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Vulnerability assessment and retrofit solutions of precast industrial structures

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Abstract. The seismic sequence which hit the Northern Italian territory in 2012 produced extensive damage to reinforced concrete (RC) precast buildings typically adopted as industrial facilities. The considered damaged buildings are constituted by one-storey precast structures with RC columns connected to the ground by means of isolated socket foundations. The roof structural layout is composed of pre-stressed RC beams supporting pre-stressed RC floor elements, both designed as simply supported beams. The observed damage pattern, already highlighted in previous earthquakes, is mainly related to insufficient connection strength and ductility or to the absence of mechanical devices, being the connections designed neglecting seismic loads or neglecting displacement and rotation compatibility between adjacent elements.

Following the vulnerabilities emerged in past seismic events, the paper investigates the seismic performance of industrial facilities typical of the Italian territory. The European building code seismic assessment methodologies are presented and discussed, as well as the retrofit interventions required to achieve an appropriate level of seismic capacity. The assessment procedure and retrofit solutions are applied to a selected case study.

Keywords: precast structures; seismic assessment; seismic retrofit; industrial buildings

1. Introduction

In the last few years two relatively strong seismic events, L'Aquila 2009 Mw 6.3 (Chiarabba *et al*. 2009) and Emilia 2012 Mw 6.11 (Lauciani *et al*. 2012), struck the Italian territory showing the high seismic vulnerability and inadequacy of existing precast industrial buildings (Toniolo and Colombo 2012, Liberatore *et al*. 2013, Magliulo *et al*. 2013, Belleri *et al*. 2014). In the Emilia seismic sequence the epicentre of the main shock was located near a large industrialized area and several industrial facilities were severely damaged: local and global collapses caused both fatalities and production downtime.

Poor structural elements detailing, lack or under-designed mechanical connections, absence of a diaphragm action, unexpected interaction between adjacent structural or non-structural elements,

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such as cladding panels, are worldwide recognized as the major causes of poor seismic performance of conventional RC precast buildings (Iverson and Hawkins 1994, Muguruma *et al*. 1995, Arslan *et al*. 2006, Senel and Palanci 2013, Brunesi *et al*. 2014). The main vulnerabilities observed in precast industrial buildings during Emilia earthquakes (Belleri *et al*. 2014) are related to inadequate horizontal load transfer mechanisms between precast members. Other damage scenarios observed are the overturning of masonry infills, short-column failures due to ribbon glazing and discontinuous cladding panels, out-of-plane failure of double-pitched beams, column's RC forks failure and column loss of verticality.

The most severe type of damage (Belleri *et al*. 2014) is the loss of support and consequent fall of both structural and non-structural elements: the former is related to beam-to-column and roof-to-beam connections relying solely on friction, being the use of mechanical devices as connections between precast members mandatory in Italy since 1987 only in seismic zones (D.M. 3/12/1987), whereas the Emilia region has been classified as seismic prone only in 2003 (O.P.C.M. 2003), with the most recent review of the Italian seismic hazard map; the latter, as in the case of cladding panels, is related to the use of mechanical devices designed solely for out-of-plane loads, neglecting in-plane loads and displacement compatibility associated to the interaction with the supporting structure.

During an earthquake, the connections between precast elements need to accommodate high relative displacements and rotations due to a stiffness lower than the connected RC precast elements. Furthermore the connection displacement and rotation demand is emphasized by the flexibility of the structural layout owing to the high storey heights and the in-plane flexibility of roofs with no mechanical links between joists and with extensive presence of skylights.

Considering structural elements connections relying solely on friction, the magnitude of shear friction capacity is generally not sufficient if compared to the shear demand at the contact surfaces during an earthquake (Magliulo *et al*. 2008, Liberatore *et al*. 2013, Belleri *et al*. 2014). As shown in recent experimental tests (Magliulo *et al*. 2011), the value of neoprene-concrete coefficient of friction varies between 0.09-0.13 and even lower values of friction could be found if the seismic vertical component is considered. These small friction coefficients could have determined anticipated sliding between structural members and their consequent fall.

In the aftermath of Emilia earthquakes, the Italian Civil Protection, in collaboration with Italian universities, distributed a document (Gruppo di Lavoro 2012) with a summary of the most relevant deficiencies of the precast industrial buildings hit by the earthquakes and a series of retrofit solutions to increase precast buildings structural performance. The document subdivides the rehabilitation and retrofit process into two levels: the first level aims at solving the most relevant structural problems providing continuity between precast structural elements, as with mechanical devices in lieu of friction based connections; the second level aims at increasing the global building performance in order to achieve a retrofitted structure able to sustain a spectrum-compatible seismic event with a peak ground acceleration (PGA) at least 60% of that required by the current building code (D.M. 14/01/2008). The distinction into two retrofit levels and the requirements for each level were dictated by the contingency of providing quick rehabilitation rules to obtain financial support from the Italian government in order to increase the structural safety of the industrial precast buildings hit by the earthquakes and to reduce production downtime. This distinction into two retrofit levels is not considered herein.

The present paper discusses the assessment of precast industrial buildings typical of the Italian and the Southern European territory with particular attention to the most relevant problems highlighted by the Emilia seismic sequence, like connections between structural elements and local vulnerabilities of the structure. The assessment procedure is developed following the indications of EN 1998-3:2005 (CEN 2005a). Retrofit solutions are proposed in order to improve both local and global response of precast industrial buildings when subjected to earthquakes.

2. Seismic vulnerability assessment

2.1 General assessment procedures

Several assessment methods are suitable to estimate the seismic vulnerability of existing precast buildings. Although with the advancement of sensor technology, increase of computational power and development of system identification methodologies, structural health monitoring techniques received increased attention as a potential tool for damage diagnosis and prognosis of civil infra-structures and buildings (ASCE Technical Committee on Structural Identification 2013, Farrar and Worden 2007, Fan and Qiao 2011) and applications to precast RC structures have been investigated (Belleri *et al*. 2014), this type of approach is generally not considered in building codes. The assessment procedures found in national and international building codes, as in D.M. 14/01/2008, EN 1998-3:2005 and FEMA 356, are directly related to worldwide recognized seismic design procedures.

Linear static methods are the simplest way to estimate the seismic response of a building, although their applicability requirements in terms of structural plan/elevation regularity and fundamental period are generally restrictive for the precast industrial buildings considered: the latter because one-storey RC hinged frames with high and slender columns are characterized by high fundamental periods; the former because the mass and stiffness distribution could sometimes be irregular due to cladding or infills arrangement along the perimeter or to the presence of multi-storey RC office buildings inside the main precast structure, which compromise the building regularity.

Response spectrum analyses allow to consider and combine different vibration modes. The well known and widely adopted approach is basically an elastic analysis in which a behaviour factor is introduced in order to account for structural non-linearity as plastic hinge formation. The main issue of this method is related to the choice of the behaviour factor.

Non-linear assessment methods are based either on static or dynamic analyses. In the case of non-linear static (pushover) analysis, as the N2 method proposed by Fajfar and Gaspersic (1996), an appropriate lateral force distribution is applied to the system with increasing intensity and a force-displacement capacity curve is obtained, converted into an equivalent single degree of freedom system curve, as a function of the modal participation factor, and compared to the demand spectrum; different pushover variants are available as including higher modes contribution (Chopra and Goel 2002, Kreslin and Fajfar 2011) or adapting the lateral force distribution according to the actual structural stiffness after nonlinearities development (Bracci *et al*. 1997). In the case of non-linear dynamic analyses, the non-linear behaviour of the elements and the contribution of the higher modes of vibration are included by applying an acceleration history at the base of the structure and solving the non-linear dynamic problem at time increments, often by means of incremental dynamic analysis (Vamvatisikos and Cornell 2002).

The seismic response of a structure can be described taking as reference the peak ground acceleration (PGA) associated to the achievement of the target limit state considered, such as the collapse of a connection or the failure of a structural element. Seismic vulnerability indexes are obtained comparing available and required PGA and return period for a certain limit state with a defined probability of exceedance.

All the assessment analysis procedures presented above permit the definition of a building seismic vulnerability index on global scale, considering the overall behaviour of the structure. However, typical industrial precast structures often exhibit independent local failure mechanisms which jeopardize the global seismic performance. The lack of a rigid roof diaphragm and, more generally, the lack or under-designed connections between structural elements could lead to partial collapse of the structure. The peculiar characteristics of precast structural elements and connections in transferring seismic loads must be carefully taken into account to develop a reliable structural model and the safety assessment should be performed considering all the possible mechanisms of local collapse before carrying out a global seismic performance evaluation.

Novel performance based assessment procedures have recently been investigated. Methods based on displacements control and evaluation (Priestley *et al*. 2007) have been proposed and applied to different types of structures as precast industrial buildings (Belleri *et al*. 2012). Besides not being implemented in current building codes, their performance and suitability have not been investigated in the case of extensive masonry infills between adjacent columns and in the case of precast RC cladding panels with rigid connections.

2.2 EN 1998-3:2005 assessment procedure

EN 1998-3:2005 (CEN 2005a) adopts the aforementioned standard assessment analysis procedures in order to evaluate the effects of the seismic action. In addition, peculiar aspects are considered and herein reported.

The damage in the structure is defined according to three limit states, namely Near Collapse (NC), Significant Damage (SD) and Damage Limitation (DL), whose probability of exceedance is 2%, 10% and 50% in 50 years, respectively. It is worth noting that SD limit state corresponds to the no-collapse requirement limit state adopted in the design of new structures (EN1998-1: 2004) while NC limit state considers the full deformation capacity development of the structural elements.

The seismic evaluation of existing buildings, as the precast industrial structures considered, is strongly influenced by the knowledge level achieved by the engineer in the initial phases of the assessment procedure. The main aspects to be investigated should cover the identification of the structural system, the data collection about the properties of the structural elements, in terms of strength and stiffness, and the properties of the materials adopted. For precast structures, the connection details between structural elements play a significant role, as they affect the global behaviour of the structure during a seismic event.

Three different knowledge levels can be identified in relation to the amount of the data acquired: limited (KL_1), normal (KL_2) and full knowledge level (KL_3). The determination of the appropriate level takes into account the survey of the geometrical properties of the elements that may affect the structural response, the structural details, as reinforcement distribution and connections characteristics, and the mechanical properties of the materials. The above information could be obtained from available documentation, field investigation and in-situ/laboratory measurements and tests. For each knowledge level a confidence factor CF is associated: 1.35, 1.20 and 1.00 for knowledge level KL_1, KL_2 and KL_3, respectively.

The seismic capacity of the members is evaluated considering a reduction of the mean values of the material properties, obtained from in-situ tests and from additional sources of information, by the confidence factor CF, corresponding to the knowledge level achieved, and by an additional factor equivalent to the material partial safety factor γ_M adopted in the design of new structures (EN1998-1: 2004). Only in the case of ductile members and linear analyses the material properties are reduced by the sole confidence factor.

In the case of analyses involving a behaviour factor, a value equal to 1.5 is allowed for concrete structures, higher values can be used if suitably justified with reference to the local and global available ductility.

3. Precast industrial structures: local vulnerabilities and retrofit solutions

The typical structural layout of Italian and European RC precast industrial structures is characterized by fixed end cantilever columns pin-connected to pre-stressed RC beams supporting pre-stressed RC roof elements. The columns are commonly placed in socket foundations not interconnected by tie beams. The cladding is provided by masonry infills between lateral columns or by external RC precast panels. In the latter case the panels can be placed horizontally, connecting adjacent columns, or vertically, connecting the grade beam to the roof level.

Although traditionally considered as non-structural elements in the design process, the type of cladding influences the overall structural performance and vulnerability assessment. Masonry infills and precast cladding panels with rigid connections significantly increase the building stiffness and contribute to carry the seismic loads if detailed appropriately, while precast cladding panels with flexible or slotted connections do not interfere with the lateral force resisting system and the horizontal stiffness is provided by the cantilever action of the columns. In the former case, cladding interfering with the structural system, the vulnerability assessment and the retrofit are focused on force transfer between adjacent elements while in the latter case, not interfering cladding, the displacement capacity, the deformation compatibility and the connection ductility need to be specifically considered.

As mentioned before, the global performance of the building is often jeopardized by local vulnerabilities. This paper does not intend to evaluate seismic performance of structures with connection relying solely on friction; the implementation of mechanical devices between structural elements is considered herein a fundamental pre-requisite, before evaluating the seismic performance.

To assess local vulnerabilities, with and without interfering cladding, it is important to correctly estimate both the load and displacement demand. In the case of structures with interfering cladding, a first estimation of the seismic vulnerability could be carried out evaluating the load demand according to the design spectrum constant acceleration region, although for elongated diaphragms the in-plane flexibility of the floor could significantly contribute to lengthen the fundamental period and reduce seismic loads. In the case of flexible structures, i.e. without interfering cladding, a first estimation of the load demand on elements and connections considers the seismic spectral demand corresponding to a period based on 50% reduction (EN1998-1: 2004) of the columns flexural stiffness; the global and local displacement and ductility demand and deformation compatibility need to be assessed considering the diaphragm flexibility and the effective stiffness reduction of the columns, the latter being typically lower than 50% due to the low axial load and to the limited amount of longitudinal reinforcement. In the case of irregularities in the building plan and in the stiffness distribution or in the case of elongated diaphragms, the vulnerability assessment is carried out by means of finite element models, typically by means of response spectrum analyses.

Regarding retrofitting solutions, the present paper considers the seismic performance enhancement of local vulnerabilities. It is possible to assign all the seismic loads to an additional lateral force resisting system, as cross bracing or external RC walls, and adopting a behaviour factor according to new buildings, however this solution is not considered herein. When additional bracing systems are implemented, it is fundamental to check the interaction and the compatibility between the new system and the existing one, especially when the existing structure is stiffer than the new bracing system, as it could be the case when masonry infills are present.

The following section considers the most common vulnerabilities observed in precast industrial buildings following recent earthquakes in the Italian territory and provides retrofit solutions compatible with the hinged frame structural scheme adopted in the design process and in the construction practice.

3.1 Beams, joists and floor diaphragm

The typical structural layout of the considered industrial buildings is characterized by horizontal elements constituted by double pitched pre-stressed RC beams or, for shorter spans, by pre-stressed RC L-beams, I-beams and T-beams. Regarding joist elements, these are mainly constituted by double-T pre-stressed beams or by proprietary micro-shed elements. Hollow-core slabs are adopted mainly for intermediate floors; the roof, commonly not accessible as a floor, presents large openings for lighting purposes.

The joists are typically connected directly to the supporting beams by means of mechanical connections, while in the past this connection was provided by friction; no link exists between adjacent roof elements and no additional cast in place topping is poured, mainly due to the presence of large openings and to the relatively low vertical load demand. The diaphragm action at the roof level is therefore not provided: all the seismic roof loads act directly on the beams which experience horizontal loads not accounted for in the design phase. The beams are subjected to out-of-plane bending which could cause their premature failure; besides this, the leaning of the beam due to the column top rotation during a seismic event leads to an additional torque on the beam.

To enhance the beams out-of-plane performance it is possible to act on two aspects: increasing the lateral load capacity of the beam itself or reducing the lateral load demand. The former solution is accomplished for instance with fibre-reinforced polymers (FRP), while the reduction of the beam lateral load demand is obtained by providing a diaphragm at roof level, which will act as a deep beam in carrying the horizontal seismic loads. In the case of double-T roof elements the diaphragm could be realized acting on the top of the roof interconnecting adjacent elements (Fig. 1(a)), without interruption of the industrial activities. In the case of limited skylight openings it is possible to restore diaphragm load transfer capacity by introducing a planar steel truss in addition to the aforementioned retrofit scheme. When the roof is constituted by micro-shed or open-shape precast elements with openings between consecutive joists, a diaphragm action could be triggered by introducing diagonal tendons in the structural layout (Fig. 1(b)) and improving the beam-joist connection capacity in order to make the roof behaving as a planar truss.

The diaphragm in-plane bending and shear capacity are provided by lateral chords and mechanical connectors respectively (Fig. 1(c)). The flexural and shear strength and stiffness are computed considering the contribution of the double-T flanges and the additional mechanical devices (Zheng and Oliva 2005). The diaphragm stiffness needs to be checked and tuned in order to reduce seismic load transferring to the beam. A possible way to do this is by a finite element

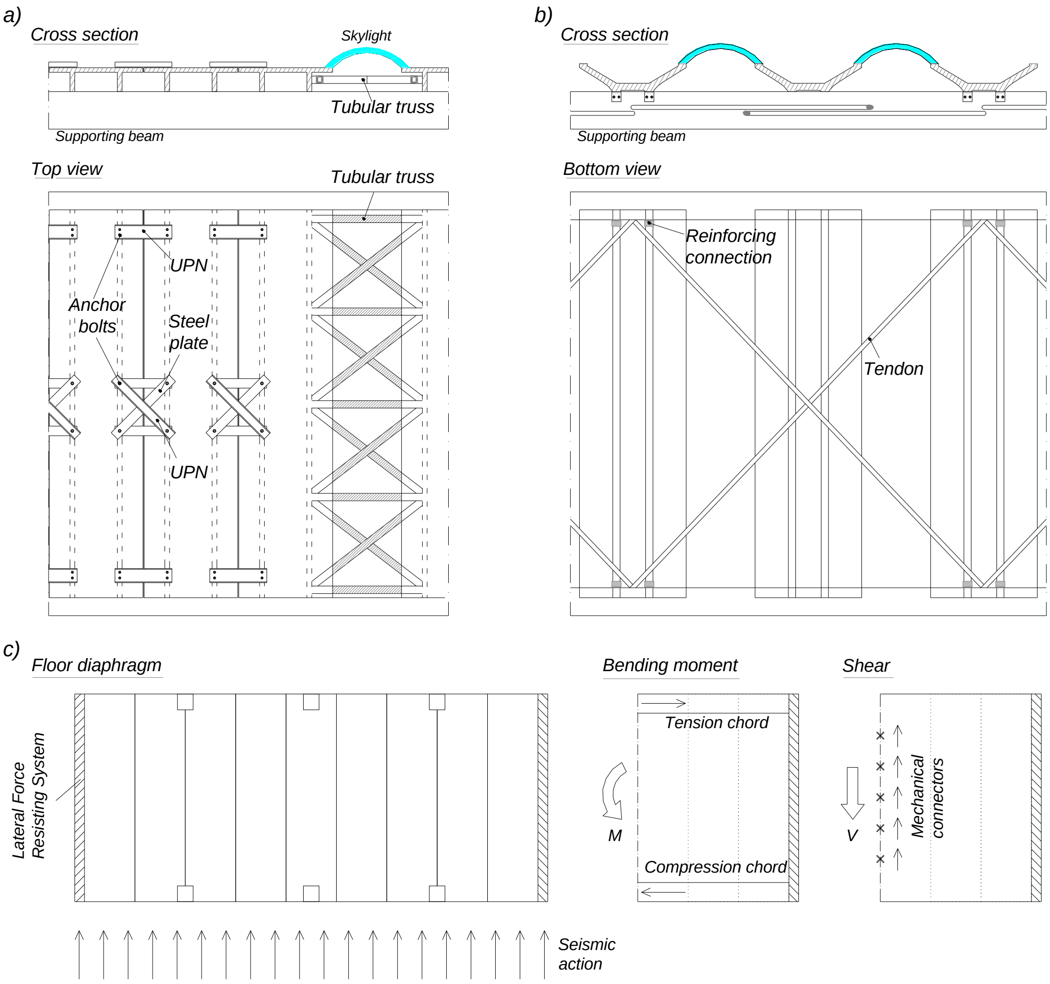


Fig. 1 Diaphragm stiffening solution: (a) double-T beams and (b) open-shape beams
Note: UPN stands for European Standard Channel

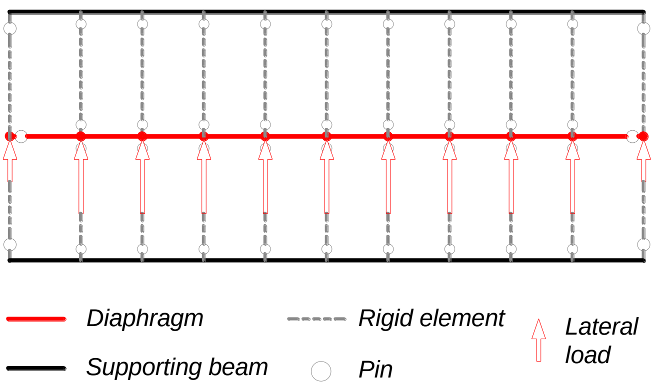


Fig. 2 Diaphragm finite element model for stiffness evaluation

model in which the diaphragm is modelled as a beam with equivalent Young's and Shear modulus (Zheng and Oliva 2005) and rigid pinned-pinned elements connect the diaphragm to the supporting beams (Fig. 2).

When a diaphragm action is not strictly necessary to reduce beam out-of-plane loads, it is necessary to provide mechanical devices as beam-joist connections to inhibit element loss of support. These connections should be able to transfer the horizontal seismic loads and to accommodate deformations arising from seismic displacement compatibility between structural elements (Belleri *et al*. 2013), considering the concentration of strain demand at the connection level due to the higher flexibility of the connection compared to the connected precast elements.

When diaphragm action is absent and beam-joist connections are under-designed or relying solely on friction, the behaviour of adjacent precast frames is basically decoupled during a seismic event; the lateral frames are more rigid compared to internal ones because of cladding and the internal frames carry a seismic mass roughly doubled, therefore adjacent frames could move out-of-phase and beam-joist connections experience displacements and rotations that could cause their premature fall.

3.2 Columns and foundations

The lateral seismic loads acting on the beams are transferred to the columns by often under-designed connections. Dowel connections are typically adopted and are constituted by threaded rods connected to the top of the columns or to column's corbels by threaded inserts. These rods are placed inside vertical sleeves in the beams and subsequently grouted. The threaded rods are occasionally tensioned and sometimes substituted by common deformed bars protruding from the column. In the case of double-pitched beams, the beam is resting in RC forks at the top of the columns with and without the presence of horizontal steel dowels, for new and old buildings respectively.

When dowel connections or RC forks are not able to sustain the seismic loads, the simplest retrofit solution consists in additional mechanical devices. Special attention should be placed in the resulting boundary conditions, in fact when mechanical devices connect beams and columns the static scheme could change from a pinned to a fixed connection. To maintain a hinged joint a possible solution is the adoption of slotted connections which allow vertical movements arising from joint rotation and restrain horizontal movements (Fig. 3).

The change of the static scheme could be triggered by excessive relative movements between the column and the beam, therefore the estimation of the column flexural stiffness by means of sectional analysis is suggested in order to evaluate the actual displacement and rotation demand of existing and new connections.

Another column's vulnerability typically observed in past earthquakes is the development of short column failure mechanisms in correspondence to ribbon glazing or to cladding discontinuities. In this case possible retrofit solutions consist in reinforcing the columns, for instance with FRP, or providing another path to the horizontal seismic loads, as with the addition of bracing in correspondence to the short column (Fig. 4). The use of FRP, HPFRC (High Performance Fibre Reinforced Concrete), steel or RC jacketing could allow also to increase the base column strength and ductility (Fig. 5).

Regarding foundations, these are mainly constituted by isolated socket foundations on-site grouted with low strength grout and not interconnected by tie beams. Therefore, during a seismic event relative movements at ground level could happen, although the presence of the RC floor

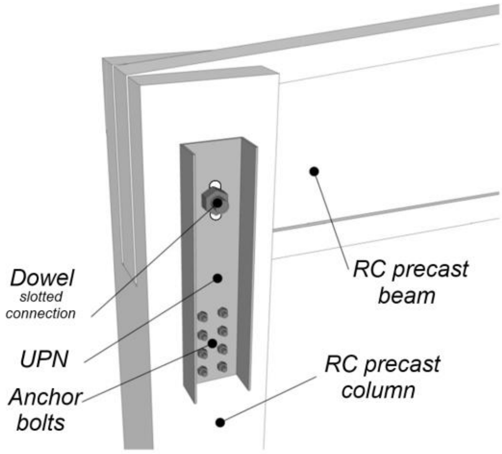


Fig. 3 Column to beam mechanical connection

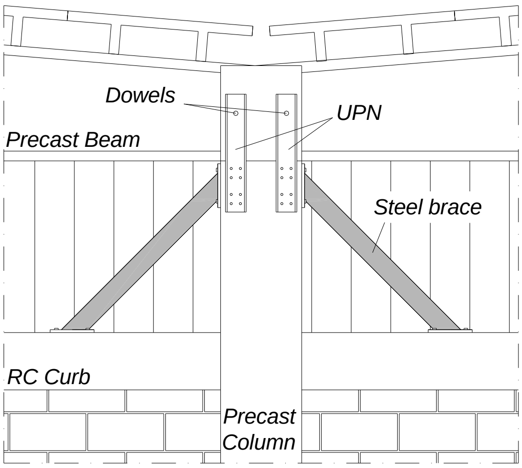


Fig. 4 Short column strengthening solutions

slab, typically present in industrial facilities, could provide a certain amount of restraint; in this situation, lateral columns could be conveniently connected to the floor slab (Fig. 6). A small amount of relative movements at ground level is not critical for structural elements in the case of flexible super-structures, due to the high storey height and to the hinged frame static scheme, but could cause damage in non-structural elements, as the connections of horizontal cladding panels at base level.

Other aspects concerning foundations are the additional vertical and horizontal loads deriving by the bracing action of infills or RC precast cladding panels rigidly connected to the super-structure and the possible elastic foundation uplift, owing to the under-designed foundation footprint of existing structures, designed by solely gravity loads or gravity plus wind loads, and the low amount of vertical gravity loads. Once elastic uplift is triggered there is no damage at the foundation level and the super-structure load demand is governed by the foundation overturning moment associated to uplift.

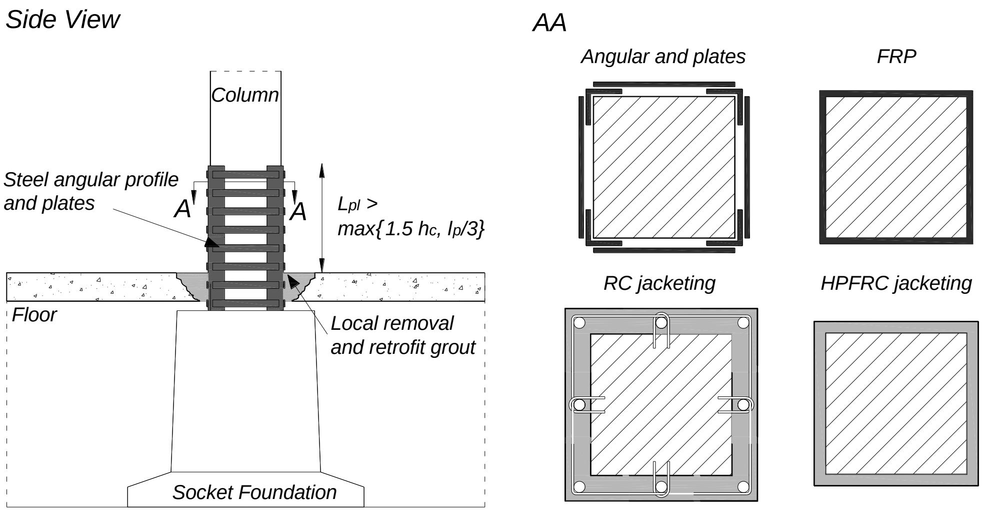


Fig. 5 Column strengthening solutions

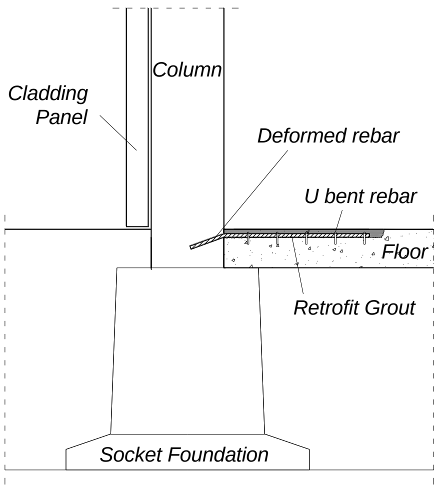


Fig. 6 Floor to column connection

3.3 Infills and cladding panels

Precast cladding panels and masonry infills, although typically considered as non-structural elements in the industrial buildings investigated, could cause severe damage in terms of both downtime and fatalities as highlighted by recent earthquakes in the Italian territory. Cladding could interfere with the lateral force resisting system: stiffness discontinuity along the column height caused by infills and precast panels could generate short column type failures while the displacement demand caused by the high lateral flexibility of the structural system could induce a connection failure at the precast panels-supporting structure joints.

Regarding masonry infills, the out-of-plane overturning is the typical failure observed, owing to the main vulnerabilities related to the absence of connections to the primary structural elements, the absence of a RC curb at the infill top connecting the infills to the columns and the presence of ribbon glazing. Possible solutions to increase out-of-plane capacity are related to the creation of restraints to avoid overturning as with additional RC beams and columns (Fig. 7(a)). Once the out-of-plane failure is inhibited and panel lateral confinement provided, as with the aforementioned RC beams, the infills could contribute to the seismic load transferring: the panel strength is related to the infill in-plane capacity (Smith 1962, Decanini *et al*. 2004, CEN 2005b). Different solutions are available to increase in-plane capacity as the creation of cross bracing by means of steel plates connected to each side of the infill with through bolts to avoid buckling (Fig. 7(b)). Fibre-reinforced polymers (FRP) and Textile Reinforced Mortar (TRM) (Papanicolaou *et al*. 2008) could also be adopted, providing appropriate connections to the main structural elements to avoid overturning.

Regarding precast RC cladding panels, horizontal and vertical elements are commonly adopted, the former connected to adjacent columns, the latter connected to roof girders/gutter-beams and ground level. In the case of no-sliding connections, as in the majority of precast industrial buildings with cladding panels, the panels contribute in carrying horizontal seismic loads and to the building lateral stiffness up to the connection failure. As a result, there is a reduction of the structural fundamental period and a consequent increase of the seismic demand on structural elements and connections. Greater than expected loads could arise and compromise the structural integrity, therefore local vulnerabilities need to be evaluated considering the period of vibration of the stiffened structure, up to failure of existing panel connections, in addition to the case where only the main structural elements are considered.

Cladding panel connections, even if designed to sustain the out-of-plane seismic loads, are rarely ductile enough to guarantee displacement compatibility between the panels and the supporting structural elements. In addition, in some situations, typically for horizontal cladding panels, the connections are not in view and no direct inspection is possible. Therefore the simplest retrofit solution consists in providing additional connections, although it is still possible to substitute existing connections when easily accessible. The new devices, both in substitution or additional to existing connections, need to be able to carry the panel gravity load, avoid panel

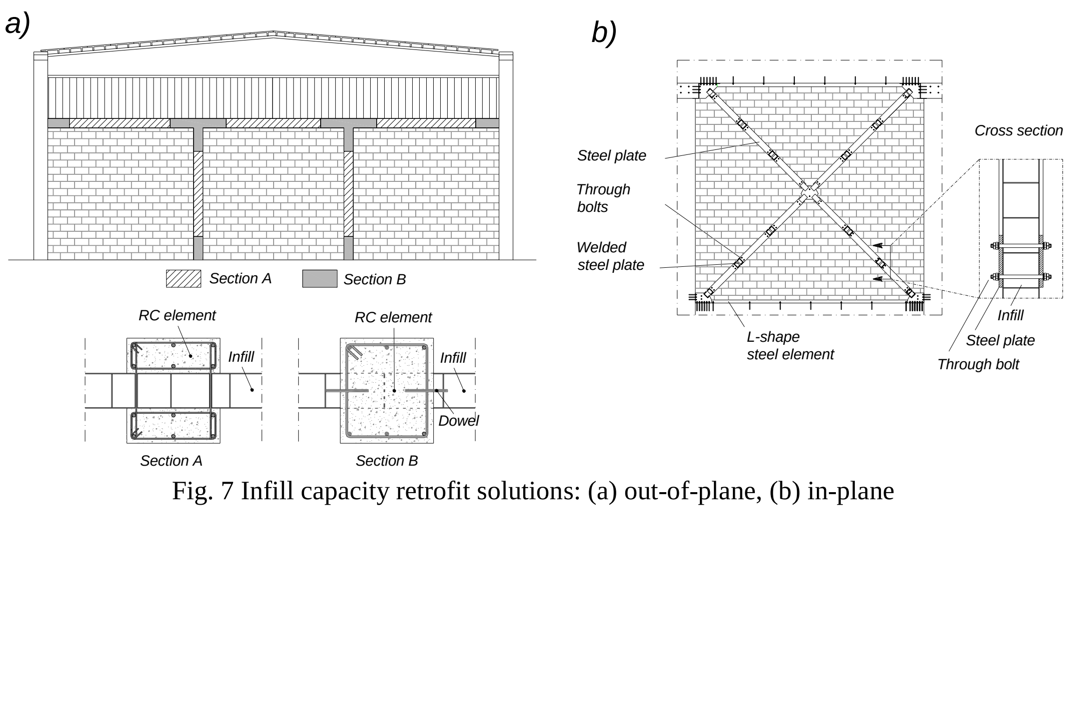


Fig. 7 Infill capacity retrofit solutions: (a) out-of-plane, (b) in-plane

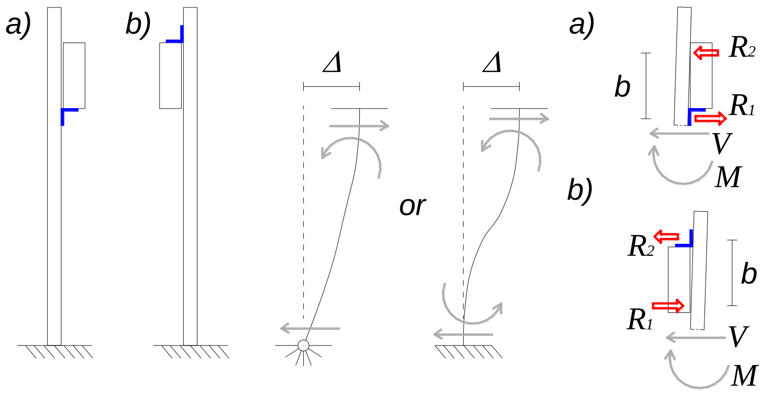


Fig. 8 Connection load demand due to out-of-plane displacement of vertical cladding panel

overturning and guarantee displacement compatibility in both principal directions.

In the case of vertical cladding panels, out-of-plane displacements could cause premature failure of existing and new connections. Looking for instance at the static scheme of Fig. 8, the horizontal displacement Δ induces a rotation in the panel which could cause the contact to the supporting beam. As a consequence a tension force R_1 arises in the case of bottom connection (Fig. 8(a)) and R_2 in the case of top connection (Fig. 8(b))

$$R_1 = V + M / b \tag{1}$$

$$R_2 = M / b \tag{2}$$

Where V and M are respectively the panel's shear and bending moment associated to the lateral displacement Δ, b is the lever arm between R_1 and R_2. The value of R_1 depends on the static scheme deriving from the effective boundary conditions. It is possible to apply capacity design to avoid connection failure by considering M and V associated to panel flexure failure. Back-up solutions have been proposed in the aftermath of the Emilia earthquake (Gruppo di Lavoro 2012) as steel cables able to provide passive restraint to panel overturning after existing connections failure, although their design is rather complex due to the coupled building-panel response due to connecting cables acting solely in tension. A possible design strategy is to provide a cable able to elastically absorb the maximum kinetic energy experienced by the panel during an earthquake.

4. Vulnerability assessment and retrofit example

The assessment procedure is applied to a precast RC industrial building representative of the most common typology of one-storey precast structures on the Italian territory. The main deficiencies associated to inadequate structural details are considered, with particular notice to the structural elements connections and to the interaction with cladding.

The building is constituted by columns spaced 6 m in X-direction and 20 m in Y-direction placed in isolated RC socket elements without additional tie-beams. The roof, constituted by double-T pre-stressed beams, is supported by double pitched pre-stressed RC beams in Y-direction

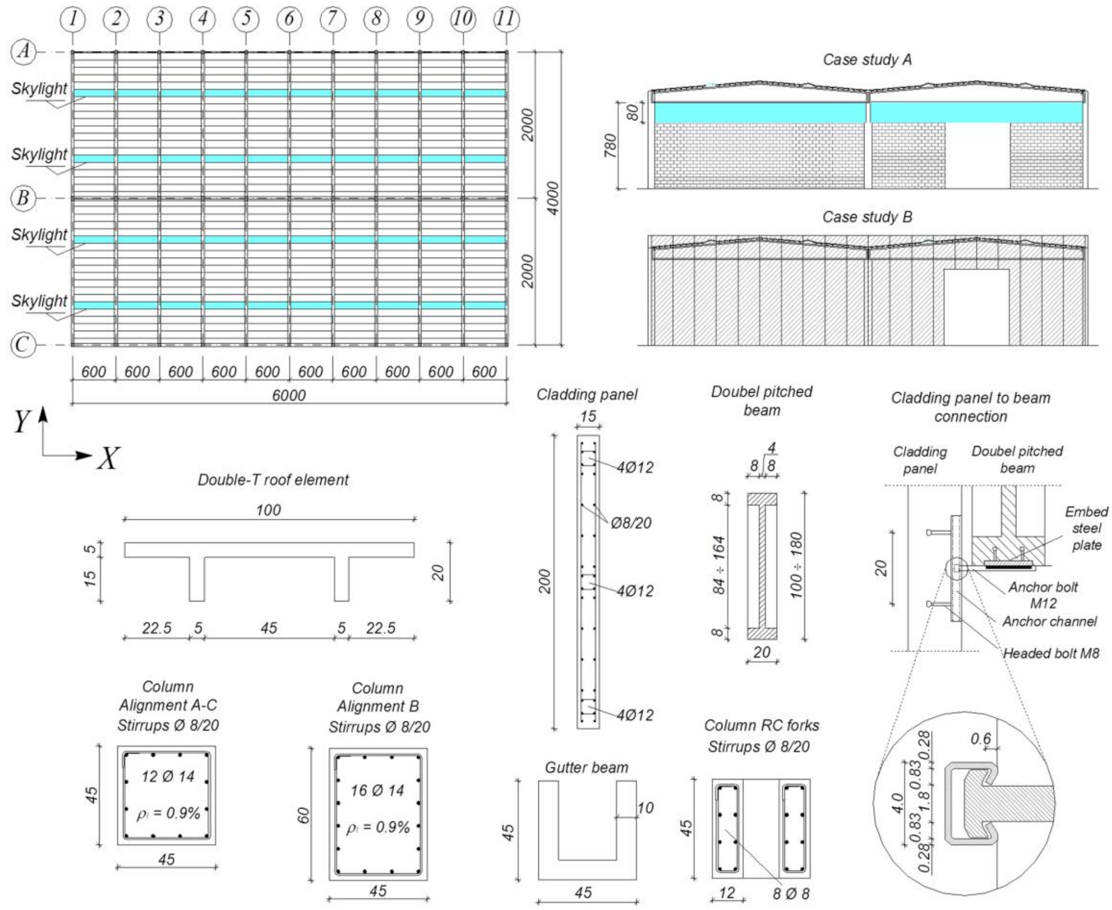


Fig. 9 Building geometry relevant data (measures in cm)

with a cross-section height varying from 1 m to 1.8 m at the centre. Two skylights, made by polycarbonate, substitute two double-T beams in each span. No mechanical connection is provided between beams and columns and between roof elements and supporting beams. The main geometry data of the considered building are reported in Fig. 9.

Two different types of cladding are considered to highlight differences in the assessment procedure: masonry infills between columns (Case study A) and vertical RC precast panels (Case study B). In case study A, the infill thickness is 15 cm and 25 cm for infills spanning in *X* and *Y* direction respectively; ribbon glazing is present along the entire perimeter. In Case study B RC pre-stressed gutter beams connect the column tops in *Y*-direction; the cladding panels are connected to the gutter beam at the top and to the supporting grade beam at the bottom. Case study B is additionally subdivided in interfering (Case study B-1) and non-interfering (Case study B-2) cladding panels.

According to EN 1998-1:2004 (CEN 2004) elastic spectrum Type 1 and Ground Type B, the reference parameters for the definition of the seismic action are S=1.5, T_B=0.15 s, T_C=0.5 s, T_D=2 s. The design ground acceleration (a_g) for the selected site is 0.092 g. The response spectrum is obtained by considering a behaviour factor q=1.5 (CEN 2005a) for both case studies (Fig. 10).

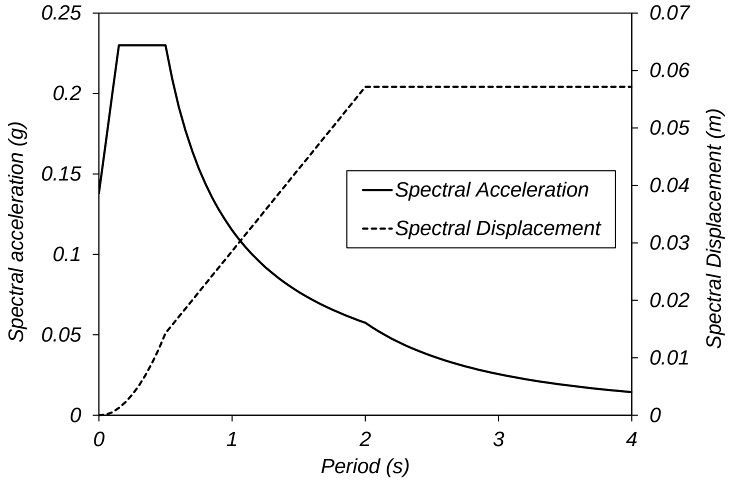


Fig. 10 Acceleration and displacement reference spectrum
Notes: behaviour factor q=1.5; ξ=0.05

Table 1 Material properties for the assessment evaluation and the retrofit design

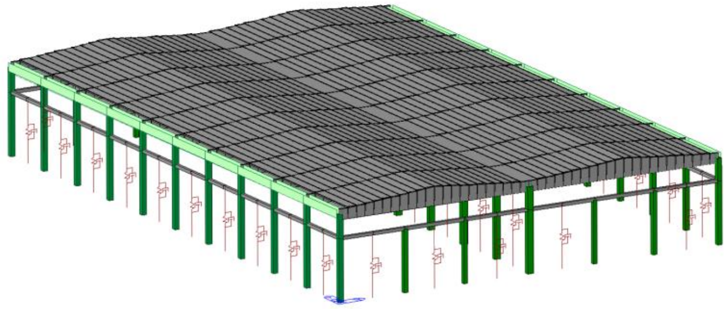
Assessment evaluation		
	E (MPa)	f_{cm} (MPa)
Concrete	37277	50
Steel	205000	450
Masonry	1200	3
Retrofit design		
	E (MPa)	f_{yd} (MPa)
Structural steel	205000	224
Pins and bolts	205000	640

E modulus of elasticity, f_{cm} mean strength, f_{yd} design yield strength

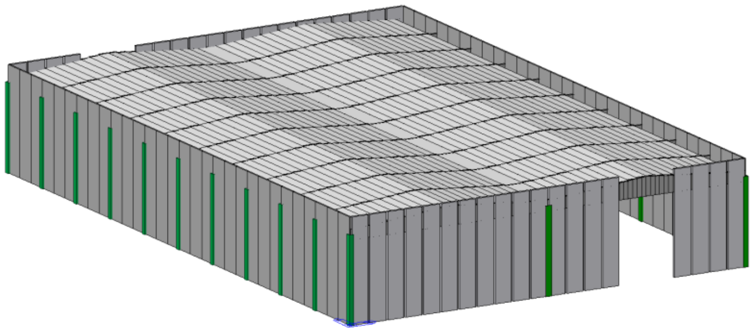
This low value is mainly associated to the low system energy dissipation, being the columns predominantly in the elastic range due to the structural flexibility. In addition, the column longitudinal and transverse reinforcement in the plastic hinge region does not fulfil anti-seismic detail requirements, such as closed stirrups, stirrup spacing and longitudinal reinforcement ratio and positioning.

The seismic vulnerability is evaluated by considering the damage of the structure correspondent to Significant Damage (SD) limit state. A limited Knowledge Level (KL_1) is assumed in order to represent the lack of available data both in terms of structural details and material properties; the correspondent Confidence Factor (CF) is 1.35. Table 1 reports the properties of the materials adopted in the assessment evaluation and in the retrofit design.

As suggested by building codes (CEN 2004, 2005a), the seismic vulnerability can be estimated by considering a reduction of the column flexural stiffness equal to 50% of the gross stiffness. This choice leads to a conservative seismic load demand but could underestimate rotation and displacement demand, therefore in the assessment procedure the displacement demand of the



(a)



(b)

Fig. 11 Finite element models: Case study A (a), Case study B (b)

columns is evaluated considering the effective stiffness, while the seismic load demand for the retrofit design is conservatively calculated considering 50% reduction of column gross stiffness.

The vulnerability assessment is carried out by means of response spectrum analysis. Beam elements are adopted in the finite element models (Fig. 11), while pin connections are provided at beam-column joints, being the friction based connections substituted by mechanical connections able to maintain the static scheme, hinged frame, assumed in the design process. As mentioned before, the floor diaphragm is modeled with an equivalent beam (Zheng and Oliva 2005) to account for the stiffness of the new floor connections. The behavior of the masonry infills is represented by a horizontal spring with equivalent in-plane stiffness. Soil-structure interaction is not considered.

Table 2 shows the seismic vulnerabilities previously described. Table 3 reports the failure mechanisms capacity, based on Fig. 9 details and Table 1 material properties, along with the corresponding vulnerability index I_a, defined as the ratio between the ground acceleration (a_g) corresponding to a selected failure mechanism and the ground acceleration demand for the considered site (a_g=0.092 g). A simplified approach (SA) and a finite element approach (FEA) are considered. In FEA a spectral analysis is conducted. In SA only the fundamental vibration mode in each direction is taken into account: the mass of the structure is multiplied by the spectral acceleration corresponding to the constant acceleration region for Case study A and Case study B-1 (interfering panels) and by the spectral acceleration corresponding to the period obtained from

Table 2 Seismic vulnerabilities

Id	Vulnerability description	Id	Vulnerability description
1	RC forks flexural failure;	6	Precast cladding panels connection out-of-plane failure;
2	Short-column at ribbon glazing shear failure;	7	Precast cladding panels connection out-of-plane displacement compatibility;
3	Masonry infills out-of-plane overturning;	8a	Column flexural failure - Line A, C;
4a	Masonry infills in-plane failure - Line A, C;	8b	Column flexural failure - Line B;
4b	Masonry infills in-plane failure - Line B;	9a	Column max displacement - Line A, C;
4c	Masonry infills in-plane failure - Line 1, 11;	9b	Column max displacement - Line B;
5	Precast cladding panels connection in-plane failure;		

the column lateral stiffness for Case study B-2 (non-interfering panels).

In the case of masonry panels, the out-of-plane capacity is governed by the maximum lateral acceleration associated to panel overturning as a rigid block, being absent a RC curb connecting the top of the panel to adjacent columns. The in-plane capacity is evaluated accounting for typical failure modes (Decanini *et al*. 2004): a capacity of 87.5 kN and 120.6 kN due to diagonal tension failure is associated to infills spanning in X and Y direction respectively.

The precast cladding panel vulnerability is related to the panel to beam connection. The in-plane and out-of-plane failure mechanisms are associated, respectively, to shear and axial capacity of the M8 bolts embedded in the panel (Fig. 9). To evaluate the panel and beam interaction due to displacement compatibility in the out-of-plane direction the following procedure was applied. The cladding panel corresponding to the center of the double pitched beam is considered. Being the panel-beam distance 1 cm the lateral rotation before panel to beam contact is 0.0055 rad (1 cm/180 cm). This rotation is associated with a lateral displacement at connection level $\Delta_{contact}$=4.3 cm (0.0055rad×780 cm). Being the panel pin-connected at the base, no internal actions arise due to out-of-plane displacements before panel to beam contact. The lateral displacement demand at connection level (Δ_d) is obtained directly from the elastic displacement spectrum at the fundamental period, resembling the building a single degree of freedom system. The use of the elastic displacement spectrum implies the equal displacement approximation.

From Eq. (1) the axial load in the connection (R_1) is

$$R_1 = V + M/b = \frac{3EI_{red}}{H^3}(\Delta_d - \Delta_{contact}) + \frac{3EI_{red}}{bH^2}(\Delta_d - \Delta_{contact}) \tag{3}$$

Considering a 50% reduction of the panel out-of-plane flexural stiffness, Δ_d=8.7 cm, b=180 cm, the previous equation leads to 15.02 kN.

From Table 3, it is observed that the I_a values obtained with the simplified approach are not always conservative due to the roof in-plane flexibility and to stiffness discontinuity, therefore in those situations a finite element model is recommended for vulnerability assessment.

Based on the results of Table 3, the following retrofit interventions are proposed (Table 4),

Table 3 Failure mechanisms capacity and corresponding acceleration vulnerability index

Id	Capacity(*)	I_a value Case study A		I_a value Case study B-1		I_a value Case study B-2	
		SA	FEA	SA	FEA	SA	FEA
1	M_{Rd}=7.5 kNm; V_{Rd}=36.19 kN	0.26	0.63	0.26	0.91	0.75	1.70
2	V_{Rd}=47.5 kN	0.65	0.54	//	//	//	//
3	Max lateral acceleration 0.031 g	0.34	0.34	//	//	//	//
4a	N_{Rd}=87.3 kN	1.40	1.30	//	//	//	//
4b	N_{Rd}=87.3 kN	1.01	1.46	//	//	//	//
4c	N_{Rd}=120.6 kN	1.17	1.53	//	//	//	//
5	V_{Rd}=10.7 kN	//	//	0.59	0.12	//	//
6	N_{Rd}=13.4 kN	//	//	3.42	3.55	10.05	7.45
7	N_{Rd}=13.4 kN	//	//	0.89	0.89	0.89	0.89
8a	M_{Rd}=178.6 kNm	0.49	1.38	0.49	1.30	1.42	1.36
8b	$M_{Rd,y-y}$=236.0 kNm; $M_{Rd,z-z}$=320.5 kNm	0.38	1.15	0.38	1.08	1.11	1.10
9a	θ_u=0.035 rad	1.12	2.58	1.12	4.23	3.28	2.46
9b	$\theta_{u,y-y}$=0.035 rad; $\theta_{u,z-z}$=0.029 rad	0.92	2.03	0.92	1.91	2.70	2.03

(*) M_{Rd}, V_{Rd}, N_{Rd} and θ_u are flexural, shear, axial and rotation capacity respectively

Table 4 Retrofit interventions

Id	Case study A	Case study B-1
1	UPN 140, L=1.25 m Anchorage bolts 4+4 M20 (Fig. 3);	UPN 140, L=1.25 m Anchorage bolts 4+4 M20 (Fig. 3);
2	Square steel pipes 60×60×6 mm, L=3.2 m Anchorage bolts 4+4 M12 (Fig. 4);	//
3	RC beams 30×30 cm-rebars 4ϕ20 RC columns 45×45 cm-rebars 8ϕ12 (Fig. 7);	//
5	//	Sliding connections Displacement capacity 12 cm;
7	//	Out-of-plane anchorage bolts 2 M16

according to what previously described, in order to achieve the seismic performance required for the site considered. In addition to the retrofit measures of Table 4(a) floor diaphragm was realized according to Fig. 1(a): the flanges of adjacent double-T roof elements are connected at each end by a UPN 160 (European Standard Channels) and at the centre by a cross connection constituted by a UPN 100 and a 100×5 mm steel plate. For Case Study B-2 only 2 M16 anchorage bolts are needed for panel to beam out-of-plane displacement compatibility.

5. Conclusions

The seismic sequence which hit the northern Italian territory in 2012 produced extensive damage to reinforced concrete (RC) precast buildings. The paper considered the seismic vulnerabilities associated to precast facilities observed in the aftermath of the Emilia earthquake, typically one-storey RC precast industrial structures with fixed ended columns placed in isolated socket foundations and pre-stressed RC roof elements.

The main vulnerabilities recorded are associated to the loss of support of structural elements relying solely on friction capacity as horizontal load transfer mechanism, being the damaged facilities built before the enforcement of modern seismic codes and before the most recent classification of the Italian seismicity. Other sources of seismic vulnerability are: development of short column mechanisms in correspondence to ribbon glazing, failure of RC forks sustaining the main beams, out-of-plane failure of roof beams due to the absence of floor diaphragms, limited ductility capacity of RC columns, isolated footings relative movements, out-of-plane and in-plane failure of masonry infills and precast cladding panels connection failure. Cladding panel connections, even if designed to sustain the out-of-plane seismic loads, are rarely ductile enough to guarantee displacement compatibility between the panels and the supporting structural elements.

Retrofit solutions were investigated, substituting friction connections with mechanical devices, in order to enhance the seismic performance of local vulnerabilities without compromising the static scheme of hinged frame commonly adopted in the design process. Although it could be possible to assign all the seismic loads to an additional lateral force resisting system, this solution was beyond the purpose of the present paper.

The seismic assessment methodologies adopted in current building codes were highlighted. Among these, the most suitable assessment strategy for the considered building type is the response spectrum analysis. In order to assess seismic vulnerabilities and design retrofit solutions a case study was selected and the assessment procedure according to EN 1998-3:2005 (CEN 2005a) was carried out. Two typical configurations were investigated: cladding interfering and not interfering with the structural system. In the former case the vulnerability assessment and the retrofit are focused on the force transfer between adjacent elements while in the latter case the displacement capacity, the deformation compatibility and the connection ductility play a fundamental role.

The assessment procedure allowed to define the spectral acceleration associated to each local vulnerability and to design the appropriate retrofit solution. It is observed that simplified procedures based on seismic loads obtained from spectral acceleration corresponding to the fundamental vibration mode in each direction are not always conservative, especially in the case of elongated and flexible floor diaphragm and in the case of stiffness discontinuities. In those cases a finite element assessment approach is recommended.

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