. The ultimate strain of the confined concrete was determined by the expression reported in (Scott et al., 1998), that gives a value of 0.03 Instead, for unconfined concrete the ultimate strain of 0.5% was established (Scott et al., 1998). The stress-strain of concrete in tension was assumed linear-brittle. For a compression cylindrical characteristic strength, f_{ck}, of 20 MPa, according to the standards formulas of Eurocode 2 (2004), gives a tensile strength f_{ctm} = 2.21 MPa and an elastic modulus E = 29962 MPa, assumed equal in compression and tension.
The steel grade of the reinforcement is B450C (f_{yk} = 450 MPa), and for this one an elastic perfectly plastic (EPP) stress-strain curve was adopted.
Also for the steel of beams and columns an EPP stress-strain curve was adopted, with an ultimate strain of 1% and 2%, respectively in compression and in tension; the yield characteristic stress value, f_{sy} , is 275 MPa, both for the beams and columns. The limit value of deformation in compression was established to control, with an indirect way, the occurrence of local buckling phenomena (Kemp, 1985). This value seems too much detrimental especially in the case of the columns. For this reason the ε_{su} value for columns was based on the formulation of Elnashai et al. (Elnashai et al. 1998) obtaining about 3.5%. The moment-curvature diagrams for beams (Fig. 2.2a) show how the behavior is highly asymmetric comparing sagging and hogging bending moments. With regards to the columns, taking into accounts the confinement effects, the diagrams of Figure 2.2b are obtained. They show as the ductility and the strength decreases when the axial force ratio ν increases (ratio between the axial load and the axial strength of the section).



a) **b)**

Figure 2.2: The moment-curvature diagrams: a) for beams b) for columns

2.1.2 Parametric analysis on the influence of the plastic hinge length

For the choice of the plastic hinge length to be introduced in the model, some formulations that are indicated in the technical literature for the reinforced concrete and steel elements were considered, and in few cases for the steel-concrete composite beams also. In fact, for the R.C. structures over the years different expressions have been formulated (Table 2.2) based on experimental results (Paulay et al., 1992; Panagiotakos et al., 2001) while for steel elements a plastic hinge length approximately equal to the profile height itself is usually assumed (Bruneau). For the steel-concrete beams, the expressions of the plastic hinge length for steel-concrete composite beams (Table 2.3) are simply equal to 1.75 times the total height of the composite beam (Chen et al. 2008) or 1.7 times the height of only steel profile (Kemp et al., 2001). The figures 2.3a and 2.3b show the variation of the plastic hinge length, respectively for a beam and a column of the designed frame. As can be seen, the 10 formulations give very different results, except for some ones that overlap: number 6 and 8 for the beam and column, number 3 and 5 for the column.

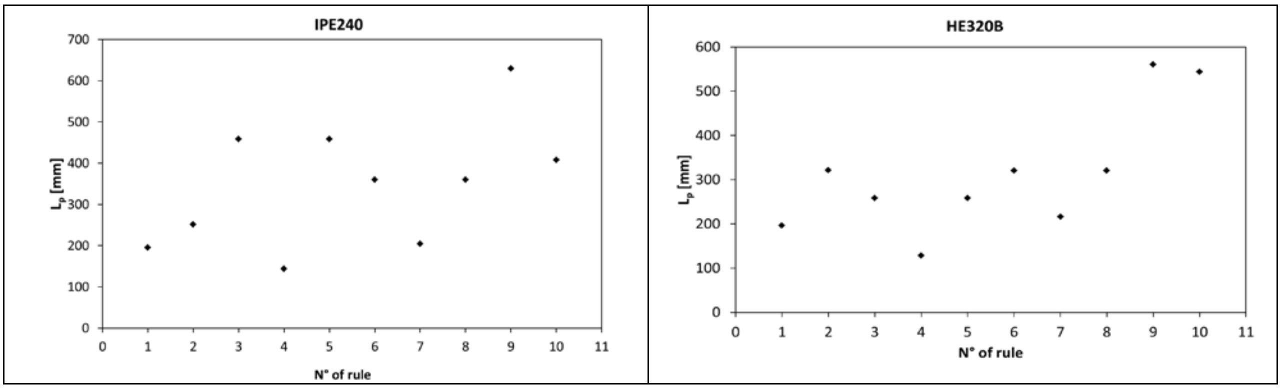


Figure 2.3: Plastic hinge length with the formulations of tables 2.2 and 2.3 for composite beam IPE240 (a) and for composite column HE320B (b).

	Expressions	Parameters	Bibliography
1	$l_p = \frac{d}{2} + 0.2\frac{z}{\sqrt{d}}$	d: height of cross-section z: shear span length	Corley, W. G, 1966
2	$l_p = \frac{d}{2} + 0.05z$	d: height of cross-section z: shear span length	Mattock, A. H., 1967
3	$l_p = 0.08L + 6d_b$	L : element length d_b: long. steel bar diameter	Priestley, M. J. N., and Park, R., 1987
4	$l_p = 0.4h$	h : height of cross-section	Park, R.; Priestley, M. J. N.; Gill,W.D., 1982
5	$l_p = 0.08L + 0.022 d_b f_y$	L: element length d_b: long. steel bar diameter f_y: yielding strenght of bars	Paulay, T., and Priestley, M. J. N., 1992
6	$l_p = 1.0h$	h: height of cross-section	Sheikh, S. A., and Khoury, S. S., 1993
7	$L_p = 0,1L_V + 0,17h + 0,24\frac{d_{bL} f_y}{\sqrt{f_c}}$	L_v: shear span length h : height of cross-section d_{bL}: long. steel bar diameter f_y: yielding strength of bars f_c: concrete cylindrical strength	Circolare NTC #C8A.6

Table 2.2 – Expressions of the plastic hinge length for r.c. elements

In order to evaluate the effect that these relationships (Tab. 5.4 and 5.5) on the non-linear response of the steel-concrete composite frame, a series of non-linear static analyses were performed using the various formulation of the plastic hinge length. In all cases the global mechanisms (hinges at the ends of the beams and the base of the columns) is attained according to the design criteria adopted. Thanks to the extreme regularity of the adopted structure, the variation of the global ductility is directly due to the variation of L_p (Fig. 2.4). In essence, the curves overlap in the elastic field up the formation of the first hinge, than the nonlinear branches diverge; the plastic hinge length that penalizes the overall response more than the others, is the one of formulation 4, that is 0.4 times the height of the section, and the one that give the best result is the one of formulation 9 optimizes equal to 1.75 times the height of the composite cross-section.

Expressions		Parameters	Bibliography
8	L_p=h	h: height of steel cross-section	Bruneau et al., 1998
9	L_p=1,75h_{tot}	h: height of composite cross-section	Chen, S. and Jia, Y., 2007
10	L_p=1,7h	h: height of steel cross-section	Kemp, A. R., Nethercot, D.A., 2000

Table 2.3 – Expressions of the plastic hinge length for steel elements (expression 8) and steel-concrete composite beams subjected to hogging moment (expressions 9 and 10)

This comparison shows how could be not suitable to adapt the formulations of reinforced concrete structure to the case of a composite MRF. In light of this consideration and the results exposed, with the support of the experimental results (Di Sarno et al., 2001; Chen et al., 2007) in the subsequent analyses a plastic hinge length equal to the section height and 1.75 times the section height, respectively for columns and beams was adopted.

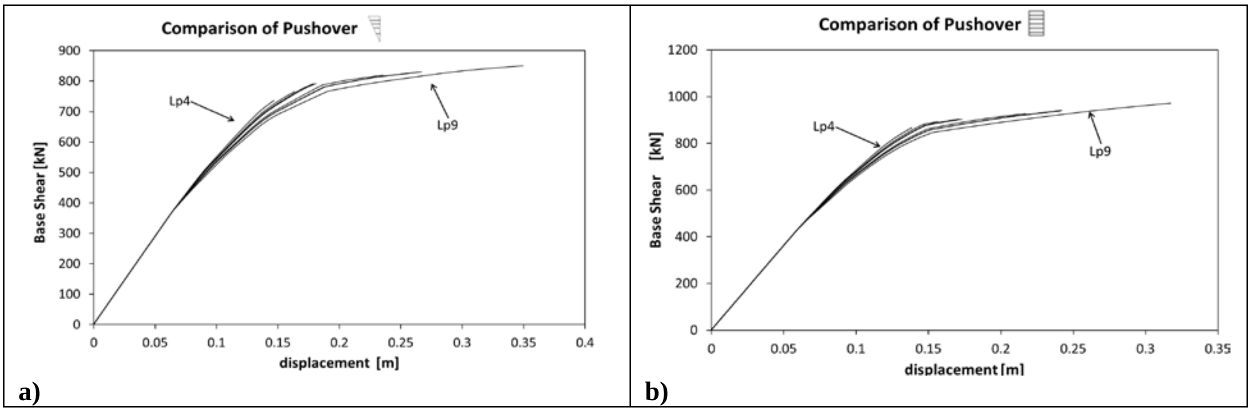


Figure 2.4: Comparison of the global nonlinear response to vary the plastic hinge length: a) modal distribution; b) costant distribution

2.1.3 Comparison of the global nonlinear response considering rigid or deformable joints

In this paragraph are reported the results of non-linear static analyses concerning the designed steel-concrete composite frame, considering or not the joint deformability. In particular, two types of full strength joints were considered: a typical welded joint (Fig.2.5 a) and flanged one (Fig.2.5 b).
Both joints where first modeled using a sophisticated approach based on the component methods (Amadio et al. 2011) where the components of the joint are schematized as non-linear springs. In the obtained macro-model (Fig. 2.6 a) the identified components are: web panel in shear (spring1), column web in tension and/or compression (springs 2), T-stub elements in tension (springs 3), beam-to-slab connection in shear (spring 4), the mechanism 1 and 2 provided by Eurocode 4 and the slub-column interaction (springs 5,6,7,8). Then, the deformability of the connection was introduced into the model through a NLlink (Multi linear plastic) element of the SAP2000, which describes the Moment-rotation curve of the connection itself (Fig. 2.6 b), evaluated by the components method. In the figures 2.7 the adopted Moment-rotation curves are shown. Using the aforementioned moment-rotation laws, two frames were modeled taking into account the deformability of welded or flanged joints. Figures 2.8 shows the comparison between the frame with rigid and with deformable flanged joints, considering both a modal and constant distribution of the seismic forces. It is worth to notice as the overall stiffness of the frame with rigid joints is higher (about 25%) than that of the frame where the deformability of the steel and concrete components of the joints are considered. Furthermore, the joints deformability causes an enhancement of the yielding and ultimate displacement of about 22% for triangular distribution and 25% for that constant one. In particular, at the SLS the maximum interstory drift, in the case of modal

distribution of seismic forces, for the frame with rigid joints results 0.0054 and becomes 0.0085 in the case of deformable joints , overcoming greatly, in the last case, the limits imposed by codes for infill panels rigidly connected. For what concerns the level of base shear, about the same value is reached for the two models. Similar observations can be extended to the case of welded joints when their deformability is considered (Fig. 2.9).

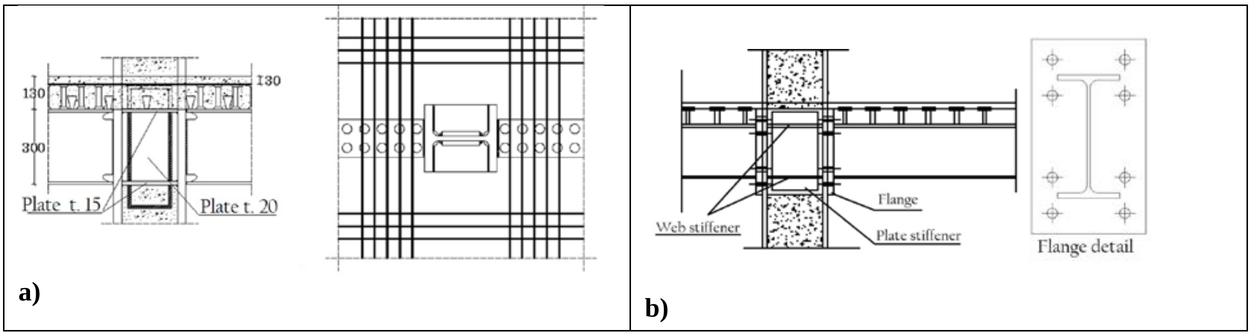


Figure 2.5: The analyzed joints: a) Welded; b) Flanged

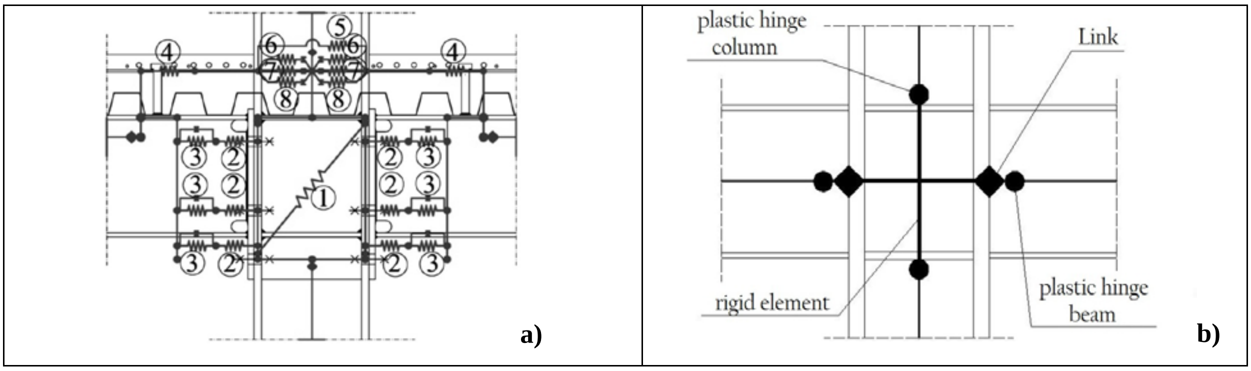


Figure 2.6: The modeling of joints: a) Macro-model; b) Simplified model

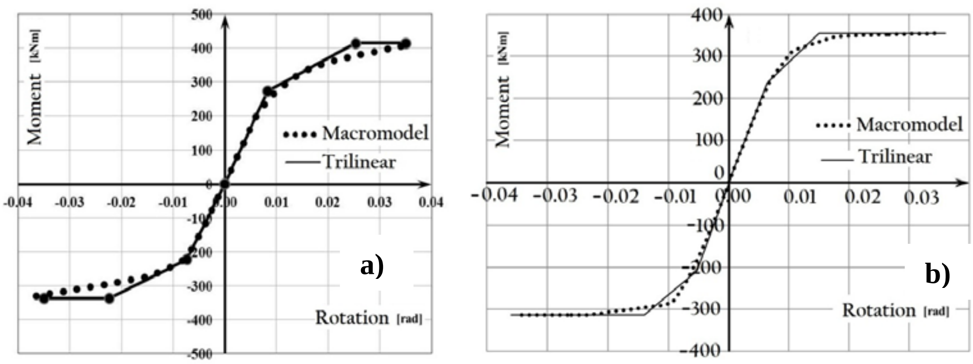


Figure 2.7: Examples of moment-rotation relationships for composite joints: a) Welded joint; b) Flanged joint.

In Tables 2.4 and 2.5, the period of the inelastic equivalent SDOF system (T*) is evaluated according to EC8; further the following parameters of the nonlinear analyses are reported: the displacement (δ_1) and the base shear (V_1) at the formation of the first plastic hinge, the design base shear (V_d), the base shear at the formation of the mechanism (V_y), the base shear corresponding to an elastic system (V_e) , the ductility factor $R_\mu = V_e/V_1$, the over-strength ratio $R_s = V_1/V_y$ and the design over-strength $R_\omega = V_y / V_d$.
All results (Fig. 2.10) show that the deformability of joints reduces the q-factor especially for welded joints; in particular for the modal distribution of the forces, considering the deformability of joints, q-factors lower than the one provided by Eurocode 8 and used for the design were obtained.

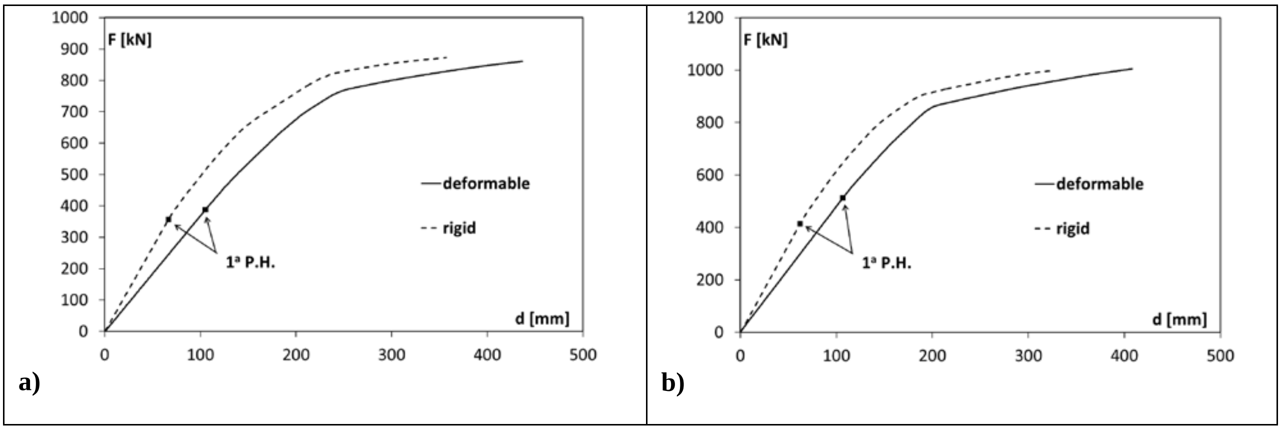


Figure 2.8: Pushover curves for MRF with rigid and deformable flanged joints: a) modal distribution; b) constant distribution of the seismic forces.

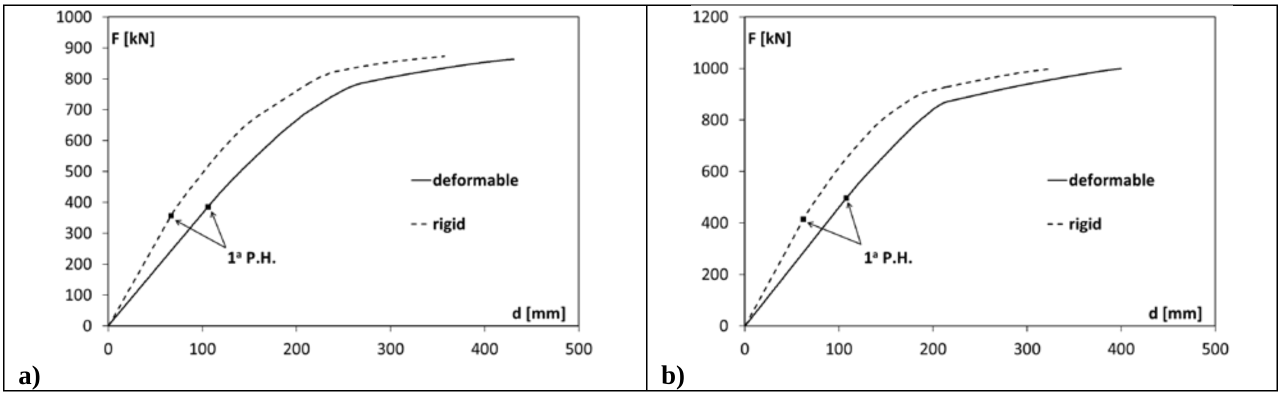
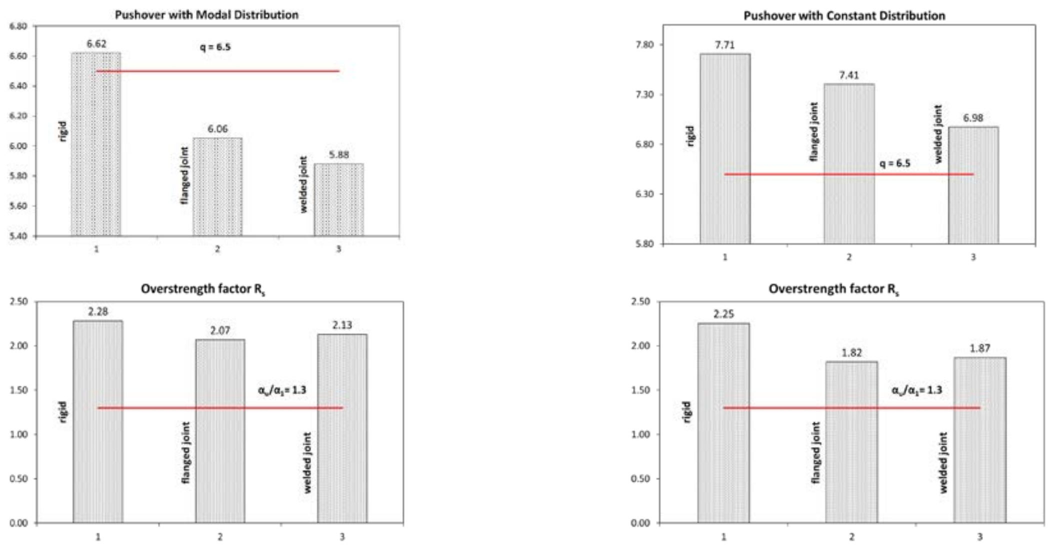


Figure 2.9: Pushover curves for MRF rigid and deformable welded joints : a) modal distribution; b) constant distribution of seismic forces.



a) b)

Figure 2.10 – Variation of the behavior factor q and over-strength factor a) for modal distribution; b) for constant distribution.

On the contrary, when the constant distribution is applied, the code provisions result always safe, even if lower ductility values, as for the modal forces distribution, are obtained when the deformability of joints is considered. Furthermore it is worth noticing that the over-strength factor assumes always a value widely over the standard code indication (α_u/α_1=1.3). Finally, it is evident (Tables 2.4 and 2.5) that the values R_ω of the design over-strength have a significant role in the overall response of structure.

Joint type	Pushover with modal distribution										
	T*	δ_1	δ_u	V_d	V_1	V_y	V_e	R_μ	R_s	R_ω	q
	[s]	[m]	[m]	[kN]	[kN]	[kN]	[kN]	[-]	[-]	[-]	[-]
Rigid	1.39	0.067	0.357	260	357	815	1723	2.11	2.28	1.37	6.62
Deformable flanged joint	1.57	0.105	0.437	260	387	801	1576	1.97	2.07	1.49	6.06
Deformable welded joint	1.59	0.106	0.431	260	377	804	1531	1.90	2.13	1.45	5.88

Tab. 2.4 –Parameters of the nonlinear static analysis with a modal distribution of seismic forces**.**

Joint type	Pushover with costant distribution										
	T*	δ_1	δ_u	V_d	V_1	V_y	V_e	R_μ	R_s	R_ω	q
	[s]	[m]	[m]	[kN]	[kN]	[kN]	[kN]	[-]	[-]	[-]	[-]
Rigid	1.23	0.062	0.326	260	413	923	2006	2.15	2.25	1.59	7.71
Deformable flanged joint	1.37	0.107	0.408	260	512	930	1927	2.07	1.82	1.97	7.41
Deformable welded joint	1.40	0.108	0.400	260	496	928	1815	1.96	1.87	1.91	6.98

Tab. 2.5 – Parameters of the nonlinear static analysis with a constant distribution of seismic forces.

CONCLUSIONS

In the paper the numerical results of a non linear analysis of a steel-concrete composite MRF are presented in order to evaluate the effect on the seismic performance of the rotational capacity of plastic hinges and deformability of beam-column joints. In particular, in the modeled frame, extracted from a building designed according to the rules of Eurocode 4 and 8, a lumped plasticity approach was adopted and the moment-rotation relationship of the plastic hinges was determined by multiplying the moment-curvature relation of the cross-section for the plastic hinge length. The expressions of this latter parameter were selected from those available for the R.C., steel and steel-concrete composite structures. The results obtained using these equations are largely different and have a direct influence on the overall ductility of the structure, highlighting the need of further experimental and numerical data to assess this knowledge for the composite structures. As regards the influence of the joint modeling, it was observed that, though the resistance hierarchy was respected according the capacity design approach, the deformability of the full strength beam-column joint modifies the overall stiffness and the global nonlinear response of the frame. This effect depends on the deformability of web panel, the steel parts in tension or compression, the shear connection between the steel beam and the RC slab and its interaction with the column.
The joints deformability also affects the behavior factor (q) and the over-strength of the structure, although the obtained overall values were not much deviate (about 12% for q and 23% for the over-strength) from those assumed in the design. Thus, the present study confirms that for a correct evaluation of the nonlinear response of a steel-concrete composite framed building is essential to correctly define the deformability of the joints and the plastic rotational capacity of beams and columns, aspects that in the current standard rules are not properly defined for the examined structural system.

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