Structural vulnerability of old precast building stock: case studies from the period 1940s-1970s

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ABSTRACT: This work aims at evaluating the structural performance indexes of different typologies of reinforced concrete precast industrial buildings built in the 40-70s of last century. Precast industrial buildings belonging to the pioneer era of this technology are quite common all over Italy and were mostly devoted to house industrial activities. As a matter of fact, they were designed according to criteria which are now obsolete, mainly for lower static actions as compared to current codes and without any concept for seismic or fire resistance. As such, they potentially feature strong vulnerability. With reference to two industrial complexes located in Brianza (Northern Italy), both built with successive expansions from the '40s to the '70s, the structural performances of 10 different typologies representative of the Italian precast building stock of this period are evaluated under static, seismic and fire actions. These typologies include vault roofs with truss systems, restrained arches, tapered beams, and moment-resisting portal frames. A preliminary assessment of the vulnerability associated to these types of structures is performed based on their geometry and reinforcement details. Tentative ranges of cost associated to their retrofit are calculated based on the experience of past interventions gained by the technical partner of the paper.

1 INTRODUCTION

The pioneer age of the precast concrete industry in Europe and Italy can be framed from right after the 2nd world war to 70s of last century. The contribution of Italy, where the autarchic regime of pre-WWII imposed to focus on concrete solutions even for industrial structures, contrarily to the wide use of steel in other European countries, was crucial. In the pioneer age, structural engineers kept inventing innovative solutions with brilliant ideas and sometimes authentic dare, progressively breaking span records and responding to a frantic construction demand, mainly concentrated into the fast-growing national heavy industry. Since then, the Italian precast concrete industry has been a world excellence and still inspires the international technical and scientific community on the topic with outstanding realisations and innovative ideas.

The precast buildings of the pioneer age were designed according to criteria which now appear obsolete (*e.g.* allowable stress method) and against lower loads as compared to current standards, disregarding any seismic or fire resistance criterion. As such, these structures represent today a possible source of high vulnerability and their retrofitting is a challenge for the national engineering and construction industry community. Some solutions and methodologies have been suggested in Magliulo *et al.* (2008), Belleri *et al.* (2015) and Toniolo and Dal Lago (2018).

2 SCOPE, METHODOLOGY AND ASSUMPTIONS

The present paper collects the experience of pilot studies carried out on two large industrial complexes both located in the region of Brianza, in Northern Italy. The first pilot

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study concerned the possible conversion of a nowadays abandoned large industrial complex covering over 30000 m^2 to commercial functions. This industrial complex was built with successive expansions from the late ‘40s to the late ‘70s by several contractors. 8 different building typologies can be highlighted (Figure 1a).

The second pilot study concerned the possible retrofitting of part of the complex of about 10000 m^2 which hosted the production factory of the precast concrete industry ELP (ELementi Prefabbricati), operating since the early ‘60s to the late ‘80s. The complex was self-built by the precast company in the ‘60s. The investigated structures are 2 typologies, highlighted in Figure 1b. The same complex contains one of the first examples of application of prestressing to roof elements with a tapered beam system built in 1965 employing the technology of draped post-tensioning of centrally epoxy-glued symmetric segments (Dal Lago, 1969). Unfortunately, this part of the complex was not required to be investigated. Additional information about the work carried out can be found in Berretta (2018) and Dal Lago *et al.* (2018).

(a)

(b)

Figure 1. Case study industrial complexes: (a) with 8 considered structural typologies and (b) with 2 considered structural typologies.

The aim of the work carried out is to preliminarily investigate the level of vulnerability of the different structures, defining and quantifying structural performance indexes associated to the attainment of the different Ultimate Limit States (ULS), as identified by the current codes, and to estimate a range of cost related to their retrofitting. This information was considered crucial by the owners of the complexes to draw strategical decisions about their present and future use.

With the aid of a wide historical archive of the original projects and drawings, and of complementary visual inspections, most of the geometries and reinforcement layouts of the structural elements could be deduced. The mechanical properties of concrete and steel were mostly deduced from the drawings, and, where not present, they were guessed through a probabilistic catalogue built over the experimental data on destructive tests performed in those decades (Verderame *et al.*, 2011). Missing information on geometrical and reinforcement details of the secondary roof elements was assumed based on the information contained in Dal Lago (1972).

The structural analysis has been performed according to the following assumptions:

- a typical modulus of each typology has been modelled through a FEM code (G+D Computing, 2010), neglecting the structural interaction with the adjacent moduli;
- a deep level of knowledge of the structural details has been assumed, neglecting additional safety coefficients related to uncertainty;
- an optimal state of conservation has been assumed for all structures, neglecting corrosion scenarios of the rebars or of the steel profiles, as from visual inspections;
- the bracing effect of internal or external infill masonry walls has been neglected;
- an elastic structural behaviour ($q = 1.5$) has been assumed under seismic actions: the lack of transverse confinement of the longitudinal reinforcement of columns, because of small-diameter stirrups spaced larger than 200 mm, jeopardise the potential energy dissipation capacity of the same longitudinal bars due to early post-yielding rebar buckling;
- a seismic soil classification type C according to EC8 has been assumed (deep deposits of dense or medium-dense sand, gravel or stiff clay), with perfectly rigid foundations.

Moreover, the structural performance indexes are based on the assumption that all simply supported elements are provided with post-inserted mechanical connections in the design stage, which is compulsory to avoid the extreme vulnerability related to possible loss of support or beam overturning (Dal Lago and Toniolo, 2018, Titi *et al.*, 2018), and that this intervention could foster the assumption of a rigid diaphragm behaviour of the roof deck (Dal Lago and Ferrara, 2018 and Dal Lago *et al.*, 2019).

3 PRECAST STRUCTURAL TYPOLOGIES

A short description of each structural typology is provided in this section, with pictures of the buildings and a technical sketch with the main dimensions of buildings and structural elements.

Typology 1 (Figure 2) is the oldest of the complex, dating back to 1948. It is made with a triangular trussed roof with 150x400 mm struts spanning 12.5 m made of diaphragmed X-shaped precast concrete elements completed with a cast-in-situ topping and with a lower steel bar acting as tie. The vertical frame structure consists of square 35x35 cm 4 m tall columns and T-shaped 60(35)x80 cm 7.5 m long beams was fully cast-in-situ. The roof slab is mixed concrete-masonry.

Typology 2 (Figure 3), completed in 1958, is made by a precast reticular 10x34 cm arch system spanning 16.2 m completed with a cast-in-situ topping and with a lower steel bar acting as tie. Precast ribbed roof elements cover the span of 2 m in between adjacent arches. The vertical frame of the structure, characterised by a double layer of L-shaped 5 m long beams (75(50)x50 cm crane and 75(50)x70 cm top) over 35x50 cm 8.8 m tall columns, was fully cast-in-situ.

Typology 3 (Figure 4) dating back to1970 has been the first fully made with precast concrete elements: foundation footings, 30x50 cm section columns 5 m tall, 30(8)x161(40) cm tapered I beams 16 m long and 1.35x0.29 cm section 6 m long TT roof elements.

Typology 4 (Figure 5), built in 1973, is similar to the previous one (the beams are spanning 18 m with deeper section of 1.8 m) except by the roof elements, which are in this case flat with hollow core section.

Typology 5 (Figure 6) dating back 1972 is characterised by a precast portal frame made with 50x50 cm 6 m tall columns and 30(10)x200(80) cm spanning 20.5 m tapered I beams. The roof is composed by 16 m long steel spatial trusses having triangular section.

Typology 6 (Figure 7), built in the early ‘70s, is made with portal frames consisting of 43(15)x56(32) cm cross-shaped section 5 m tall columns, 30(8)x160(40) cm tapered I beams spanning 13.5 m and TT elements spanning 10 m.

Typology 7 (Figure 8) is peculiar: the roof made with IPE 200 steel profiles spanning 10 m is supported with a relevant one-side only overhang of 4 m by a precast portal frame made with 40x60 cm section 5,4 m tall columns and prestressed 26(8)x180(60) cm tapered I beams spanning 26.3 m on one side and by a masonry wall built over rectangular section beams seated over the same columns of the typology 8.

Typology 8 (Figure 9) was built in between the last ‘70s and the early ‘80s with a precast frame structure made with 40x40 cm section 5.2 m tall columns, prestressed 30(8) x 200(60) cm tapered I beams spanning 25 m and 10 m long TT roof elements.

All joints of typologies 3-8 are dry-friction simple supports. Thus, for all these cases the basic assumption of a seismic vulnerability index evaluation relies upon the installation of mechanical connections aimed at restraining all simply supported elements.

Typology 9 (Figure 10) was built in the early ‘60s with clamped portal frames with variable section made by bolting and grouting of 4 segments. The gross maximum height is about 10.5 m at midspan. The inclined upper part of the portal has decreasing depth from the joint with the column (120 cm) to midspan (57 cm). The orthogonal 84 cm deep beams are T-shaped with width of 30(10) cm. The column elements are I-shaped 40(12) x 31 cm with regular stiffeners each 80 cm. The centre to centre distance of the portal frames is 5 m. A secondary beam order made of inverted U-shaped roof elements covers the distance between the portals.

Typology 10 (Figure 11) was also built in the early ‘60s with an arched strut-tie roof system spanning 14.1 m employing I-shaped concrete arch segments connected at the top with bolts and a steel tie per each arch. An indentation is present in the section at the support with the beam. The arches are placed at each 1 m and their distance is covered by masonry blocks with an upper cast-in-situ concrete topping. Beams and columns are the same of typology 9. The column has a clear height of 7.75 m from the pavement. The maximum height of the building is about 10.5 m at midspan of the arches.

Figure 2. Typology 1.

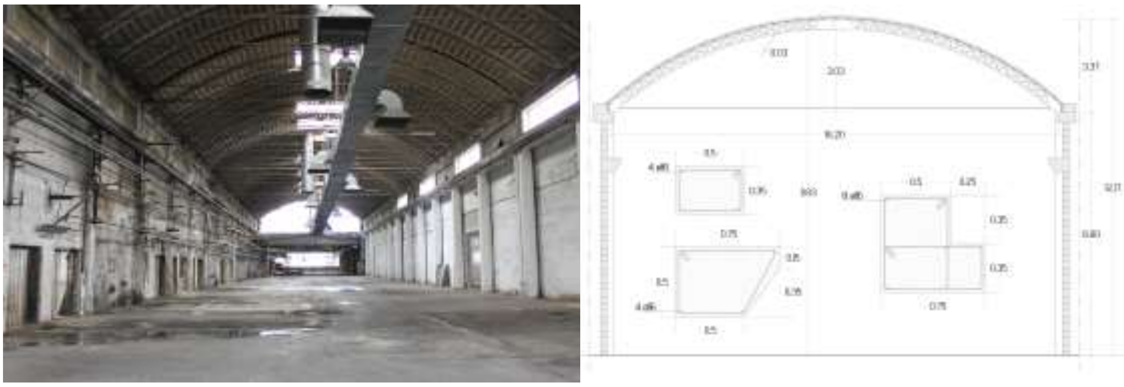


Figure 3. Typology 2.

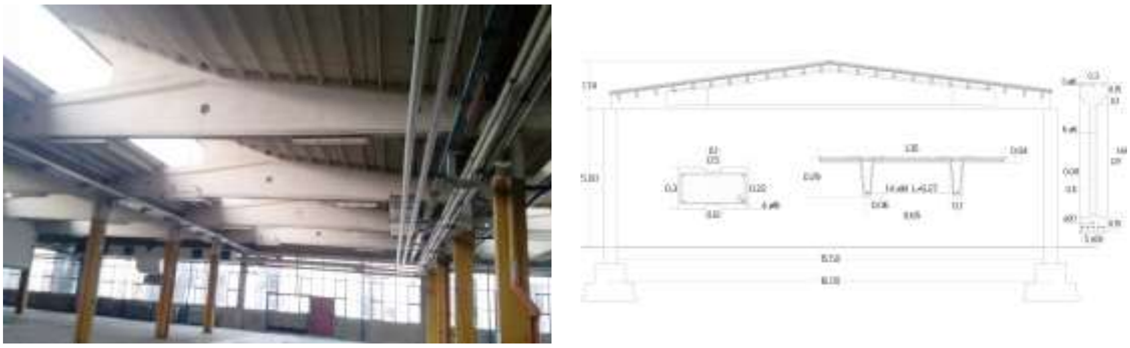


Figure 4. Typology 3.

Figure 5. Typology 4.

Figure 6. Typology 5.

Figure 7. Typology 6.

Figure 8. Typology 7.

Figure 9. Typology 8.

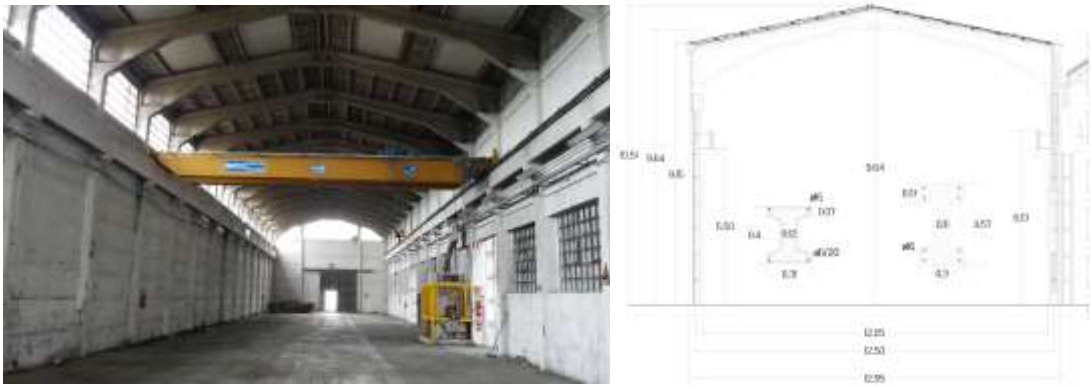


Figure 10. Typology 9.

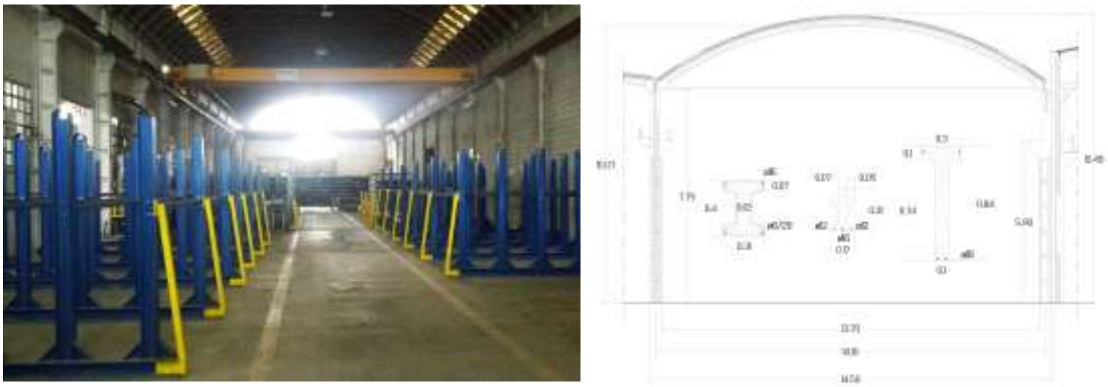


Figure 11. Typology 10.

4 INDEXES OF STRUCTURAL PERFORMANCE

Indexes of structural performances, defined as the ratio of capacity over demand, have been evaluated based on the above-discussed assumptions for each of the following ULS checks: static actions with snow as primary variable load, static actions with wind as primary variable load, seismic combination of actions with main action in each of the two horizontal directions, and fire. Whilst the last verification is expressed through a class of resistance (R) in minutes, where R60 is typically taken as the reference required value, the other checks have to be considered satisfactory when the index is higher than unity. All indexes are collected in Figure 12. The reported indexes are the minimum among the various calculated for all elements of a structural typology. The characteristic live and seismic loads have been evaluated according to the Italian design code (NTC 2008) for the selected area, as hereafter detailed: snow = 1.2 kN/m^2; wind = 0.9 kN/m^2 (pressure plus suction); 0.064g PGA over rocky soil; conventional time-temperature curve for fire. On the basis of a database obtained from previous retrofitting interventions on similar buildings performed by DLC Consulting ltd., the average expected retrofitting costs per square meter of gross surface for the evaluated building typologies have been evaluated, expressed as the mean value of the estimated range only for the sake of brevity. The cost-histograms are reported in Fig. 11 for all typologies. It is worth observing that in some cases for typologies 3-8 costs higher than 0 € have been attributed also to typologies which do not suffer from performance indexes lower than unity. This costs are related to the installation of the mechanical connections to avoid loss of support and/or overturning of the dry-friction simply supported elements, as previously discussed.

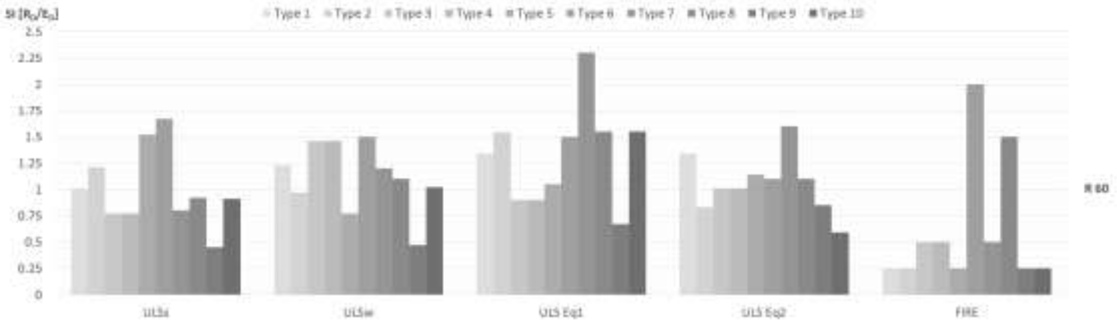


Figure 12. Structural performance indexes per typology.

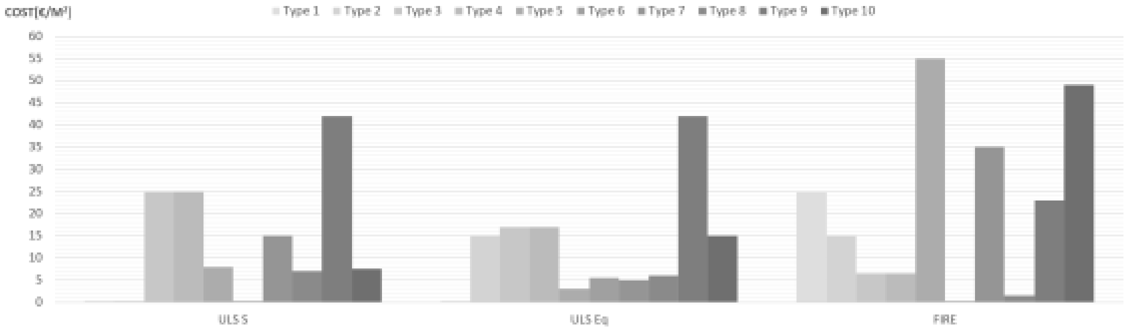


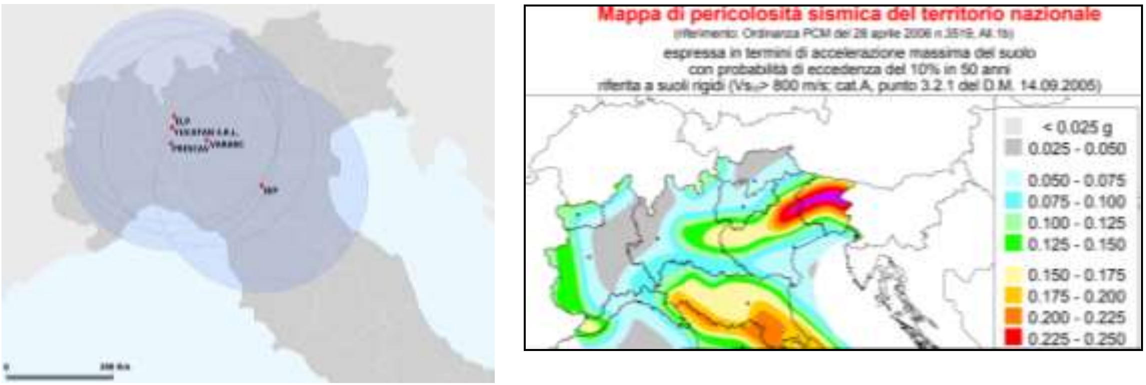
Fig. 13. Estimated mean retrofit costs per typology.

The limit PGAs for the full structural verifications of all the structural typologies are collected in Table 1 (provided the mechanical connection of simply supported unconnected elements are installed). The values vary relevantly, ranging between 0.031g to 0.140g, with a mean value of 0.076g. It can be observed that higher PGA limit values are usually associated to higher column dimensions. By assuming a distribution area not further than 200 km from the known constructors of the investigated typologies, a basin of diffusion of these structures covering most of the Northern Italian peninsula can be guessed as per Figure 14a.

Table 1. PGA limit on rocky soil for all typologies in the two horizontal directions.

Typology	Total spread mass (kg/m^2)	Column size (x,y,z)[m]	PGA limit – dir. x	PGA limit – dir. y
1	416	0.35x0.35x4	0.106g	0.090g
2	580	0.35x0.50x8.80	0.098g	0.053g
3	367	0.3x0.5x5	0.059g	0.065g
4	397	0.3x0.5x5	0.054g	0.060g
5	183	0.5x0.5x6	0.067g	0.073g
6	242	0.43x0.56x5	0.096g	0.070g
7	168	0.7x0.4x5.5	0.140g	0.100g
8	144	0.4x0.4x5.2	0.099g	0.096g
9	524	0.3x0.4x9.64	0.035g	0.044g
10	338	0.3x0.4x7.75	0.081g	0.031g

By observing the corresponding seismic hazard map typically employed for life safety limit state of standard construction as per the Italian code NTC 2008 shown in Figure 14b, it can be recognized that a non-negligible share of the old precast building stock spread over this territory could sustain the design earthquake following the actual standards despite the highlighted structural deficiencies. This observation fosters the need to study in detail each possible retrofit intervention, which could even lead to the avoidance of intervention or to a light one.



(a) *(b)*

Figure 14. (a) Hypothesized diffusion of the analyzed typologies, (b) Italian SLV seismic hazard map.

CONCLUSIONS

The structural details of the 10 analyzed structural typologies, which cover most of the building technology employed for industrial buildings in the period from the late 1940s to the late 1970s in Italy, appear to be very poor according to the actual conception and practice, in terms of both reinforcement detailing of the supporting elements and connection details. Nevertheless, the structural performance indexes show values higher or close to the unity under static load conditions, with the only exception of the typology 9 with moment-resisting portal frames. Even the seismic structural indexes appear to be in line with the low seismic demand of the main diffusion area. This suggests that the retrofitting operations could be oriented towards diffused local strengthening interventions, without the need to dramatically modify the structural system. It has to be recalled that, referring to the seismic load conditions, the results are based on the assumption of an a priori intervention through the installation of mechanical connections where dry-friction simple support connections, which are associated to a very high level of vulnerability, are present. With reference to the fire resistance, the scenario is more critical, with only two structural typologies (6 and 8) not suffering from problems and the majority featuring very low resistance time. Most of the fire problems are related to the presence of exposed steel which is unprotected or thin exposed ribs of concrete elements. For typologies 1, 2 and 10, unprotected steel is present in the tie rods, while the problem is more spread for typologies 5 and 7, where the whole roof is made of exposed steel profiles. The typologies not suffering from any fire problem are made with concrete elements of relatively large thickness. Typologies 1, 2, 3, 4 and 9 are characterized by relatively thin sections of the roof reinforced concrete elements, which also jeopardizes their fire performance. The estimated retrofit costs vary from a minimum of 15 €/m^2 for typology 8 to a maximum of 150 €/m^2 for typology 9. Retrofit costs ranging in between 50 and 90 €/m^2 are found estimated for the other typologies. The results of the presented work are deemed to provide a useful indication to designers who are approaching the problem of the retrofit/requalification of this kind of buildings and structures. It is anyway worth reminding that each real construction needs to be carefully analyzed, through appropriate testing for the characterization of the material properties and detailed survey, in order to soundly verify most of the assumptions that have been introduced in this study.

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