FRAGILITA' SISMICA DI CAPANNONI INDUSTRIALI ESISTENTI NEI RIGUARDI DELL'ESERCIZIO

SERVICE-LEVEL SEISMIC FRAGILITY OF EXISTING NON-RESIDENTIAL SINGLE-STOREY BUILDINGS

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ABSTRACT

The paper presents seismic fragility curves for non-residential single-story older steel buildings at a performance level defined in the paper as "usability-preventing damage". The examined buildings have a structure essentially made of: (i) trussed portal frames for the transverse direction and (ii) concentrically braced frames for the longitudinal direction. The buildings were designed according to the codes and standards of practice available in the years 1980-1990s in Italy. 3D non-linear finite element models were developed using OpenSees, either including or excluding the lateral-load response of cladding and roofing panels. Two alternative cladding systems were considered: (i) sandwich panels and (ii) single trapezoidal sheeting. Non-linear dynamic analyses were carried out and parametric fragility curves were consequently obtained, thus describing the probability to exceed the usability-preventing damage level.

SOMMARIO

Il lavoro presenta uno studio teorico-numerico sulle curve di fragilità sismica di capannoni esistenti, con riferimento a uno stato di danneggiamento tale da limitare la funzionalità ordinaria dell'edificio. Gli edifici considerati sono caratterizzati da due diversi sistemi sismo-resistenti nelle due direzioni principali in pianta: (i) portali con travatura reticolare nella direzione trasversale e (ii) controventi concentrici nella direzione longitudinale. I casi studio sono stati ottenuti simulando la progettazione in accordo alle normative e linee guida disponibili negli anni 1980 e 1990 in Italia. Le strutture sono state modellate in 3D utilizzando OpenSees e i modelli sono stati sviluppati con e senza il contributo strutturale dei pannelli di chiusura laterale e di copertura. Sono stati considerati due tipi di pannelli di chiusura laterale: (i) pannelli "sandwich"; (ii) lamiere grecate singole. L'articolo evidenzia i principali risultati ottenuti da numerose analisi dinamiche e fornisce le conseguenti curve di fragilità.

1 INTRODUCTION

Older buildings can suffer severe damage due to earthquake actions, thus originating considerable economic losses, and eventually collapse [1, 2]. In non-residential single-story steel structures, the type of non-structural elements, the structural scheme and the design assumptions can largely affect the building seismic performance [2, 3]. In this context, the paper describes recent developments obtained within the context of a more general research project [2, 4]. The paper describes the case studies, summarizing the design assumptions according to the codes used in Italy in the decade 1980s – 1990s [5, 6, 7]. Second, the main modelling aspects are presented, including issues concerning the building envelope panels. The work focuses on results of empirical fragility curves characterizing the exceedance of a damage state which could limit the building usual functionality.

2 BUILDING ARCHETYPES

2.1 General description

Fig. 1 illustrates transverse, longitudinal and plan views of the considered case study buildings. The archetypes were made of portal frames with a main truss system in the transverse (X-) direction, and concentric braces in the longitudinal (Y-) direction. Roof braces were used to stabilize the main truss upper chords and to connect the two longitudinal braced bays. Fig. 1 also shows schematically the building envelope, by sketching both the cladding and roofing. The buildings were assumed to be located at three Italian cities: L’Aquila, Napoli, and Milano, which are characterized relatively by high, medium and low seismic hazard respectively.

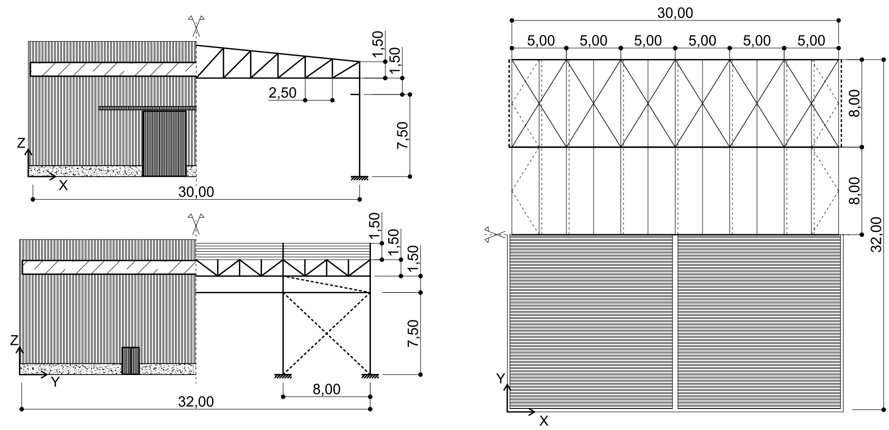

Fig. 1. Case study overall geometry

Fig. 2a shows the two considered structural schemes for the building transverse direction: (i) a portal frame with pinned column base connections (PCBs) and a column running continuous for the whole height of the building (H = 10.50 m); (ii) a portal frame with semi-continuous column base connections (SCBs) [8] and a column interrupted at the height of 9.00 m.

Additionally, as shown in Fig. 2b, two types of cross sections and connections were considered for the longitudinal concentric bracing system: (i) square hollow section (SHS) braces with welded gusset plate connections; (ii) closely spaced built-up angle (2L) section braces with bolted gusset plate connections. Two types of envelope panels were considered in both the design and analysis process: (i) sandwich panels (SP); (ii) single trapezoidal sheeting (TS) (Section 3.4).

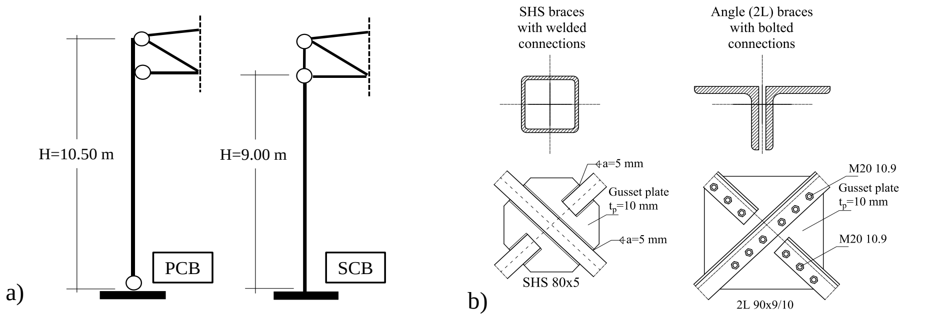

Fig. 2. Structural schemes and design assumptions

2.2 Main design aspects

The design was carried out according to the codes and standard of practice that were used in Italy during the 1980s-1990s. The design of the main truss members and connections was dictated by internal force demands due to gravity loads. Connection details varied with the main structure scheme, i.e., different truss-to-column connections characterize the PCB or SCB structural schemes. Columns and column base connections were designed considering the internal force demand for wind and seismic actions. However, wind loads were always dominant actions. The brace cross section selection was governed by a global slenderness limitation for compressed members ($\lambda < 200$). On the contrary, the brace end connection design was governed by wind loads. No capacity design rules were enforced to reproduce the common design practice in the 1980s-1990s.

3 NUMERICAL MODELLING

3.1 General aspects

The structures were modelled and analyzed using *OpenSees* [9]. The *Steel02* uniaxial material model was adopted, with the following parameters: (i) Young's modulus E_s = 210 *GPa*; (ii) Poisson ratio ν = 0.30; (iii) yielding strength f_y = 316 *MPa;* (iv) post-elastic kinematic hardening ratio E_p = 0.01 E_s. Additionally, ultimate strength f_u = 479 *MPa* [10] and ultimate strain ε_u = 34% [10] were used checking ultimate failure of anchors [2]. Columns were modelled as elastic beam-column elements considering the P-Delta formulation to simulate geometrical non-linearity. In fact, neither yielding nor buckling was predicted for the columns.

3.2 Braces and relevant connections

Imperfect brace geometries were modelled to simulate brace buckling and post-buckling response. SHS brace sections were dominated by low-cycle fatigue failure. Therefore, a fracture material model [11] was implemented to consider brace fracture. An example of the implemented axial force (N) vs. axial deformation (δ) relationship is shown in Fig. 3a. On the contrary, failure of connections dominated the 2L brace axial behavior, because of bolt bearing failure. Therefore, a non-linear shear force-displacement relationship was implemented in the model for the brace end connections, as described in [2].

3.3 Truss connections

The non-linear response of truss-to-column connections strongly affected the PCB structural response in the transverse direction. Therefore, an explicit representation of such connection failure was implemented in the numerical models. On the contrary, in the SCB cases, yielding of the column base connections characterized the non-linear response of the structure. The bending moment vs. rotation response of the column base connections was explicitly modelled following the method proposed by Della Corte et al. [12], with additional information provided in [13]. Fig. 3b shows an example of a column base connection response.

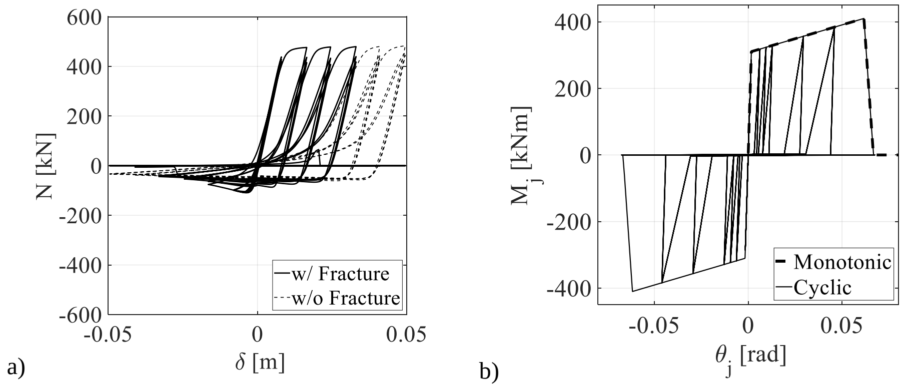

Fig. 3. (a) SHS brace response; (b) SCB column base connection response.

3.4 Envelope panels

The envelope panel response was modelled using available experimental test results for sandwich panels [14] and trapezoidal sheeting [15]. In such results, the non-linear responses of cladding-to-frame connections were included. Therefore, a non-linear response model was implemented by means of equivalent truss elements, for both the cladding and roofing panels, calibrated based on experimental results (Fig.5c and Fig.5d, respectively for SP and TS). It is worth nothing that a quite large difference is exhibited in terms of force-displacement response by the two cladding types considered in this study. For instance, the SP cladding system shows relatively large resistance and ductility when compared with the TS. This difference is due to the cladding-to-frame connections, which were bolted connections for the SPs and screwed connections for the TS. Discretization of the panel sub-assembly was made to realistically represent the transfer of forces from the main structure to the envelope system and vice-versa. For the same reason, secondary columns and siderail elements were explicitly modelled, as well as the non-linear behavior of siderail-to-column connections, with a component response following analogous rules to those described for the PCB truss-to-column connections.

4 EMPIRICAL FRAGILITY CURVES FOR THE BUILDING ENVELOPE

4.1 Main assumptions and analysis methodology

A performance level termed “usability preventing damage” (UPD) was introduced in [2]. The level is intended to indicate a spread of damage which makes the building unusable in the aftermath of an earthquake. Once the performance level was clearly identified, then fragility curves were built using results of non-linear dynamic analyses [2]. The analysis procedure for calculating the UPD fragility curves is analogous to that described in [16, 17] for the collapse performance level. Ten ground-shaking intensity levels were considered, by varying the earthquake return period of the seismic action from 10 to 10^5 years.

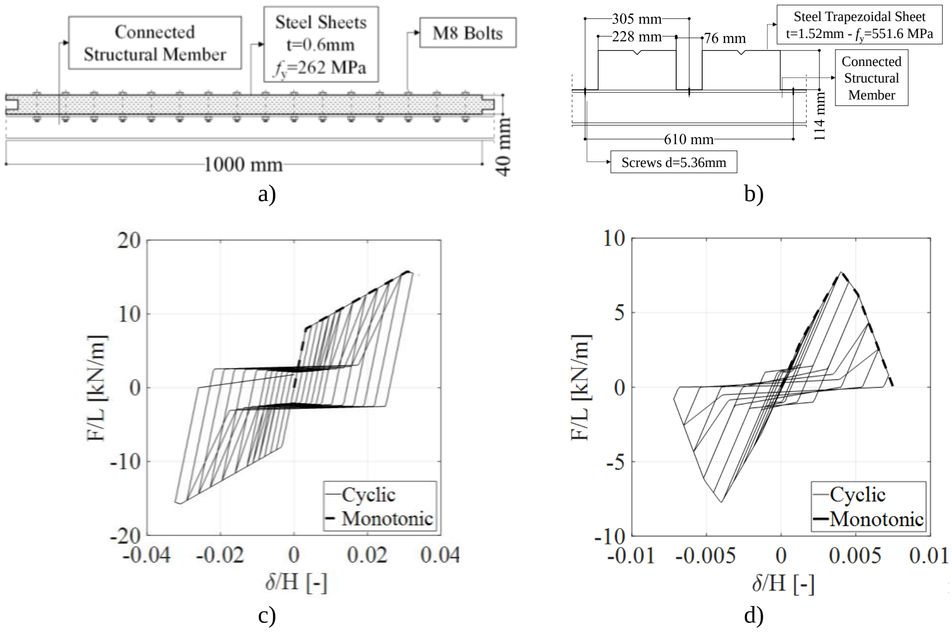

Fig. 4. Envelope panels geometry and component response: (a, c) SP; (b, d) TS.

4.2 Results

Results are summarized here in terms of frequency of exceeding the UPD level per each earthquake return period (T_R). Three different plots are provided for the frequency of exceeding the UPD: (i) frequency for the X-direction response (empty triangles); (ii) frequency for the Y-direction response (empty squares); (iii) total frequency, i.e., the frequency calculated considering the exceedance of the UPD either for the X- or Y-direction response (empty circles). Fig. 6 refers to the PCB cases, showing the effects of variations in the type of brace cross section, the type of cladding and the building site. In any of the considered cases, the frequency of exceeding the UPD performance level for the X-direction response is larger than it is for the Y-direction response. Therefore, the transverse (i.e., portal frame) direction response governed the UPD fragility. Also, comparing the results for the SHS braces with those for the 2L braces (i.e., comparing the first row with the second row of plots in Fig. 6), one can clearly observe that there are no significant differences between the two bracing systems. This is essentially the result of the UPD fragility being governed by failures occurring due to the transverse (portal frame) response. On the contrary, the UPD performance level was largely governed by the type of cladding. In fact, comparing the first column with the second column of plots in Fig. 6, it is apparent that the UPD performance level was exceeded more frequently with a TS cladding than it was with a SP cladding, for any given earthquake return period. In the case of TS cladding, UPD failures started to occur at T_R= 50 *yrs*, whilst in the case of SP cladding UPD failures started to occur at T_R = 2500 *yrs*. Also, empirical fragility curves for the TS cladding appears to have less dispersion than the those for the SP cladding. The building site also had large effects on the computed fragility. In fact, one can observe a strong reduction of the UPD fragility from L'Aquila to Milano, with an intermediate response for Napoli. Especially, it is noted that there were no UPD failures for buildings at Milano, even at the largest considered earthquake intensity.

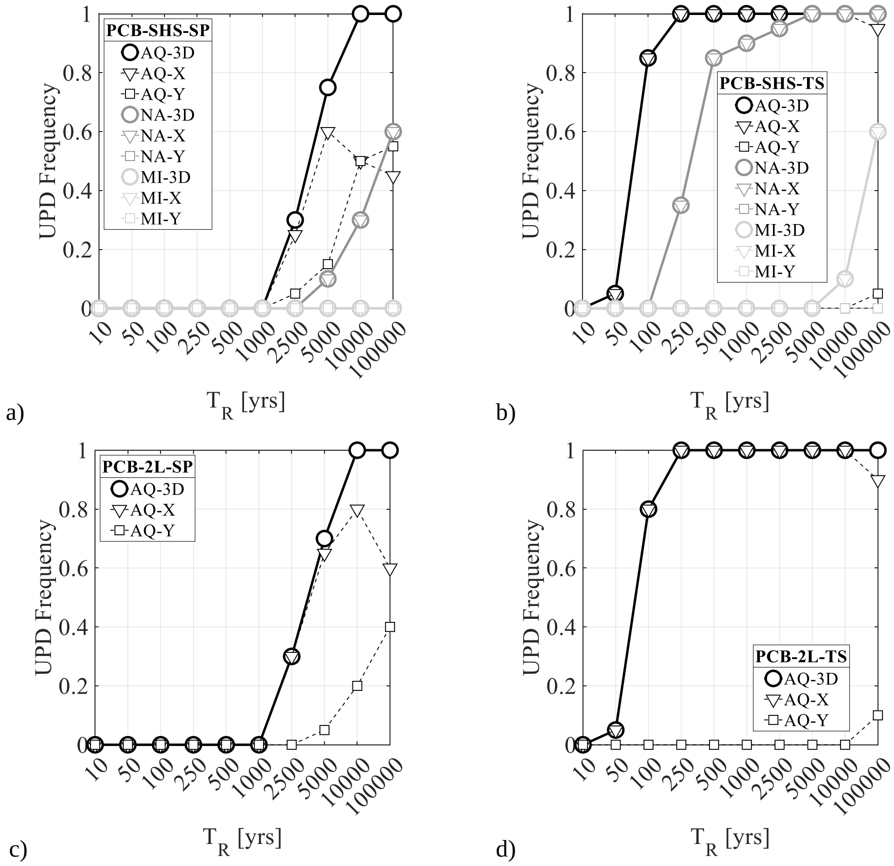

Fig. 5. Frequency of UPD for the PCB models: (a) SHS-SP; (b) SHS-TS; (c) 2L-SP; (d) 2L-TS.

Similar results were obtained for the SCB cases. The corresponding fragility curves are summarized in Fig. 7, again for the various considered brace cross sections, cladding types and building sites. The TS cladding always shows the worst performance in terms of UPD exceedance, where failures started to occur at T_R = 250 *yrs*. On the contrary, the SP cladding experienced failures starting from T_R = 2500 *yrs*, as already shown for the PCB structures. A relatively larger effect of the brace cross section can be noted in these cases for the SP cladding. This occurs because of failures of the 2L-section brace connections, triggering the UPD level for a smaller longitudinal drift [2].

The effect of the type of column base connections (PCB vs. SCB connections) can be observed by comparing results in Fig. 6 with those in Fig. 7. For any of the considered cases, the PCB structures experienced more UPD-level exceedances compared to the SCB structures, as a direct consequence of the corresponding reduction of the capacity [2].

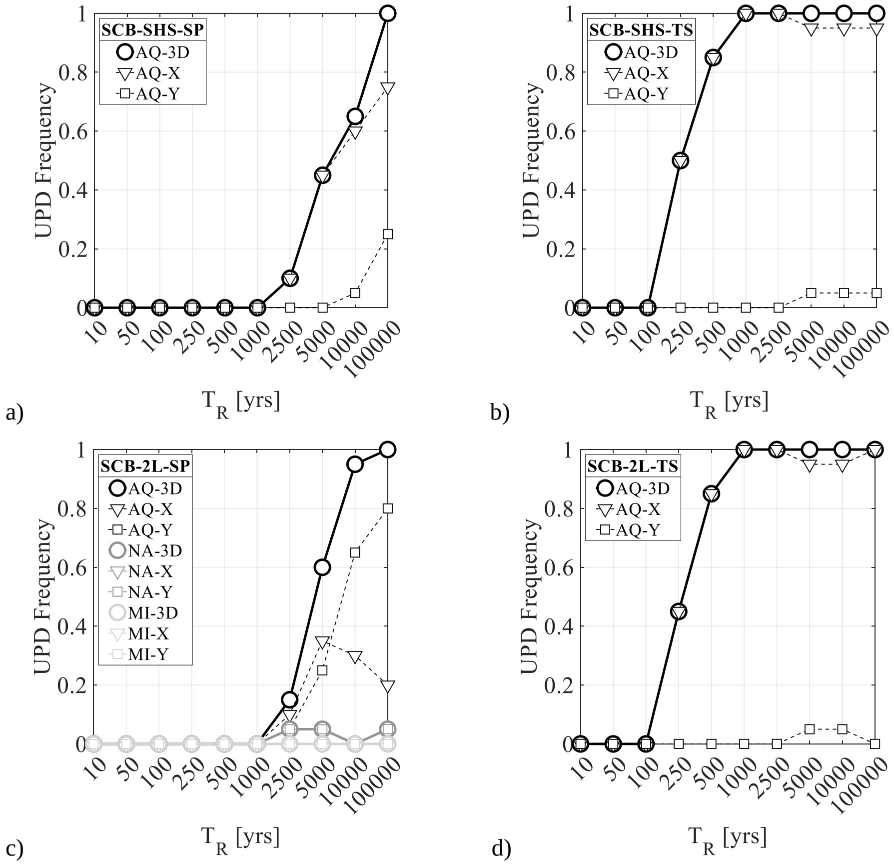

Fig. 6. Frequency of UPD for the SCB models: (a) SHS-SP; (b) SHS-TS; (c) 2L-SP; (d) 2L-TS.

CONCLUSIONS

The numerical results presented in this paper show that the type of cladding panels significantly affects a “usability-preventing damage” (UPD) level, with much larger vulnerability observed for screwed trapezoidal sheeting compared with bolted sandwich panels. In fact, the UPD started to be exceeded for an earthquake return period T_R = 50 years in the case of screwed trapezoidal sheeting and T_R = 2500 years in the case of bolted sandwich panels. This was the result of the large differences in the resistance and ductility of the two considered cladding systems. Considering this large variability of response, characterizing the UPD for the cladding system should be the subject of further research efforts.

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KEYWORDS

Building; Cladding; Connection; Modelling; Earthquake.