Oct 27th, 9:00 AM - 10:30 AM

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Rehabilitation of early-age precast concrete structures: a feasibility study on an industrial complex progressively built in 1940s-1970s in Northern Italy

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Abstract. Precast concrete industrial buildings have been widely spreading over the Italian territory mostly devoted to industrial development after WWII. The pioneering structures built in the early decades of this period have been designed with criteria which are now obsolete and with lower static actions with respect to the actual standards, moreover disregarding any conception of seismic or fire resistance. As such, they are a potential source of strong vulnerability. With reference to a large industrial complex located in Brianza (Northern Italy) built with successive expansions from the '40s to the '70s, the structural performance indexes associated to static, seismic and fire loads are evaluated for the different classical typologies of that period, including roofs with truss strut-tie systems, restrained arches, and tapered beams. The costs associated to strengthening/retrofitting are also estimated for each typology.

Keywords: Precast structures, Structural retrofit, Historical, Mechanical connections.

Introduction

Precast concrete structures have been widely spreading after WWII over Europe, being Italy among the most innovative and active markets, especially since the autarchic period of the late pre-WWII period, where the lack of steel production facilities and the abundance of raw material to get cement and concrete over the national territory brought the construction industry to focus mostly on concrete solutions. Since then, the evolution of the precast concrete industry has made giant steps forward [1,2], keeping the Italian precast concrete industry a world excellence. The initial post-WWII decades up to the '70s can be defined as the pioneering age of the precast construction industry in Italy. Brilliant ideas and authentic dare were spread over the structural engineers, who kept inventing innovative solutions capable to successively break span records and to respond to a frantic demand, mainly concentrated into the fast-growing national heavy industry.

The precast buildings at that time were designed according to obsolete criteria and lower loads with respect to the actual standards, moreover 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 engineering community.

The present paper shows some excerpts (a more complete overview of the work is available in [3]) from a pilot study conducted on an industrial complex located in the region of Brianza, in Northern Italy, shown in Fig. 1. This industrial complex covering over 30k m^2 has been built with successive expansions from the late '40s to the late '70s, and it represents a sort of catalogue of

the evolution of the precast industrial structures of that period, being composed by many different structures which can be classified into 8 typologies. The aim is therefore to preliminarily investigate the level of vulnerability of the different structures, by deducing structural performance indexes associated to the attainment of the different Ultimate Limit States (ULS) identified by the current regulations.
With the aid of a wide historical archive of the original projects and drawings, and of 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 guessed on the basis of a probabilistic catalogue built over the experimental data on destructive tests performed in those decades [4].
The structural analysis has been performed according to the following assumptions: - a typical modulus of each typology has been modelled through a FEM code [5], neglecting the problem of structural interaction with the adjacent moduli;
- a wide 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, which has been suggested by the visual inspections;
- the uncertain bracing effect of internal or external infill masonry walls is neglected;

Fig. 1. The case study industrial complex

- an elastic structural behaviour (q = 1.5) is assumed even under the seismic load conditions, due to the lack of transverse confinement of the longitudinal rebar associated to small-diameter stirrups spaced more than 200 mm, which jeopardise the potential dissipative sources of the longitudinal rebars;
- the seismic soil classification according to EC8 [6] is C (deep deposits of dense or medium-dense sand, gravel or stiff clay) and the foundations are assumed as perfectly rigid.

Moreover, the structural performance indexes are based on the assumption that all simply supported elements by dry-friction are provided by 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 [7-9], and that this intervention could foster the assumption of a rigid diaphragm behaviour of the roof deck [10,11].

Precast structural typologies

Typology 1 (Fig.2) is the oldest of the complex, dating back to 1948. It is made with a triangular trussed roof with 15x40 cm 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 made with square 35x35 cm 4 m tall columns and T-shaped 60(35)x80 cm 7.5 m long beams is fully cast-in-situ. The roof slab is mixed concrete-masonry.
Typology 2 (Fig.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, is fully cast-in-situ.
Typology 3 (Fig.4) from 1970 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 (Fig. 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 (Fig. 6) from 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 (Fig. 7), built in the early ‘70s, is made with portal frames composed by 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 (Fig. 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 (Fig. 9) has been 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)x200(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.


Fig. 2. Tipology 1


Fig. 3. Tipology 2


Fig. 4. Tipology 3


Fig. 5. Tipology 4


Fig. 6. Tipology 5


Fig. 7. Tipology 6


Fig. 8. Tipology 7


Fig. 9. Tipology 8

Evaluation of the structural performance indexes and expected retrofit cost scenarios

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 with snow as primary load, static with wind as primary load, seismic with main action in each of the two horizontal directions, and fire. Whilst the last verification is expressed in class of resistance (R) in minutes, where R60 is typically taken as the reference required value, the other checks are to be considered satisfactory when the index is higher than unity. All indexes are collected in Fig. 10. The reported indexes are the minimum among the various calculated for all elements of a structural typology. The characteristic live and seismic loads are taken as those suggested by the Italian regulations [12] for the selected site, which are the following: 1.2 kN/m^2 of snow; 0.9 kN/m^2 of wind (pressure plus suction); 0.064g PGA over bedrock; conventional time-temperature curve for fire. On the basis of a database of information obtained from previous retrofitting interventions on similar buildings performed by DLC Consulting, the average expected retrofitting costs spread over one square metre of gross surface of the typology are evaluated, expressed as the mean value of the estimated range only for the sake of brevity. The costs histograms are reported in Fig. 11 for all typologies. To be noted that in some cases for typologies 3-8 costs higher than 0 € are attributed to typologies which do not suffer from performance indexes lower than unity. This cost is 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.

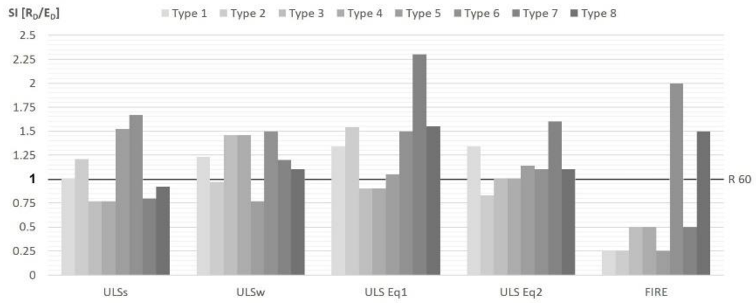

Fig. 10. Structural performance indexes per typology

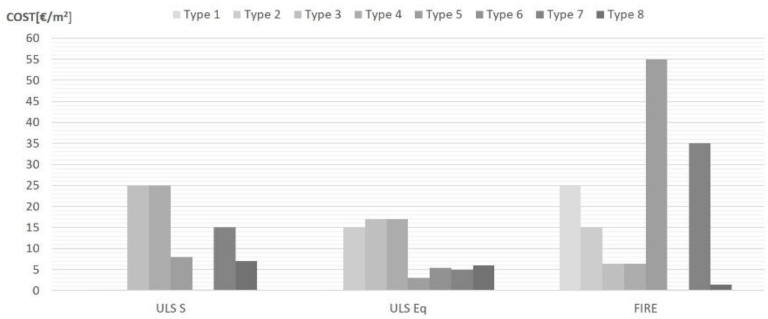

Fig. 11. Estimated mean retrofit costs per typology

Conclusion

Analysing the obtained results, it is noted that the various structural typologies present structural indexes higher than unity for most of both static and seismic load conditions, with exceptions of some values however not lower than 0.75, which suggests that the retrofitting costs can be contained in 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 displaying very low resistance time. Most of the fire problems are related to the presence of exposed steel which is unprotected. For typologies 1 and 2, 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 with exposed steel profiles. The typologies not suffering from any fire problem are made with concrete elements of relatively large thickness. Typologies 1, 2, 3 and 4 are characterised by relatively thin portions of the roof reinforced concrete elements, which also jeopardises their fire performance. Further analyses including destructive and non-destructive testing for the characterisation of the material properties and for the detection of the real position and quality of reinforcement is needed to confirm the results of this feasibility study and to design the retrofit interventions.

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