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STATE OF THE ART OF SEISMIC DESIGN OF RC PRECAST FARM BUILDINGS

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1. Introduction

In rural areas included in seismic zones, earthquake-resistance design criteria must be adopted for every construction, including farm buildings. Therefore the objective of their structural design is the achievement of such a level of safety and functionality that minimizes risks for workers and production also in case of earthquake. At the same time farmers need earthquake-resistant buildings involving the lowest construction costs as possible, in order to maintain their level of competitiveness especially towards other farms located in non-seismic zones.

Seismic classification of Italian territory was recently redefined according to most recent results of studies regarding soil seismicity, and became effective in 2005. This work examines some consequences of such classification, with particular reference to farm buildings with reinforced concrete (RC) structure, since they are widespread in several Italian regions.

1.1 *Seismicity of the territory*

First laws aiming at identifying seismic zones in Italy were promulgated after the earthquake of Messina and Reggio Calabria in 1908 [1]. Since then, until 1980 [4] the classification criterion consisted in adding to the list of seismic zones the areas that had been subjected to strong earthquakes. Territories were classified since 1927 [2] into three different categories of seismicity, according to the gravity of the recorded events and to geological characteristics. Classification criteria changed subsequently to law nr. 64/1974 [3], which required periodic updates of the list of seismic zones, according to solid technical issues. The distinction of such zones in categories of different seismic levels had to be assumed as basis for structural design. Practical effect of such dispositions were produced in the early '80s, when the National Research Council proposed a new map of Italian seismic zones [18], that was adopted as the official one. This classification consisted in three categories (with seismicity increasing from the first to the third one) and a relevant amount of national land was considered as non seismic (Fig. 1).

More recent researches, carried out by the National Seismic Service [8], led to a redefinition of seismic zones, that became official owing to the promulgation of an Ordinance of Civil Protection [5], which introduced some relevant innovations. Firstly the whole national land is presently subdivided into seismic zones, which are categorized into four classes, according to the magnitude of expected earthquakes. It is assumed that 1 is the index of higher intensity and 4 of the lowest one. Moreover the seismic zones 1, 2, 3 have been widely extended in comparison with the previous ones of 1st, 2nd, 3rd category, and in those areas the full respect of seismic design rules is required. Eventually in zones 4, Regions can deliberate whether require or not earthquake-resistant design, which has however to be carried out according to simplified technical rules. New seismic classification was drawn up by the national central Government, and Regions have to update it in time.

A more detailed analysis of the new zoning can be carried out by considering one region of the Italian territory. The Emila-Romagna region is examined as a significant case for the theme in hand because two conditions are met: a large amount of land is destined to agriculture and the reclassification of seismic areas involved most of the territory.

In this region indeed previous seismic territorial classification defined 89 municipalities belonging to 2nd category, while all the other areas were declared non seismic. New classification (Fig. 1) involves a remarkable increase of seismic zones, by introducing 16 more municipalities in the area with level of seismicity 2, and by assigning the level of seismicity 3 to further 214 municipalities. Remaining 22 commons are located in zone 4 [7].

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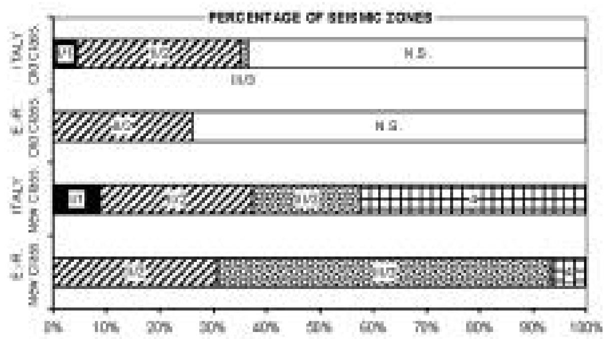

Fig. 1 - Percentage of seismic municipalities (I, II, III: previous seismic categories; 1, 2, 3, 4: actual seismic zones; N.S.: non seismic zones) in Italy and in the Emilia-Romagna (E.-R.) region according to previous and actual zoning.

1.2 *Features of RC farm buildings*

Prefabrication technique allows to produce RC structures suitable for agricultural uses. These constructions are widely employed in Italy, and a large diffusion is registered in the Emilia-Romagna region [19]. Generally such buildings are characterized by constantly spaced sequences of equal-span portals, ceilings and infill elements made up by precast simple or insulated panels, with several chances of finish. Some authors [13] pointed out the effectiveness of using standardized pre-manufactured elements for farm buildings, planned with modular criteria.

RC constructions prove to be suitable to contain different farming activities, according to indoor layout organization. Garaging of faming machines and toolshed represent typical uses of the considered typology of structure. For this purpose special internal subdivisions are generally not required, since flexible use of the space is preferred. At the meantime a suitable paving is required, for sustaining distributed, concentrate, static and dynamic loads, originated by machinery storing.

A further class of building use involves zootechnics: precast structures are fit to be employed for livestock housing or facilities. In this case an appropriate distribution of openings is required to satisfy necessities of accessibility, natural ventilation and illuminance. High flexibility in erecting perimetric infill panels allows to consider the possibility of re-using the structure, once productive cycle of breeding would have expired.

Finally, precast concrete buildings can be used as storehouses of agricultural products, also by introducing, if necessary, internal compartments fit to contain cold stores. Within the purview of this employment it is to be considered that ensiling use requires a provided structural typology. It is in general characterized by the collocation of the infill panels along column inner line, in order to ease wall strength to lateral forces originated by contained materials.

With regard to aesthetic features and the landscape impact, precast structures are suitable for a wide range of finishes and chromatic and material solutions, also in case they are designed with seismic-resistant criteria.

1.3 *Aims of the study*

The study aims at deepening the theme of seismic design of farm buildings. The main objective is the analysis of the state of art of seismic design requirements of precast concrete buildings in rural areas. In particular, one-storey frame structures are considered and their mechanical performances under earthquake are evaluated.

Moreover the study aims at estimating the difference in structural safety and costs between a seismic-resistant RC farm building and an analogue one non seismically designed. Aesthetic-perceptive requirements of the considered construction typologies are also taken into account, with reference to the landscape context.

2. Materials and methods

The exposed themes were in-depth analyzed through the identification and development of two study cases. They consist in the structural design of two farm buildings with precast RC structure, located in an area which was non seismic according to previous classification of the territory and became seismic due to the new zoning. Therefore the first case study is a structure designed without seismic requirements, while the second one consists in a building with the same architectonic features, designed according to aseismic provisions.

Safety evaluation of the designed buildings were performed, also through non linear structural analyses. Construction costs were assessed in the two cases and compared.

As already mentioned, Emilia-Romagna region showed to be a significant context for the present study. In this region the municipality of Castel San Pietro Terme (province of Bologna) is identified for the location of the study cases. The seismic classification of this area was indeed changed from non seismic to seismic of class 2. Moreover the greater part of the considered territory is farmland and prefabricated buildings made of reinforced concrete are largely employed for agricultural uses.

A rural building with current dimensions was examined in the two cases. It measures 16 x 35 m in plan, 5 m tall at eave, with a 28% incline gable roof. A precast RC structure was defined, consisting of equal-spaced portals, with columns constrained at their base by a rigid connection to the foundation plinth, and RC gable beams (Fig. 2).

2.1 *Structure layout*

The structure was idealized as a portal tall *h*, with span measuring *l*, having fixed joints at column bases and hinge connections between the top of every column and the corresponding girder end (Fig. 3). Roof beam was considered having infinite axial stiffness.

The structure is subjected to distributed vertical actions, due to self weights and dead loads (q_p), and to snow loads (q_s). Wind action was accounted for as horizontal distributed variable loads (q_{w1} and q_{w2}), with intensity increasing from ground level to the top of the building. Characteristic values of distributed loads are:

q_p = 27.19 kN/m;
q_s = 8.97 kN/m;
q_{w1} = 3.92 kN/m at ground level;
q_{w1} = 4.21 kN/m at the top of the building;
q_{w2} = 2.54 kN/m at ground level;
q_{w2} = 2.72 kN/m at the top of the building.

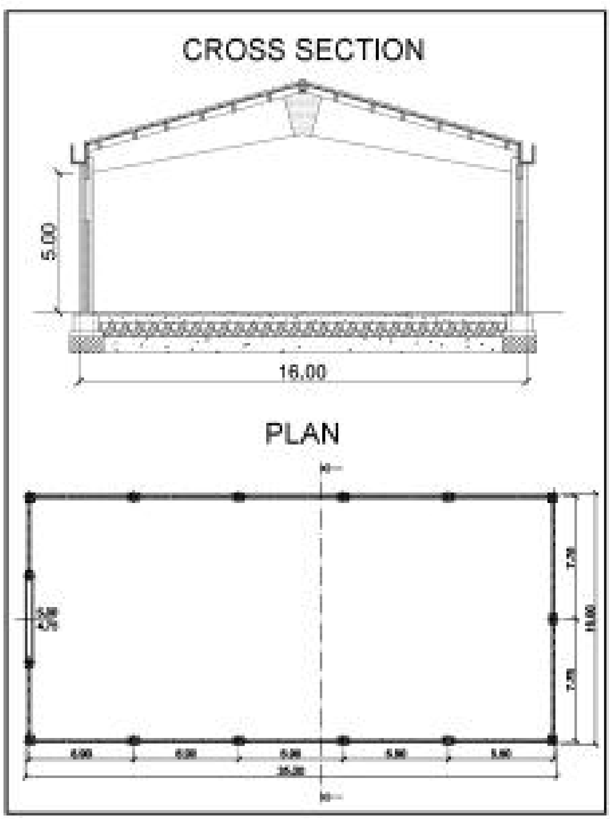

Fig. 2 - Layout of the studied building: plan and cross section.

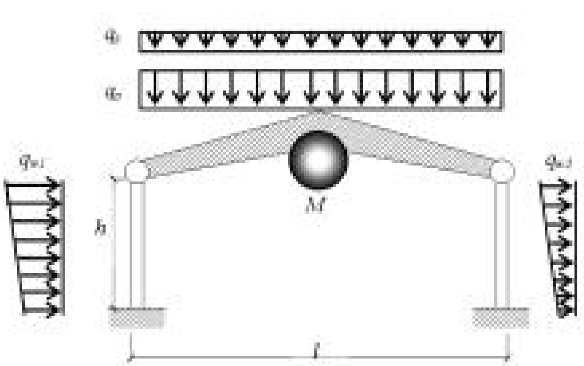

Fig. 3 - Structure layout adopted for static and dynamic analyses.

Structural masses were idealized as a lumped mass *M* allocated in the girder midpoint, equally distributed over the two pillars.

European codes for structure design were followed, namely the Eurocode 1 (EC1) [9] as for design load definition and combination, the Eurocode 2 (EC2) [10] as for concrete structure design rules and the Eurocode 8 (EC8) [11] as for earthquake resistant design criteria.

Structure design considered the employment of C40/50 class concrete for precast elements, C25/30 class concrete for foundation structure to be built on site, reinforcements made of Fe44K steel, and environmental conditions involving the RC structure belonging to 5-a class, (lightly aggressive chemical ambient), which implies that cover concrete thickness has to be not smaller than 25 mm.

2.2 *Non seismic design of the structure*

Non seismic design of the structure took into account permanent loads due to structural self-weight and dead loads, and variable loads due to snow and wind actions.

Structure verification at Ultimate Limit State (ULS) in the non seismic case required the following load combination:

$$\sum_{j\geq 1}\gamma_{Gj}G_{kj} + \gamma_{Q1}Q_{k1} + \sum_{i\geq 1}\gamma_{Qi}\psi_{0i}Q_{ki} \qquad (1)$$

γ_{Gj} indicates the safety factor for generic dead load G_{kj}, γ_{Q1} and γ_{Qi} indicate safety factors for the characteristic values of live loads Q_{k1} and Q_{ki}, and ψ_{Qi} are the combination factors. In the considered case, G_{kj} must be factorized by 1.35, whilst Q_{k1} and Q_{ki} must be amplified 1.5 times, when considering only the dominant variable action and 1.35 times when taking into account both snow and wind actions.

Once design combinations of axial force and bending moment had been identified, the column was dimensioned. According to current building practice, a square section was adopted, with side 0.40 m wide and with four reinforcement rods ϕ 20. Transverse reinforcements consists in hoops ϕ 8 with interfit of 0.20 m which is reduced to 0.10 m near to the two ends of the column, for a length equal to its section side. Dimensioning and displacement of reinforcement bars satisfy current code regulations for non seismic design (Eurocode 2, part 1-1, par. 5.4.1): the cross sectional area of the entire longitudinal reinforcement respects the minimum (0.3% of the concrete cross sectional area A_c) and the maximum requirement (8% of A_c). Hoop spacing respects the upper limit of 0.24 m, which is reduced to 0.14 m in proximity to column ends. Safety verifications at ultimate limit state were carried out by comparing design axial force N_{Sd} and bending moment M_{Sd} with corresponding resistances. As the code requires, two load combinations were considered:

– maximum vertical loads (permanent and snow loads) and wind, corresponding to:
N_{Sd} = 468 kN and M_{Sd} = 96.8 kN m;
– minimum vertical loads (only permanent loads) and wind, corresponding to:
N_{Sd} = 368 kN and M_{Sd} = 103.4 kN m.

Respective resistances were determined as the points (N_{Rd}, M_{Rd}) of the boundary of the resistance domain of column section, where:
$M_{Rd} / N_{Rd} = M_{Sd} / N_{Sd}$.

Their values are N_{Rd} = 1073 kN and M_{Rd} = 222 kN m for the first combination, and N_{Rd} = 620.2 kN and M_{Rd} = 174.3 kN m for the second one.

The gable beam was designed according to Eurocode 2 with reference to the above-mentioned loads. Dimensions of the beam cross sections and reinforcements are illustrated in Fig. 4.

Foundation structure design was carried out on the hypothesis that base ground is made of deposits of loose-to-medium cohesionless soil, with or without some soft cohesive layers, or of predominantly soft-to-firm cohesive soil. Every column is founded on a square-based plinth with side measuring 2.00 m and thickness of 0.40 m, through a precast trap, as it is shown in Fig. 5. The square-based plinth is reinforced with 12 ϕ 12 on both top and bottom side along the two principal horizontal directions. Isolated foundations are cheaper than continuous ones, as grade beams or slabs, and their adoption is not recommended in seismic zones.

2.3 *Seismic design of the building*

According to seismic classification of the construction site, the structure was designed to allow the following performances:

Class	Buildings	Factor γ_I
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc	0.8
II	Ordinary buildings, not belonging in the other categories.	1.0
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.	1.2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1.4

TABLE 1 - Importance classes for buildings, according to European codes.

– to withstand a design seismic action expressed in terms of a reference period of 475 years, without local or global collapse;
– to withstand a design seismic action with reference period of 95 years, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself.

Design seismic action is quantified according to the category of local seismic zone, to the building importance class and to ground conditions. Structures are classified in four importance classes, depending on the consequences of their possible collapse for human life and economic activities, and on their importance for

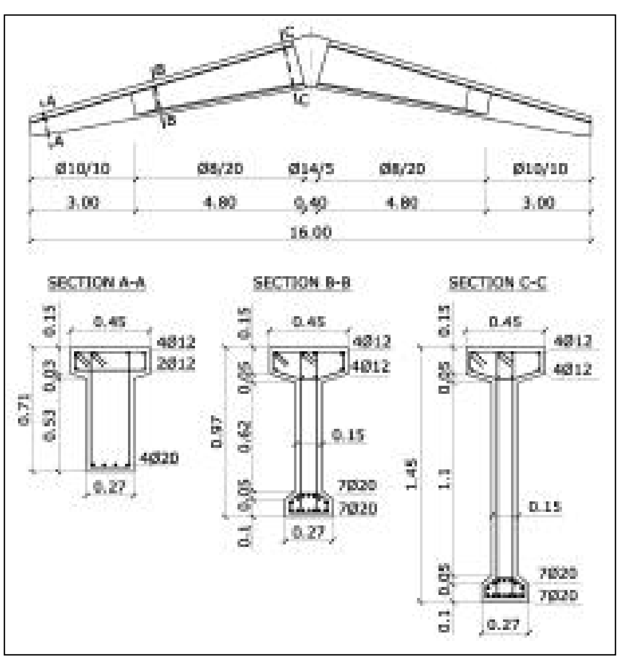

Fig. 4 - Elevation and cross sections of the gable beam.

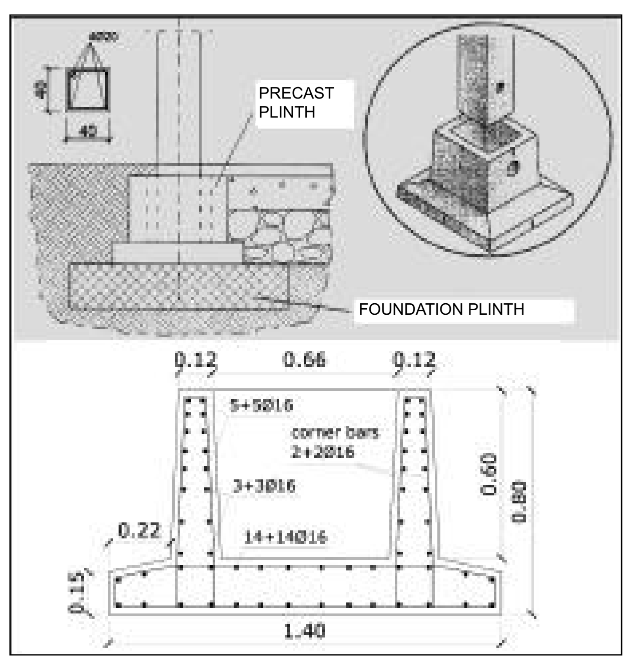

Fig. 5 - Detail of column foundation: cross section of the pillar (top on the left); sections, dimensions and axonometric projection of the precast plinth.

public safety and civil protection in the immediate post-earthquake period (see tab. 1). Every class is characterized by a value of the importance factor γ_I. Such factor shall be applied to the ground reference acceleration a_{gR}, which amounts to 0.25 g in seismic zone 2.

Structural mass M of the portal is expressed through the following equation:

$$M = \sum_{i \geq 1} G_{kj} + \sum_{i \geq 1} \psi_{Ei} \cdot Q_{ki} \qquad (2)$$

where ψ_{Ei} is the reduction factor for variable actions, as it is defined in EC1, being 0 for snow actions on roof; hence it results: M = 54 500 kg.

The preliminary dimensioning of the structure led to the adoption of columns with 0.6 m side square section and of roof beams with the same characteristics as those of the non seismic case. According to these features the horizontal stiffness of the portal was computed as follows:

$$K = 2 \cdot 3 \frac{EJ}{h^3} \qquad (3)$$

E indicates concrete Young modulus for dynamic actions and J the momentum of inertia of a single column section. It results K=13.63 kN/mm.

Dynamic behaviour of the structure in the transversal direction (Fig. 3) corresponds to a Single-Degree-of-Freedom (SDOF) system, so that its natural period was computed through the relation:

$$T = 2\pi \sqrt{\frac{M}{K}} \qquad (4)$$

which led to the value: T = 0.40 s.

Eqn. (2) was applied also to determine the fundamental period of the structure designed without seismic-resistant requirements. It resulted equal to 0.90 s.

In longitudinal direction, dynamic behaviour can be considered the same as that of a one-floor frame having columns with fixed base and top ends connected by trusses representing roof panels.

According to this structural layout (Fig. 6), natural period is the same as that computed for cross direction of the building, because masses ascribed to each column and its translational stiffness are the same.

ULS design employed an acceleration response spectrum Sa_d (Fig. 7) equal to elastic response spectrum divided by the behaviour factor q, which provides for structural ductility:

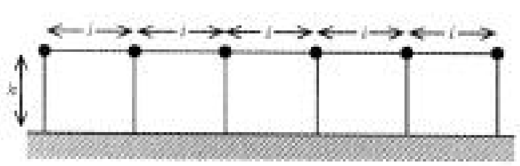

Fig. 6 - Layout of the structure in longitudinal direction.

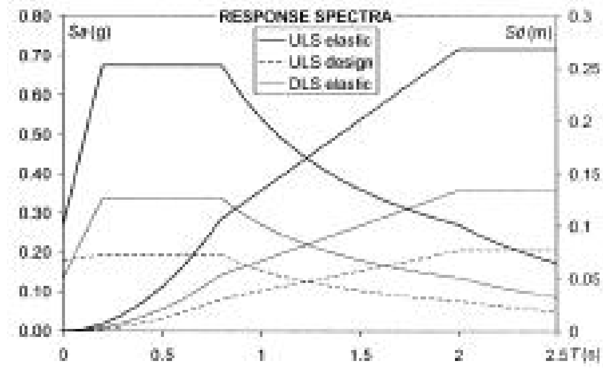

Fig. 7 - Acceleration (Sa) and displacement (Sd) response spectra: Ultimate Limit State (ULS) elastic spectra, Damage Limitation State (DLS) spectra, and inelastic ULS design spectra for q = 3.5.

$$Sa_d = a_g \cdot S \cdot \frac{2.5}{q} \qquad (5)$$

The term a_g represents design ground acceeleration ($a_g = \gamma_I \, a_{gR}$) and S is the soil factor, which is equal to 1.35 in this case (soil type D). The value of q is assumed to be 3.5, according to Italian code requirement for structures with isostatic columns [6].

Lateral force method of analysis allowed to evaluate seismic response of the structure, by adopting the combination of actions defined in Eurocode 1 for seismic design condition, given by the following expression:

$$\sum_{j \geq 1} G_{kj} + \sum_{i \geq 1} \psi_{2i} \cdot Q_{ki} + \gamma_I A_{Ed} \qquad (6)$$

where ψ_{2i} represents the combination factor of variable actions, and it is equal to 0 for snow and wind loads, whilst A_{Ed} represents the seismic design action.

According to [5] and [11] the horizontal components of the seismic action were taken as acting simultaneously. The action effects due to the such combination were computed as the sum of the effects due to the application of the seismic action along one horizontal axis of the structure and 0.30 times the effects of the same seismic action along the orthogonal horizontal axis.

Each column of the building resulted subjected to the combination of design actions N_{Sd} = 273 kN, M_{Sd} = 288.6 kN m, which had to be combined with the bending moment 0.30 M_{Sd} in the orthogonal direction. It led to design the longitudinal reinforcement consisting of 12 ϕ 20 (see Fig. 8). Safety verification required the definition of bending resistance M_{Rd} of the column (it is the same along the two main horizontal axes) corresponding to N_{Sd} [15]. Condition of safety verification are given by the following relation:

$$\left(\frac{M_{Sd}}{M_{Rd}}\right)^{1.5} + \left(\frac{0.30 M_{Sd}}{M_{Rd}}\right)^{1.5} < 1 \qquad (7)$$

As M_{Rd} =340 kN the condition (7) is satisfied.

Dimensioning and displacement of reinforcement bars satisfy current code regulations for seismic design (par. 5.5.3 in [5], as modified by [6]): the cross sectional area of the entire longitudinal reinforcement respects the minimum (0.01 A_c) and the maximum requirement (0.04 A_c).

Transverse reinforcement consists of ϕ 8 square hoops engaging the longitudinal corner bars, and of ϕ 8 rectangular hoops engaging the remaining reinforcement bars (Fig. 8). The spacing of the hoops is 0.075 m within the critical region of 1 m at the base of the column and 0.15 m in the upper 0.6 m zone, while in the central portion it is equal to 0.2 m. Such transversal reinforcement satisfies both current code regulations [5, 6] and ductility requirements provided by Eurocode 8 for seismic zones, expressed by the following relation:

$$\alpha\omega_{wd} \geq 30\mu_{\varphi}\nu_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_0} - 0{,}035 \qquad (8)$$

α represents the confinement effectiveness factor; ω_{wd} the mechanical volumetric ratio of confining hoops within the critical region; μ_φ the required value of curvature ductility factor; ν_d the normalized design axial force; $\varepsilon_{sy,d}$ the design value of tension steel strain at yield and b_c and b_0 respectively the gross cross-sectional width and the width of confined core.

Damage Limitation State (DLS) verification considered a seismic action with a return period of 95 years, corresponding to the 50% of the elastic response spectrum adopted for the ULS design. Structural stiffness used for displacement evaluation was that of post-cracking concrete, conventionally fixed equal to 50% of the uncracked elastic stiffness of the cross section. Maximum value of inter-storey drift was computed according to the displacement response spectrum for DLS (Fig. 7). It resulted equal to 4.9‰, smaller than the limit of 5‰ established by the code for buildings having non-structural elements of brittle materials attached to the structure. Displacement at DLS for the non seismically designed structure resulted 15‰, therefore far beyond the serviceability limit.

According to the structural layout adopted, the beam has the function of distributing seismic actions to the columns. Therefore safety verifications of the beam were carried out considering the combination of axial force caused by seismic action and flexural moment due to gravity loads, according to Eqn. 6. Reinforcements showed in Fig. 4 resulted partly inadequate for the design in seismic condition: in this case, in the upper flange of the beam 6+6 ϕ 20 steel bars are required (instead of the bars of the upper flange reported in Fig. 4, sections A-A, B-B, C-C) in order to satisfy code provisions about minimum and maximum reinforcement ratio (par 5.5.2.2 in [5], as modified by [6]).

The beam-column joints were dimensioned according to capacity design criteria, i.e. they shall have shear resistance not smaller than the horizontal force

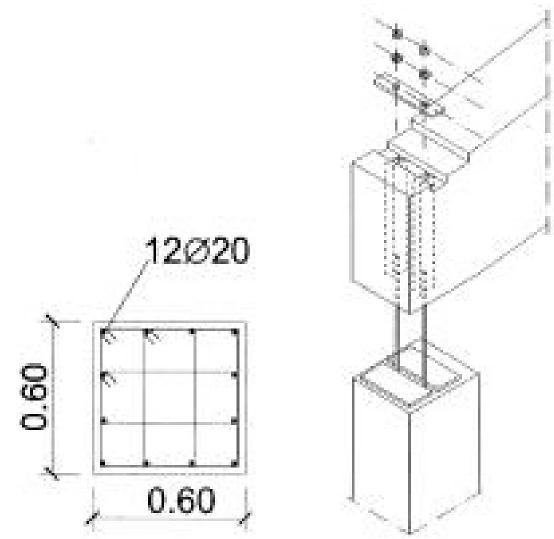

Fig. 8 - Cross section of the column and detail of beam-column joint for seismic design.

inducing yield flexural moment at column base multiplied by the overstrength factor γ_{Rd} = 1.35. Such connection can be carried out through a couple of ϕ 20 steel bars of 6.8 class, fixed to the column top end and allocated in a couple of slots made in the beam end (Fig. 8).

Eventually seismic design criteria led to define a foundation system different from that considered in the non seismic case. Foundations in seismic areas shall indeed withstand horizontal actions induced by ground motion, contrast consequent differential displacement, and perform enough strength to uniformly distribute dynamic actions over the ground. Therefore the project provided for precast plinths fixed to grade beams designed along the perimeter of the building, with transverse RC connection beams below pavement. Foundation dimensioning led to adopt precast plinths with walls 0.90 m tall and 0.20 m thick at the base and 0.15 m thick at the top. Reinforcement consists in 8 + 8 ϕ 16 horizontal bars in the walls and 2 + 2 ϕ 16 vertical bars in each corner. Perimetric grade beams have rectangular cross section, 1.20 m wide and 0.60 m thick, reinforced by 10+10 ϕ 16 longitudinal bars and ϕ 8 double hoops with spacing of 0.30 m. Transverse connection beams are reinforced by 5+5 ϕ 16 longitudinal bars and ϕ 8 hoops with spacing of 0.30 m. Longitudinal reinforcement of perimetric and transverse foundation beams corresponds respectively to 0.27% and 0.62% of the cross sectional area both in the bottom and in the top. It resulted therefore greater than the minimum of 0.2% prescribed by current regulations (see [5], par. 5.4.7.1).

2.4 *Non linear analyses*

The considered structures were idealized with a fiber model [17]: the critical sections of the columns were discretized into an adequate number of inelastic fibres, reproducing mechanical behaviour of confined concrete, unconfined concrete and steel bars.

Then pushover analyses were performed. As it is known, they are non-linear static analyses carried out under conditions of constant gravity loads and monotonically increasing horizontal loads [14]. They were applied to verify the structural performance and to estimate the plastic mechanisms and the distribution of damage. In particular the considered structures were idealized as SDOF systems (see Fig. 3), with loads applied to the top of the columns. Collapse of the structure is assumed to correspond to failure of confined concrete. Expected result was the comparison of the "capacity curves" – the relation between base shear force and the control displacement – in the two design cases.

3. Results and discussion

The pursued study allowed to define two structures suitable to satisfy performance requirements of technical codes respectively in non seismic and in seismic areas. Architectonic features are the same in the two cases.

3.1 *Structural performances*

Results of pushover analyses are illustrated in Fig. 9 in terms of capacity curves correlating the lateral force *F* normalized to vertical load *N* ($\eta=F/N$), and the lateral roof displacement *d* normalized to column height *h* ($\delta=d/h$). The curves provide a visualization of the non linear inelastic behaviour of the two considered structures under horizontal loads. It is worth to point out that in this kind of structures, idealized as SDOF systems, such visualization results highly accurate, as the adopted force distribution represents correctly the distribution of seismic actions. On the contrary, in Multi Degreee-of-Freedom (MDOF) systems (e.g. multi-storey structures) the research of the most appropriate force distribution is currently in progress, and requires complex analytical procedures [16], which can lead to remarkable approximations mainly in case of asymmetric buildings [12]. Therefore the adopted method appears particularly suitable for RC farm buildings, as they usually consist in symmetric one-storey structures.

Yielding forces were computed in the two cases: in the seismic one it is $\eta_{e,s}$=0.37; in the non seismic is $\eta_{e,n}$=0.05. Results show that the limit elastic displacement for seismic design ($\delta_{e,s}$) is almost 2.5%, whilst the one in the non seismic case ($\delta_{e,n}$) is only 0.5%. Furthermore the ultimate displacement for seismic design ($\delta_{u,s}$) is 7.1% , whilst in the non seismic design the ultimate displacement ($\delta_{u,n}$) is 4.3%.

Besides expressing significantly different values of the ultimate displacement, capacity curves give also an indication of the quantity of energy that the structural systems can absorb, partly transforming it into elastic displacement and partly dissipating it through inelastic deformations. The area subtended by the capacity curve represents indeed the strain work of the structure: it can be easily observed that the work required to lead the seismically designed structure to collapse is nearly nine times the correspondent work in case of non seismic design. This ratio can hence be assumed as a synthetic indication of safety increase towards seismic action.

Following code provisions, the capacity curves are idealized as elasto-perfectly plastic force-displacement relationships, plotted in Fig. 9 as dotted lines. Then ULS demand curve is reported and it meets the elastic branch of the idealized capacity curve of the seismically designed structure at inter-storey drift of 3% (performance point).

In the non seismic case displacement demand results almost 5%, as it corresponds to the intersection

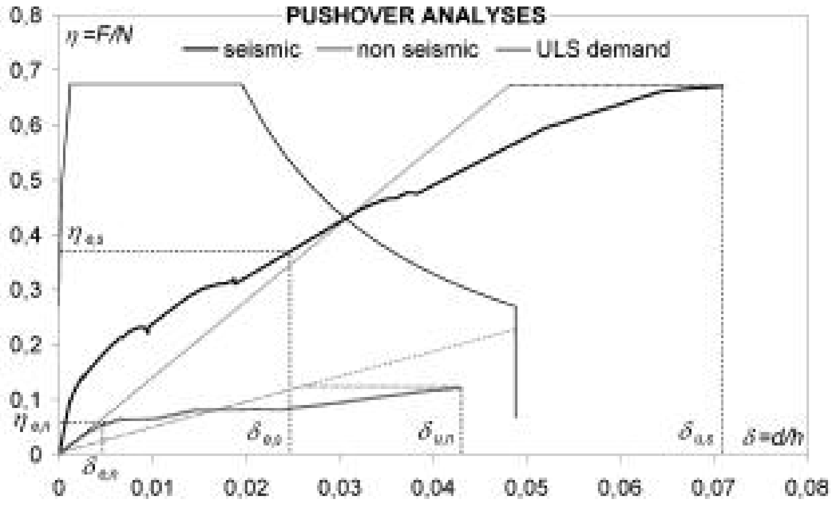

Fig. 9 - Capacity curves of the two designed structures and demand curve.

between the elastic branch of the idealized capacity curve and the ULS demand curve. Such demand exceeds the ultimate displacement of the structure, so this structure has to be considered unsafe in a seismic area of the considered category.

ULS seismic demand was also taken into account for a seismic zone of class 3, therefore with lower intensity, in order to assess the level of safety of the two structure in this case. Demand displacement of the antiseismic structure resulted 2.4%, i.e. smaller than the elastic limit displacement and thus indicative of operational conditions. Demand displacement of the non seismic-resistant structure resulted 4.1%, corresponding to a condition near to collapse.

3.2 *Construction costs*

Construction costs were estimated under the two considered design hypothesis and their values, subdivided into primary items, are illustrated in Fig. 10, where it can be distinguished that main increases involve the foundation system and the RC structure.

In particular the differences in the costs of foundation systems are given by the differing quantity of concrete and steel in the two cases. In the non seismic case foundations consist in isolated plinths, which require on the whole 24 m^3 of structural concrete, 9 m^3 of oversite concrete, and 1440 kg of steel reinforcements. In the sismically designed structure, foundations consist in two longitudinal and six transverse grade beams, requiring on the whole 86 m^3 of structural concrete, 15 m^3 of oversite concrete, and 4920 kg of steel reinforcements Foundation cost in the seismic case results more than three times the corresponding cost in the non seismic case.

In both cases a general dig 0.50 m deep over the whole building area is needed for French drain and pavement. Additional digs for the foundations amount to 65 m^3 in case of isolated plinths and 140 m^3 in case of grade beams. Thus the relative difference in cost of digs between the second and the first case is 36% therefore remarkably smaller than the relative difference between costs of foundations.

As for the RC precast structure, in both cases it is

Construction costs	ORDINARY MODULE, C_M	TERMINAL MODULES, C_T
non seismic	€ 19 700	€ 38 000
seismic	€ 23 000	€ 44 300

TABLE 2 - Costs of the two typologies of modules of the farm building in case of seismic or non seismic design.

formed by 15 plinths, 15 columns, 6 gable beams, 563 m^2 of double tee units for roofing, 70 m of eaves beams, and 33 m of front cornices. In case of seismic design, structure costs are computed as 10% greater than those of non seismic case, as it resulted from estimates provided by construction companies. Such increment is due to the greater quantity of materials required and by the more complex technology of joints between structural elements.

Seismic resistant building engages additional costs corresponding to 16% of the global expense referred to non-seismic designed construction. As the envisaged buildings have modular structures, the cost of each module can be computed in order to assess the cost of a farm buildings consisting in a different number of the same basic structural module. Thus two typologies of modules were considered:

– "ordinary module", having width *l* and length *i* (see Fig. 3 and Fig. 6), including two columns and a beam, with the corresponding cladding and roofing;
– terminal modules, including the two fronts of the building and the corresponding influence area of roofing and cladding.

The respective costs, reported in tab. 2, allow to quantify the global cost of a farm building having a different number of modules from the considered one.

Being *n* the number of portals of the structure, a building consists of (*n* -2) ordinary modules and the terminal modules and its global cost is:

$$C = (n - 2)\, C_M + C_T \qquad (8)$$

Constructions costs have been computed in case of both non seismic and seismic design for *n* varying from 3 to 10, corresponding to building areas of 230 ÷

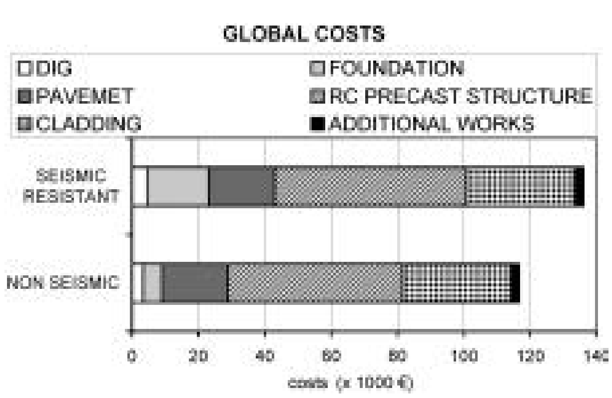

Fig. 10 - Comparison between construction costs for non-seismic and seismic-resistant design.

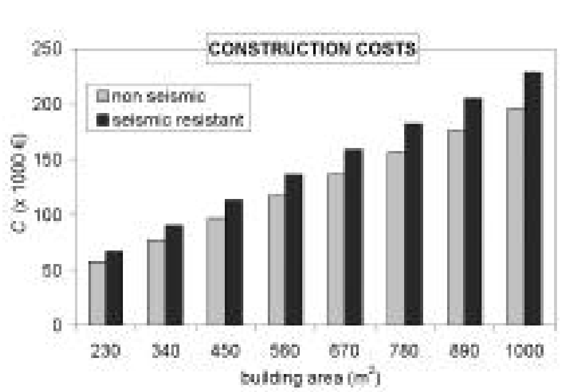

Fig. 11 - Constructions costs in case of non seismic and seismic design (histograms) for different building areas; relative differences between the two cases.

1000 m^2. Such dimensions of farm buildings resulted particularly widespread in the considered area [20]. Resulting costs *C*, illustrated in Fig. 11, show a relative difference between costs in the two cases, which results 17% for building area of 230 m^2 and 16% in all other cases.

3.3 *Exterior finishing*

Structural layout of seismic resistant building is compatible with different chromatic and material solutions of exterior finish, because earthquake-resistant structural elements don't imply aesthetic restraints in comparison with structures designed only to sustain vertical and wind loads. Two possible surface treatments of exterior walls can be adopted in order to obtain landscape integration of the construction (Fig. 12): plastering with painting of traditional building colours, or face brick exterior surface, which reminds another local agricultural construction tradition and offers good performances of weather proofing. Roofing can be made of pantiles, in accordance with visual features of the landscape context.

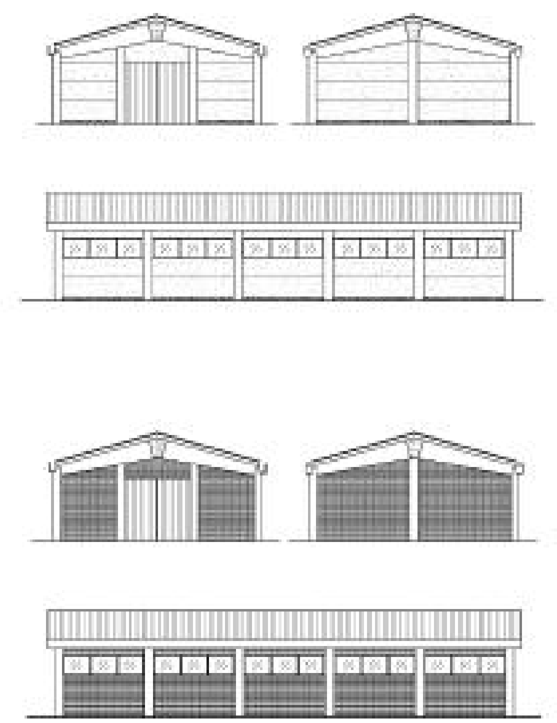

Fig. 12 - Building elevations showing two different material solutions for cladding: plaster walls and face bricks.

4. Conclusions

Seismic design of rural buildings implies the adequate dimensioning of structural elements and the coherent definition of construction details. European codes provide for design criteria to obtain the prescribed safety level of the building. The study focused the attention on RC precast structures that are largely widespread in rural areas of an Italian region. That construction typology was considered and two study cases were designed respectively in non seismic and in seismic conditions. Features and performances of the two designed structures were compared.

The study analyzed economic sustainability of an earthquake-resistant farm building, pointing out that cost increases of 16% with respect to the non seismic case. Such investment allows farmers to have buildings that can withstand a seismic action with high probability to occur during building venture life without damage and associated limitations of use.

Pushover analyses were performed adopting fiber models for both the considered cases. Expected performances of the seismic-resistant structure at ULS were confirmed. On the contrary results of the non seismically designed structure pointed out that its collapse is expected under an earthquake corresponding to the ULS design action, and that it would not satisfy serviceability requirements under DLS seismic conditions. The adopted method of analysis is suitable for comparing performances of different structures: it can be usefully applied in further research developments to assess seismic behavior of a wider sample of agricultural structures.

Finally, the theme of seismic design of rural buildings should be developed together with the identification of specific design solutions for their optimal landscape integration. This item results interesting also with reference to latest evolutions of seismic codes and the consequent availability of few studies about such issue.

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SUMMARY

Structural design of rural buildings deals with several requirements, among which seismic resistance. In this study the state of the art of Italian seismic zoning and its possible consequences over the design of rural buildings are analyzed. Study cases are proposed referring to a municipality of Emilia-Romagna region, regarding precast concrete structures, that are widely employed in farms. Linear and non linear criteria are adopted for modelling seismic-resistant elements. Pushover analyses are performed to verify the designed structures. Results are expressed in terms of parameters quantifying structural reliability and construction costs.

Keywords: seismic design code, design criteria, rural precast structures, economic evaluation.