RISPOSTA SISMICA DI EDIFICI MONOPIANO IN ACCIAIO: RUOLO DEI CRITERI DI PROGETTO

SEISMIC RESPONSE OF SINGLE-STOREY STEEL BUILDINGS: ROLE OF DESIGN CRITERIA

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ABSTRACT

Italian and European codes allow constructions in seismic areas to be designed with either dissipative or elastic structural behaviour. In the first case, the concept of capacity design is the basis of structural dimensioning; both strength and ductility verifications are required. In the second case, structural elements are designed to remain in the elastic field under the assigned design seismic input; ductility verifications are not enforced. These two design approaches might lead to very different seismic performances depending on the role of the non-ductile elements and connections in the elastic design. In fact, the non-ductile elements and connections might represent a source of weakness and lead to premature failures. In the present work, critical issues related with the modelling of the post-elastic behaviour of a non-dissipative single-storey industrial steel structure are discussed and comparisons are made with the same structure designed with dissipative structural behaviour.

SOMMARIO

La normativa italiana e gli Eurocodici consentono di progettare una costruzione soggetta ad azione sismica prevedendo per essa un comportamento strutturale di tipo dissipativo oppure elastico. Nel primo caso, la progettazione in gerarchia delle resistenze costituisce la base del dimensionamento strutturale e questo richiede verifiche di resistenza e duttilità. Nel secondo caso, gli elementi strutturali sono progettati per rimanere in campo elastico sotto l'azione sismica di progetto. Nelle strutture in acciaio, questi due approcci progettuali possono condurre a prestazioni sismiche molto differenti. Infatti, gli elementi non duttili e i collegamenti possono costituire gli anelli deboli che inducono collassi prematuri. Nel presente lavoro si discutono le criticità legate alla modellazione del

comportamento post-elastico di un edificio industriale mono-piano in acciaio non-dissipativo, in confronto con la stessa struttura progettata prevedendo un comportamento strutturale dissipativo.

1 INTRODUCTION

Two different strategies for the seismic design of steel structures are possible in the Italian and European codes. The first way requires capacity design and dissipative structural behaviour, where the demand resulting from the seismic action is calculated considering the dissipative capacity and structural damage is accepted in case of design-level seismic accelerations. The second way is based on the adoption of an elastic behaviour of the building: all the members and connections are required to remain in the elastic field for the given design seismic input.
In the present study, an industrial single-storey steel structure (Fig. 1) located in Milan (Italy) is used as benchmark problem, taken from a recent research project (ReLUIS-RINTC 2019-2021) [1]-[2]. Two designs were carried out following both the elastic approach as well as the dissipative approach. Afterwards, the two structural projects are compared in terms of performance under increasing lateral loads.

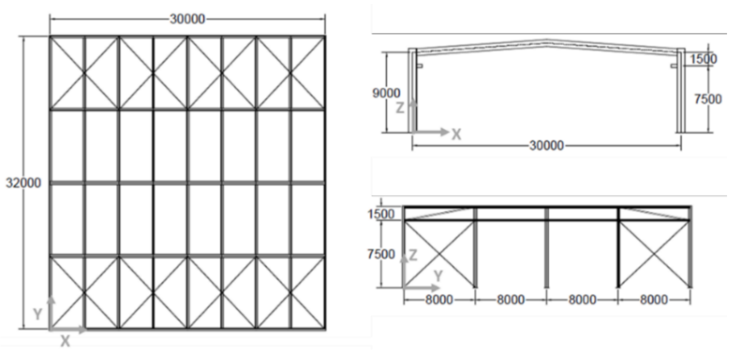

Fig. 1. Global geometry of the case study.

2 CASE STUDY

2.1 Structural system and main dimensions

In the transverse direction, five parallel duo-pitch portal frames comprise the main structure. The portal frames are interconnected perpendicularly by simply supported beams at the apex, at the eaves, and at the crane-supporting bracket level. In the longitudinal direction, concentrically braced frames provide stability and resistance to horizontal actions (Fig. 1). The structural continuity of the portals at the apex (beam-to-beam connection) and eaves (beam-to-column connection) is ensured by bolted end plates. The roof is made on a single level, where the longitudinal beams, purlins, roof bracings, and non-structural elements are located. The columns are nominally hinged at their basis. The design of members and connections was made according to relevant sections of Eurocode 3 [3]-[4] and the Italian Code (NTC 2018), choosing an elastic behaviour.
Hot-rolled H sections were selected for columns, beams, and purlins, while square hollow sections were chosen for vertical and roof braces. Steel grade is S 275 for all structural elements. Table 1 summarises the adopted sections for the main elements. The case study is supposed to be equipped with an overhead crane. Masses and loads were calculated considering the self-weight of the crane, the weight of the crane-supporting beams, and the crane load.

Table 1. Elastic case study sections.

Column	Rafter	X-braced	Single-braced	Long. beam	Purlins	Roof bracing
HE900A	HE800A	RHS-CF 60x60x4	RHS-CF 115x115x4	HE220A	HE240A HE200A	RHS-CF 60x60x4

2.2 Connections

Brace connections (Fig. 2) comprise a plate welded to the tubular profile and bolted to a gusset plate, which is in turn welded to the beam and to the column (Fig. 2(a) and Fig. 2(c)). The mid-span connection of cross-bracings is entirely welded (Fig. 2(b)). Roof bracings converge centrally on the purlin by means of bolted connections to a gusset plate welded to the purlin itself. The same criteria were used for the connection of the single brace.

Beam-to-column joints are conceived as bolted end plate connections, as shown in Fig. 3(a). The connection involved ten pairs of M24 bolts, three of which were located at the level of the haunch below the beam. The section composed of beam and haunch reaches a height of 1244 mm at the point of connection to the end plate; connection to the end plate is achieved by means of full-strength penetration welds. The end plate and the transverse stiffeners of the column are 30 mm thick. The web of the haunch is 15 mm thick.

To ensure a pin behaviour of the base connection, the column section is tapered by means of 30 mm thick plates placed in continuity with the column flanges. The ground connection is made by two M39 diameter anchor bolts. Fig. 3(b) shows the main details of the connection.

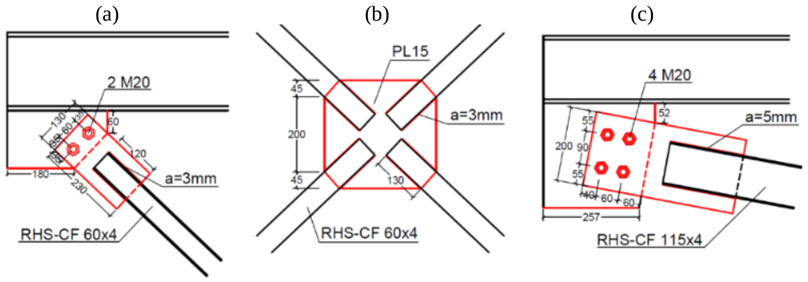

Fig. 2. End (a) and central (b) connections of the X-bracing system; brace connections for the single-diagonal bracing system(c).

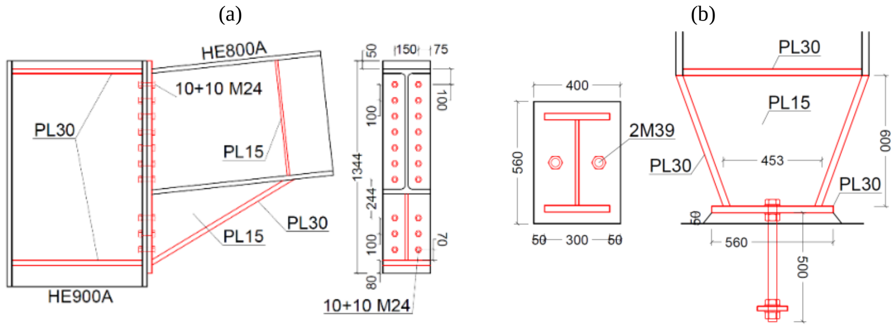

Fig. 3. (a) Beam-to-column joint and (b) column base connection.

3 NONLINEAR MODELLING

The open-source finite element (FE) software OpenSees [5]-[6] was used for the seismic performance assessment of the considered case study. Material nonlinearities were included using both distributed and lumped plasticity. Geometric nonlinearities were modelled through the large displacements and small strains approach. The SteelBRB constitutive law [7] was used to model the steel stress-strain relationship. The model developed in OpenSees includes the envelope panels, both cladding and roofing panels, considering their inelastic response. An overview on modelling aspects is presented in the following. More details are provided in [8].

3.1 Model for structural elements and connections

Columns, beams, and braces were modelled as force-based nonlinear beam-column elements. The sections used in the model are HE and RHS, generated as an assembly of rectangular sub-sections. The torsional stiffness (J_tG_s) was introduced by adding this contribution (in series) to that of the fibre section, by means of the OpenSees section aggregator.
Three aspects require special consideration for modelling the bracing system: (i) buckling of the brace members, (ii) flexibility, resistance, and stability of the gusset plate, and (iii) response of the bolted shear connections. The approach proposed by Hsiao et al. [10,11] was followed to simulate the nonlinear out-of-plane behaviour of the gusset plate connection by means of rotational nonlinear spring at the physical end of the brace. The bolted connection was represented by means of a translational spring with resistance and stiffness calculated according to Eurocode 3 [5] placed in series with the rotational one.
The beam-column moment-resisting connection was modelled using IdeaSTATICA [12]. A capacity verification was carried out for both positive and negative bending moments. For a positive bending moment, failure was reached by yielding of the lower flange of the beam. In the case of a negative bending moment, failure started because of tensile failure of the first (upper) bolt row. Following this event, the connection still offers some moment resistance, despite a loss of stiffness. The capacity of the connection is then investigated by progressively removing the bolt rows until the remaining bolt rows are insufficient to resist the shear force, which is taken as the ultimate failure of the connection. The identified moment-rotation relationship (Fig. 4) is then assigned to the rotational spring representing the beam-column joint in the model.

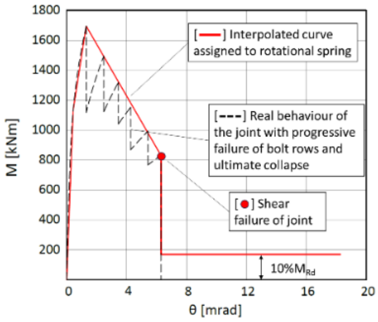

Fig. 4. Moment vs. rotation for beam-column joints.

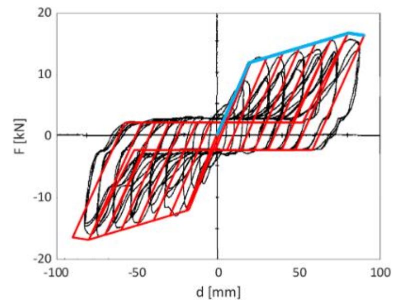

Fig. 5. Cladding panels: experimental vs numerical cyclic response.

3.2 Model for non-structural elements

The developed model incorporates the nonlinear behaviour of the cladding and roofing panels. The sandwich panel used in the considered case study corresponds to type A tested by De Matteis and

Landolfo [13]. Couples of truss elements calibrated on the experimental tests described in [13]-[14] were adopted to reproduce the nonlinear dynamic contribution of groups of assembled panels. The calibrated trilinear curve is shown in Fig. 5.

4 FINITE ELEMENT ANALYSIS

Pushover analysis was performed to evaluate the development of resisting mechanisms activated by horizontal loads, considering: a modal force (MF) distribution and a uniform force (UF) distribution. The global failure criteria were chosen according to the lateral-force resisting mechanisms: (i) in the transverse direction (X), the model of the moment resisting frames focused on the global nonlinear response, including the local brittle behaviour of beam-column joints; (ii) in the longitudinal direction (Y), the model of the concentrically braced frames focused on the nonlinear behaviour of braces and the brittle behaviour of the bolted connections.

4.1 Nonlinear static analysis: transverse direction

Fig. 6a shows the capacity curves resulting from the pushover analyses in transverse direction for both the model with sandwich panels (SP) and the bare frame model (BF). The brittle behaviour is due to failure of the beam-column joints belonging to the central portal frame because of negative bending moments. The other structural elements remain elastic. The central portal frame represents the weak sub-system for two reasons: (i) it is connected to the adjacent portals only by the longitudinal beams (and purlins); (ii) it has a mass concentration due to the position of the crane (in this case applied to the central portal frame), resulting in a distribution of modal forces (MF) acting with larger intensity on this subsystem. These reasons are more pronounced in case there are no roofing panels, which provide a better redistribution of forces and a stiffer structure with respect to transversal actions. In the case of uniformly distributed forces (UF), the difference between SP and BF is reduced. The stiffness of the bare frame model subjected to the UF distribution is much larger than it is for the MF distribution, because the MF distribution considers the additional crane mass applied to the central frame whilst the UF ignores this effect.

4.2 Nonlinear static analysis: longitudinal direction

Fig. 6b shows the capacity curves resulting from the pushover analyses in the longitudinal direction for both the model with sandwich panels (SP) and the bare frame model (BF). The structure shows a brittle behaviour due to the simultaneous failure of the bolted connections at the brace bottom ends because of the exceedance of the bolt shear resistance. Along the increasing branch of the capacity curve a stiffness variation due to buckling of the cross bracings is observed. Buckling of the braces with a single diagonal arrangement is not achieved. Considering the case of a modal force distribution (MF), the initial elastic response branches show negligible differences between the structure with panels and the bare frame. The two response curves separate each from the other due to the X-bracing buckling. This buckling occurs for larger base shear forces in the model with panels, because of the resistance contribution offered by the cladding panels. Sandwich panels provide a significant contribution to the overall strength of the building as shown by the difference of the peak forces. The use of a uniform force distribution (UF) shows the higher contribution offered by the lateral cladding panels compared to the effect of the roofing panels, in terms of stiffness, resulting in a roof system not properly acting as a rigid diaphragm.

5 DISSIPATIVE VS. ELASTIC DESIGN

The same structure was designed following the concept of capacity design as illustrated in [15]-[16]. The cross sections adopted for the structural elements were different in some cases and the connections were designed to possess sufficient resistance with respect to more ductile collapse mechanisms.

(a) (b)

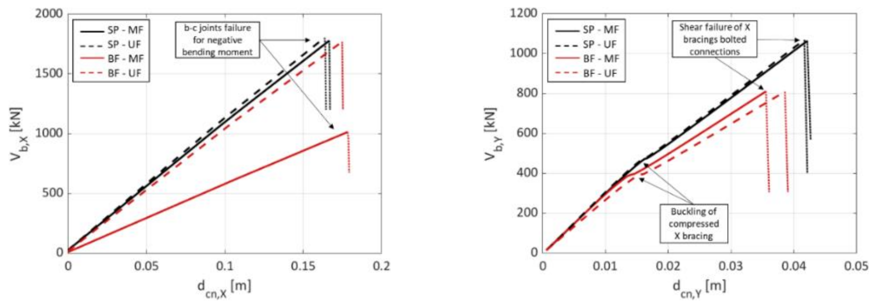

Fig. 6. Pushover results for the "non-dissipative" structure: (a) transverse (X) direction; (b) longitudinal (Y-) directions.

Table 2 summarises the sections adopted in the dissipative design. The modelling assumptions are the same for beams, columns, and bracings while the brittle components, i.e., beam-column joints and braces connections, were removed from the model and, therefore, do not constitute a failure criterion. This leads to a different definition of the collapse criteria. In the transverse direction (X), the global collapse condition is identified by means of a limit value of the storey drift evaluated at the top section of the column. This limit is assumed equal to 0.10, according to FEMA-350 [17]. In the longitudinal direction (Y), a limit is assigned to the maximum strain in the braces according to Hsiao et al. [18].

Table 2. Dissipative case study sections.

Column	Rafter	X-braced	Single-braced	Long. beam	Purlins	Roof bracing
HE900M	HE800A	RHS-CF 90x90x2.6	RHS-CF 120x120x3	HE300B	HE220A	L50x4

5.1 Nonlinear static analysis

With reference to the MF distribution (the UF distribution results are not reported in this paper), Fig. 7 and Fig. 8 show the results of the pushover analysis in the transverse and longitudinal directions, respectively, for both the dissipative and non-dissipative structures. In the transverse (X) direction, the capacity curve is characterized by two almost linear branches, with a smooth transition from the initial to the final linear branch. During the smooth transition, the stiffness variation is due to the progressive yielding of the beam. The collapse condition is reached for a displacement of 0.9 m at the top section of the column. In the longitudinal (Y) direction, before global failure is reached, the following is the sequence of nonlinear events: X-brace buckling and yielding, plastic deformations in the cladding system, column yielding due to longitudinal bending moments, and eventually shear failure of the cladding panels. The contribution of the cladding panels is more evident in the longitudinal direction compared to the transverse direction, especially after yielding of the X-bracing, which causes an important drop in the stiffness of the main structure.

5.3 Comparison of results

In the transverse (X) direction, Fig. 8 clearly shows the different displacement capacity of the two models. The displacement capacity of the elastic structure is much smaller than that of the dissipative one. The different cross sections of members in the two cases results in the initial stiffness of the elastic design being lower than that of the dissipative one. The same difference can be seen in terms of base shear resistance. The contribution of sandwich panels partially reduces the difference in the responses of the two structures. In the longitudinal (Y) direction, similar considerations can be made, except for the initial stiffness, which is always greater for the elastic-design. In the light of these results, it is evident that the dissipative structure has more resources than the structure designed with the elastic approach, resulting in a larger resistance.

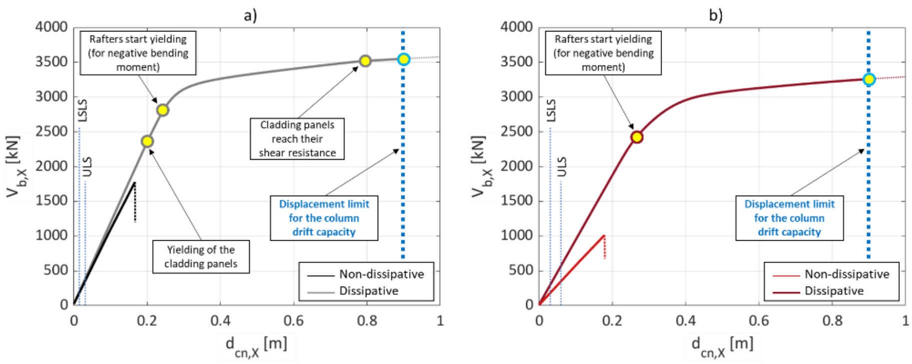

Fig. 7. Pushover results for the transverse direction: (a) model with panels; (b) bare frame.

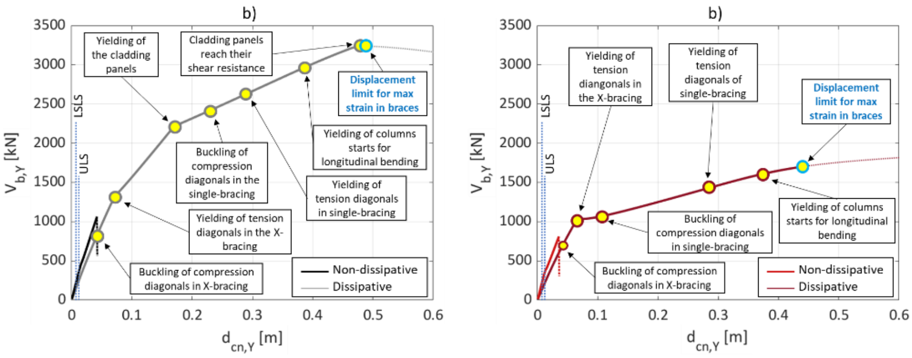

Fig. 8. Pushover results for the longitudinal direction: (a) model with panels; (b) bare frame.

CONCLUSIONS

In the present study, attention was given to modelling moment-resisting frames as well as concentrically braced frames designed without avoiding the possible occurrence of brittle failure mechanisms under lateral loads increased beyond their design values. Results obtained from nonlinear static (pushover) analyses show a global brittle behaviour of the steel structure, as opposed to the large ductility that can be exploited if a dissipative seismic design is considered. Although results are limited to a specific case study, i.e., one-storey non-residential steel building designed for low-seismicity, it is indeed important to pay attention in future studies to the consequences that a non-dissipative design in seismic areas could have in terms of resilience of the constructed facilities.

ACKNOWLEDGMENT

This work was developed within the research project ReLUIS-RINTC 2019-2021 funded by the Italian Department of Civil Protection whose support is gratefully acknowledged.

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KEYWORDS

Nonlinear finite element models, Seismic analysis, Steel structures.