# Seismic Assessment of RC Existing Irregular Buildings

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**SUMMARY**
In the paper the seismic assessment of a set of reinforced concrete framed structures representative of real existing buildings designed only to vertical loads has been carried out. In particular, structural types having different positions of the stiff staircase structure which determine in-plan irregularity have been studied (3 cases: NS, CS and ES). Further, structural types having different number of storeys (2 cases: 2 and 4), presence and position of masonry infills (3 cases: bare, infilled and pilotis frames) and concrete strength (3 cases: 10, 18 and 28 MPa) have been considered. Non-linear static analyses according to the Italian seismic code on 3D models designed taking into account the presence of staircase structure and of masonry infills have been performed. The results show the remarkable role of in-plan irregularity and of masonry infills on the seismic performance of existing buildings.

*Keywords: Reinforced Concrete, existing buildings, in-plan irregularity, push-over, masonry infills*

## 1. INTRODUCTION

Plan asymmetric distribution of lateral load-resisting elements' stiffness or of floors' mass characterize in-plan irregular buildings. When subjected to horizontal actions, in-plan irregular structures show a torsional coupling response, i.e. floor rotations in addition to translations, which produces a non-uniform demand in the lateral resisting elements. In-plan irregularity is one of the most frequent causes of severe damage and failure of Reinforced Concrete (RC) structures, as shown by past earthquakes (e.g. L'Aquila 2009).
Modern seismic codes, e.g. NTC08 Italian code (D.M. 14.01.2008) and Eurocode EC8-part 1 (CEN, 2003), address earthquake-resistant design to some guiding principles, among them structural simplicity, uniformity, symmetry and redundancy. Concerning uniformity, EC8 states that "*Uniformity in plan is characterised by an even distribution of the structural elements which allows short and direct transmission of the inertia forces created in the distributed masses of the building. … Uniformity in the development of the structure along the height of the building is also important, since it tends to eliminate the occurrence of sensitive zones where concentrations of stress or large ductility demands might prematurely cause collapse.*", further specifying that "*The use of evenly distributed structural elements increases redundancy and allows a more favourable redistribution of action effects and widespread energy dissipation across the entire structure.*". As a matter of fact, conceptual design should be addressed towards simple structures having a symmetrical in-plan distribution of both lateral stiffness and mass with respect to two orthogonal axes, and uniformity in development of resisting elements along the height of buildings. Besides, codes recognize that a non-uniform arrangement of infills in RC framed buildings can determine an irregular response. In the assessment of existing structures (CEN, 2005) compliance with the regularity criteria provided for new buildings (EC8-1) needs to be evaluated in identifying the structural system; further, lower values of the behaviour factor $q$ in case of irregular structures are also suggested.
In the past the performances of in-plan irregular buildings have been widely studied (Paulay, 1997; Fajfar, 2000; Bosco et al., 2008; Bhatt and Bento, 2011) and the influence of key factors on their

seismic response has been analysed.
In the elastic range the seismic response mainly depends on the eccentricity between the centre of stiffness and the centre of mass, and on the ratio between the uncoupled translational and torsional periods (Bosco et al., 2008). Moreover, as a consequence of the larger contribution of the higher modes in case of irregular buildings, also the spectral shape can influence seismic response (Fajfar et al., 2005).
Inelastic response appears to be influenced by further factors, among them the position of the centre of strength with respect to the centre of mass (Bosco et al., 2008), the interaction among bi-directional horizontal and vertical forces in resisting elements (De Stefano and Pintucchi, 2002), and seismic motion characteristics (Peruš and Faifar, 2005; Lucchini et al., 2009). Therefore, nonlinear time-history analyses are advisable to appropriately evaluate the seismic response of in-plan irregular buildings. Nevertheless, with the aim to perform non linear seismic analyses utilizing more simplified methods, in the last years large research efforts have been devoted to modify the standard pushover procedure (Freeman et al., 1975; Fajfar, 2000), which provides a good estimate of inelastic demand when no torsional effects are present (Krawinkler and Seneviratna, 1998), in order to account for torsional response of in-plan irregular buildings.
To this end, Fajfar et al. (2005) combine the results of the standard pushover procedure with those provided by an elastic response spectrum analysis; Bosco et al. (2008) perform the standard pushover analysis by applying eccentric lateral forces; Chopra and Goel (2004) consider different lateral force patterns each one referred to a vibration mode; Antoniou and Pinho (2004) adopt a force/displacement pattern which is updated during the analyses to account for the variation of dynamic characteristics.
Generally, these methods improve the accuracy in determining the seismic response of in-plan irregular structures with respect to the standard pushover method. Also, they provide results in good agreement with those obtained by time-history analyses (Baros and Anagnostopoulos, 2008).
With the aim to evaluate the role of in-plan irregularity on the seismic performances of existing buildings, a wide parametric analysis has been carried out in the present work. Some structural types representative of typical RC existing buildings designed only to vertical load and widely present in the post-1971 Italian building stock have been analysed.
In the selected structures a crucial role in determining in-plan irregular configurations is assigned to the characteristics and position of the staircase. In fact, staircase structure is frequently made up of cantilever steps and inclined cranked beams that connect two adjacent floors. These elements introduce discontinuities into the regular RC structure and, due to their large stiffness, can cause in-plan irregularity.
Therefore, varying the in-plan position of staircase, the seismic performance of structures having different grades of in-plan irregularity have been analysed. Further different number of storeys, presence and position of masonry infills and concrete strength values have been also considered in the extensive parametric analysis. Seismic response has been determined through non linear static analyses adopting a time-invariant force distribution that includes lateral forces and torsional moments at each floor.

## 2. SELECTION AND DESIGN OF BUILDING TYPES

Seismic assessment has been performed on existing RC framed buildings representative of post-1971 Italian buildings designed only to vertical loads. The selected types represent an extension of the building types analyzed by Masi and Vona (2004) for seismic vulnerability studies.
Making reference to the codes in force, the available handbooks and the typical current practice of the period under study, the selected types have been designed by means of a simulated design procedure proposed in Masi (2003). Safety verifications have been performed adopting the allowable stress method, as was usual in the period under study. With respect to the constituent materials, mechanical properties typically used in the post-1971 period have been used, i.e. medium quality concrete C20/25 ($f_{ck}$=20MPa) and deformed steel with grade close to S400 type (FeB38K, $f_{yk}$=400MPa).
The selected types have 2 and 4 storeys (2s, 4s) representative of low- and mid-rise buildings (Milutinovic e Trendafiloski, 2003), respectively, and rectangular plan shape with total dimensions 22.5×10.0m (X and Y direction, respectively) (Figure 1). Interstorey height is constant and equal to

3m.
The structures have lateral load resisting frames only along the longitudinal direction X. In particular, along this direction three resisting frames, with bay length varying in the range 5-2.5m (the latter corresponding to the staircase width), are present. Dimensions of beam sections are constant at every bay and storey and equal to 30×50cm.
Along the transversal direction Y, the structure has two bays (5m long) with only the exterior frames having a rigid beam (30×50cm) while, in the interior frames, columns are connected through the one-way RC slab (a strip with dimensions 22×20cm is considered in the model).
The smallest dimension of columns cross-section is 30cm while the greatest one varies with the number of storeys and the in-plan position; it is worth noting that the column members of the 2s types are identical, in terms of section dimensions and reinforcement details, to the two upper storeys of the 4s types.
The staircase structure is made up of two inclined cranked beams at each storey, arranged in two adjacent frames along the Y direction. Cross-section dimensions of staircase beams are equal to 30×50cm.
Varying the in-plan position of staircase two main structural types can be defined: a) Central Stair (CS), having the staircase in symmetric position with respect to Y axis (Figure 1a); b) Eccentric Stair (ES), where the staircase is located in asymmetric position with respect to both X and Y axes (Figure 1b). In order to highlight the influence of in-plan irregularity, a regular reference structure without stair (No Stair, NS) has been also considered, representative of building types where either the stiffness of the staircase members is negligible or the staircase layout is such that no significant contribution to the lateral load resisting system is provided (Figure 1c).
Masonry infills are made up of two panels (cavity wall type) of hollow brick with effective thickness equal to 20cm. Varying the presence and position of infill masonry walls along the exterior frames in Y direction, three types can be defined (Masi, 2003), that is BF (Bare Frame), IF (Infilled Frame), and PF (Pilotis Frame) (Figure 2). It is worth noting that BF types are representative of structures where infills either have many and/or very large openings or are badly connected to the structure, so that their contribution to the strength and stiffness of the structure can be neglected.

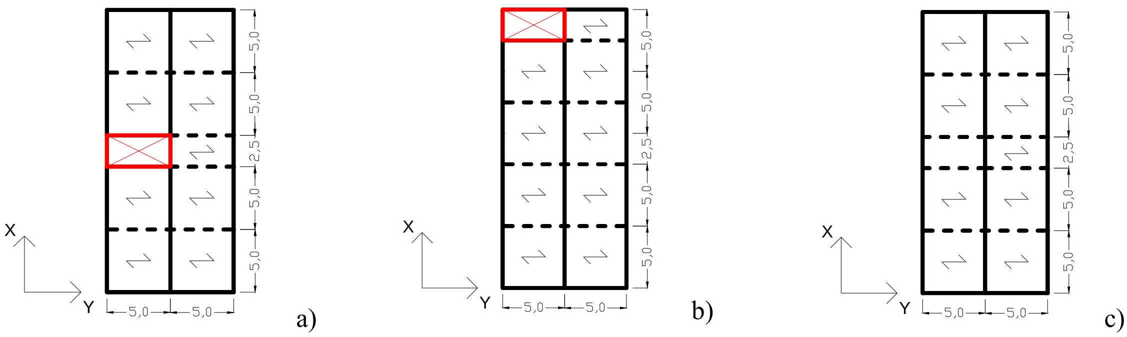


**Figure 1.** a) Central Staircase (CS), b) Eccentric Staircase (ES), c) No Staircase NS types

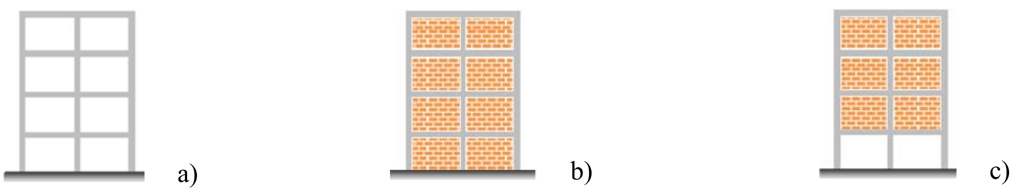


**Figure 2.** a) Bare Frame (BF), b) Infilled Frame (IF), c) Pilotis Frame (PF) types

## 3. METHODOLOGY

Structural modelling has been performed using the finite element code SAP2000 (1995) taking into account the geometric characteristics of structural elements and the distribution of in-plan and in elevation masses. Each structural member (beam and column) has been modelled by *beam* finite elements defined by the cross-section dimensions and the mechanical properties of the materials. Masonry infill panels have been modelled by using an equivalent diagonal strut, whose area has been determined by multiplying the panel thickness (*t*) by an equivalent width (*w*). The expression due to Papia et al. (2003) has been used to compute *w*, providing a value equal to about 110cm (ratio w/d = 0.19, where *d* is the diagonal length of infill panels). At each floor a diaphragm constraint has been adopted assuming that diaphragms exhibit sufficiently in plan stiffness to be modelled as rigid. As shown in Masi et al. (1997), such an assumption can be considered valid for RC floor slabs with dimensions and characteristics (e.g. absence of large openings or re-entrances) such as those present in the buildings under examination. A full restraint has been assigned at the base of the first storey columns. A macro-modelling based on lumped plasticity has been adopted to analyse the non linear seismic response of the structures. At both ends of each structural member a bending moment–rotation relation has been defined through a bi-linear curve described by the values of the yielding moment ($M_y$), of the chord rotation ($\theta_y$) and of the ultimate chord rotation ($\theta_u$) (green line in Figure 3a). $\theta_y$ and $\theta_u$ have been evaluated according to Italian NTC08 Commentary (Circolare n. 617/09) which provides the same expressions of EC8-3 (CEN, 2005). When a brittle failure was predicted, the M-θ relation above mentioned has been modified (red line in Figure 3a). In particular a bending moment value $M(V_{Rd})$ lower than the yielding value $M_y$ has been calculated as a function of the ultimate shear resistance $V_{Rd}$. To this purpose, $V_{Rd}$ value has been calculated according to the NTC08 Commentary which provides values lower than EC8-3. To take into account the effects of significant axial force variations, the moment-axial force interaction M-N (Figure 3b) has been considered for the plastic hinges of the inclined elements of the staircase. Moment values have been computed considering a parabola–rectangle diagram for concrete under compression with maximum and ultimate strength values equal to $f_{cm}/CF$, strain at peak stress $\varepsilon_{co}$=0.002, and unconfined ultimate strain $\varepsilon_{cu}$=0.0035. No tensile strength has been considered. An elastic–perfectly plastic stress–strain diagram is considered for steel, with maximum strength equal to $f_{sm}/CF$ and ultimate strain $\varepsilon_{su}$=0.01. Bearing in mind the variations of the mechanical properties of constituent materials typically found in real buildings, three concrete strength values (i.e. $f_{cm}$ = 10, 18, 28MPa) and one steel strength value ($f_{ym}$ = 400MPa) have been considered in evaluating the capacity of the structures under examination. A confidence factor value equal to 1 (CF = 1) has been assumed referring to an exhaustive knowledge level. Uncracked stiffness properties of members have been adopted, with concrete modulus of elasticity ($E_c$) equal to 27085N/mmq.

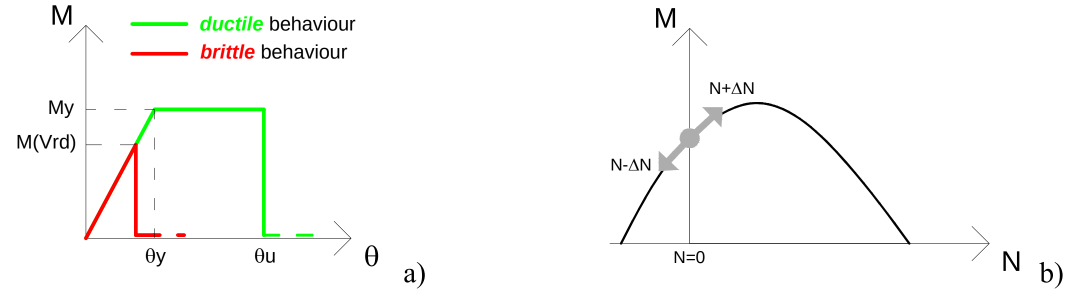



**Figure 3.** a) Moment – rotation M-θ relation of plastic hinges. b) Moment – axial force interaction of staircase inclined members

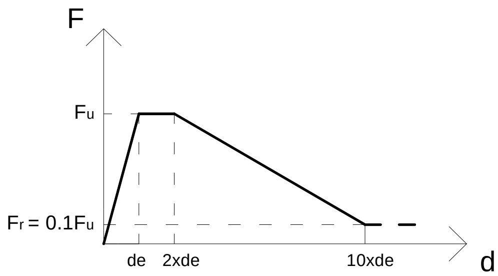



**Figure 4.** Axial force-displacement relationship of the equivalent struts modelling masonry infills

The non linear and degrading behaviour of the diagonal strut simulating masonry infill panels has been modelled through a plastic hinge located at the ends of the strut acting only under compression loads. The force-displacement relationship *F-d* (Figure 4) has been defined on the basis of the following parameters: initial stiffness $K_i = (E_w \cdot w \cdot t)/d = 56000\text{kN/m}$, displacement at the elastic limit $d_e = F_u / K_i = 3.37 \cdot 10^{-3}\text{m}$, ultimate strength $F_u = w \cdot t \cdot f_w = 189\text{kN}$, where $E_w$ = 2000N/mm$^2$ is the modulus of elasticity and $f_w$ = 1.20N/mm$^2$ is the compressive strength of masonry.

The seismic performances of the selected types have been evaluated through the Non-Linear Static (pushover) Analyses (NLSA) according to NTC2008 and, particularly, adopting the detailed provisions reported in the relevant Commentary (Circolare n.617, 2009). Pushover curves, which represent the relation between the base shear force (*V*) and the control node displacement (*d*), have been determined under conditions of constant gravity loads and monotonically increasing horizontal loads according to a modal pattern distribution for each of the two orthogonal directions in plan. In case of rotation coupling response of structures, seismic loads are both lateral forces ($H_X$, $H_Y$) and torsional moments ($M_\theta$), defined according to the MPA method (Chopra and Goel, 2004):

$$P_i = \begin{Bmatrix} H_{i,X} \\ H_{i,Y} \\ M_{i,\theta} \end{Bmatrix} = \begin{Bmatrix} m_i \times \phi_{i,X} \\ m_i \times \phi_{i,Y} \\ I_O \times \phi_{i,\theta} \end{Bmatrix}$$

where $m_i$ is the mass at each floor, $I_0$ is the polar moment of inertia of the floor diaphragm about the vertical axis through the centre of mass, $\phi_{xn}$, $\phi_{yn}$, $\phi_{\theta n}$ are the modal displacements.

A bi-linear curve relevant to an idealized equivalent single degree of freedom (SDOF) system has been computed following the provisions provided by NTC08 Commentary, which are substantially consistent with EC8 provisions. Seismic performances have been evaluated starting from the period of vibration $T^*$ of the equivalent system assuming an EC8 elastic spectrum with ground type A (Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface). In particular, for a given value of the peak ground acceleration ($a_g$) it is possible to evaluate the spectral pseudo-acceleration ($S_e$) and the spectral displacement ($S_d$) relevant to $T^*$. Consequently, the target displacement of the MDOF system ($d_{max}$) is calculated by multiplying $S_d$ by the modal participation factor ($\Gamma$). On the basis of the push-over curve determined for each type, the seismic demand corresponding to the $d_{max}$ value is evaluated in terms of either shear ($V_{Sd}$, for brittle elements) or rotation ($\theta_{sd}$, for ductile elements). Therefore, varying the $a_g$ value the seismic performances have been evaluated making reference to the Limit State of Life Safety according to NTC08 (corresponding to the performance requirements of the Limit State of Significant Damage according to EC8-3) (Figure 5). Specifically, the minimum $a_g$ value causing $\theta_{sd}$ = ¾ $\theta_u$ (for ductile elements) and $V_{Sd}$ = $V_{Rd}$ (for brittle elements) has been calculated ($a_{g,LS}$).

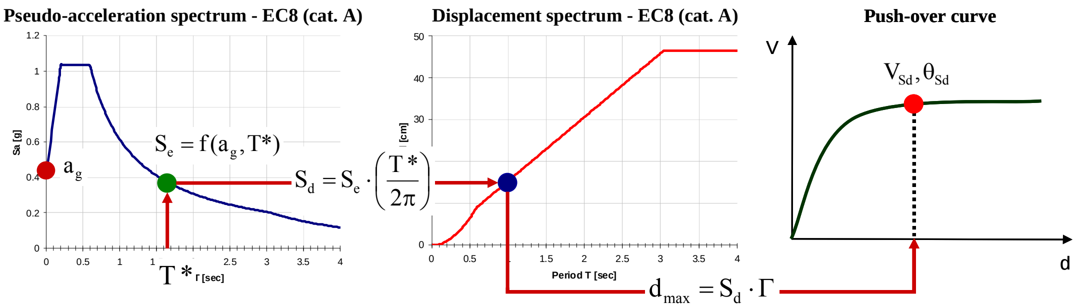



**Figure 5.** Procedure to assess the seismic performances of the structures

## 4. RESULTS

In this section the non linear seismic performances of the selected irregular building types have been analysed and compared. In particular, two main irregularity factors have been pointed out: (i) presence and in-plan position of staircase structure, (ii) presence and position along height of masonry infills.

With respect to other structural parameters, it is worth noting that results show that varying the concrete strength values ($f_c$ equal to 10, 18 and 28N/mmq) a slight influence on the performances of structures has been found. Therefore, for sake of brevity only the seismic performances evaluated considering $f_{cm}$=28 N/mmq are reported and discussed in the following.

### 4.1 Role of presence and in-plan position of staircase structure

As a consequence of its large stiffness, the staircase structure can determine an asymmetric distribution of in-plan stiffness and strength. Concerning stiffness irregular distribution, in each building type the structural eccentricity values between centre of mass and centre of stiffness, along both X and Y directions, have been calculated. Considering BF types, central staircase type (CS) is symmetric with respect to Y-axis, while a low eccentricity with respect to X-axis can be found (about 5% of the plan dimension in Y direction). On the contrary, ES type has remarkable eccentricity values along both Y and X axes. Specifically, in ES type the eccentricity along X is equal to about 28% of the in-plan dimension in the same direction.

Modal response spectrum analyses have been firstly performed on the structures under study showing that they have significantly different modal characteristics. Table 4.1 reports the values of the period of vibration T and the relevant percentage of the effective modal mass M* evaluated in the X, Y and θ (rotation around the vertical Z axis) directions of seismic motion, considering types with four storeys (4s). The values relevant to the fundamental mode are reported in bold.

The fundamental period of vibration of NS type is 1.35 sec with a mode purely translational along Y, while the fundamental mode of CS type is rotational and it has fundamental period equal to 0.90 sec. Finally, as for ES type, the fundamental mode is roto-translational with a period of 1.18 sec and modal mass equal to 10%. All types have a similar period of vibration along the X direction, that is equal to about 0.8 sec.

**Table 4.1.** Values of vibration period T and effective modal mass M* of BF types with four storeys (4s) evaluated along X, Y and θ directions

| | | X-dir | | Y-dir | | θ-dir | |
|---|---|---|---|---|---|---|---|
| | | T[sec] | M* [%] | T[sec] | M* [%] | T[sec] | M* [%] |
| | *ES* | 0.81 | 83.5 | **1.18** | **55.0** | 0.60 | 73.2 |
| *4s* | *CS* | 0.77 | 80.7 | 0.81 | 84.0 | **0.90** | **27.6** |
| | *NS* | 0.78 | 83.9 | **1.35** | **82.0** | 0.92 | 18.6 |

For CS type the mass corresponding to the fundamental (rotational) mode is small and the period value is close to that evaluated in the Y direction (0.81 sec). Therefore, torsional effects are negligible and non linear analyses have been carried out considering only horizontal forces. On the contrary, for ES type both horizontal forces and torsional moments have been considered.

As already mentioned, the seismic response is remarkably different between the two in-plan directions. Non linear analyses show that stiffness and maximum base shear values in the X direction are higher than those in the Y direction. In particular, for NS type the base shear in the X direction is nearly twice as much as that along Y, while the stiffness ratio between X and Y directions is around 5. In the staircase types (CS and ES), due to the large contribution of the inclined elements to the stiffness and strength along the Y direction, lower differences between the two directions can be found.

In order to understand the influence due to presence and in-plan position of staircase, Figure 6 shows the comparison between the push-over curves determined in both X and Y seismic direction of NS, CS and ES buildings types having 4 stories (4s).

In the X direction all types (NS, CS, ES) have practically coincident values of elastic stiffness and maximum base shear (around 1000 kN, that is 12% of the total building weight). Therefore, staircase presence and position do not affect the seismic response along that direction. On the contrary, large differences have been found in the Y direction, that is the direction along which the rigid inclined members of staircase are arranged. Considering the presence of staircase in central position (CS type), higher values of elastic stiffness and maximum base shear are found with respect to NS type, with increments respectively equal to +150% and +130%. Results for ES type are intermediate to those evaluated for NS and CS types. In particular, the eccentric position of staircase causes a decrease of both elastic stiffness (-46%) and maximum base shear (-22%) compared to CS type.

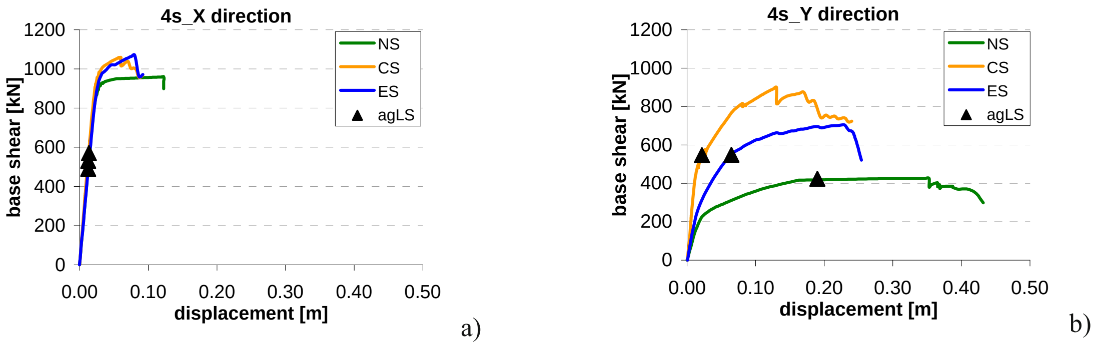



**Figure 6.** Performance of the NS, CS, ES bare frame types with four storeys (4s) relevant to the (a) X and (b) Y direction

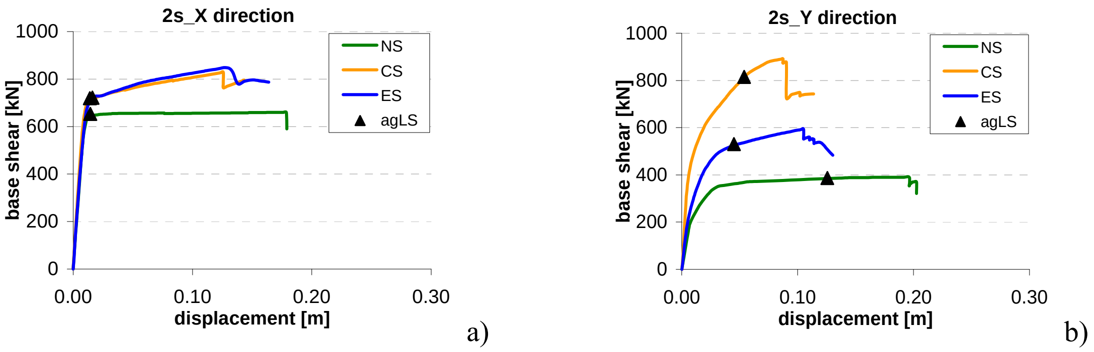



**Figure 7.** Push-over results along the (a) X and (b) Y direction referred to NS, CS and ES types with 2 storeys

All analysed structures suffer a brittle failure along the X direction, as a consequence of the shear failure of the beam members in that direction. Therefore, very low values of the ground acceleration related to the Limit State of Life Safety, $a_{g,LS}$, are found, that is $a_{g,LS}$ equal to about 0.05g.

In the Y direction the values of $a_{g,LS}$ are significantly different both among the types under study and with respect to the X direction. For NS type $a_{g,LS}$ is equal to 0.32g, while lower values have been computed for ES type ($a_{g,LS}$ = 0.12g) and CS type ($a_{g,LS}$=0.065g).

Results in the Y direction depend on the following factors: 1) failure behaviour of structural members, 2) distribution of seismic demand values along the stiff and flexible sides of ES type.

All members of NS type suffer a ductile failure and the structure fails in flexural bending involving the beams of the external frames. On the contrary, for the vertical members of staircase a brittle failure has been predicted. As a result of their great lateral stiffness, the plane frames where staircase is located take most part of the horizontal inertia forces, then the columns of CS and ES types fail in shear for low values of seismic action. Moreover, due to the large rotational displacements, in ES type seismic demand on the frames near the centre of stiffness (stiff side) is lower than that on the frames at the flexible side (near the centre of mass), as well as it is lower than that in the staircase frames of CS type.

The main role of the brittle failure of structural members and of staircase position on seismic performances is confirmed by the results found on the two storey types (Figure 7), whose elements suffer a ductile failure.

In 2s types very different performances have been evaluated with respect to 4s types. In particular, in CS type $a_{g,LS}$ is equal to 0.32g (0.065g in 4s type), while a lower value ($a_{g,LS}$ = 0.23g) has been calculated in ES type (0.12g for the 4s type). In NS type $a_{g,LS}$ is equal to 0.39g.

Generally, 2s structures fail in bending (ductile failure) involving the columns of staircase in CS and ES types, and the beams in NS type.

## 4.2 Role of presence and position of masonry infills

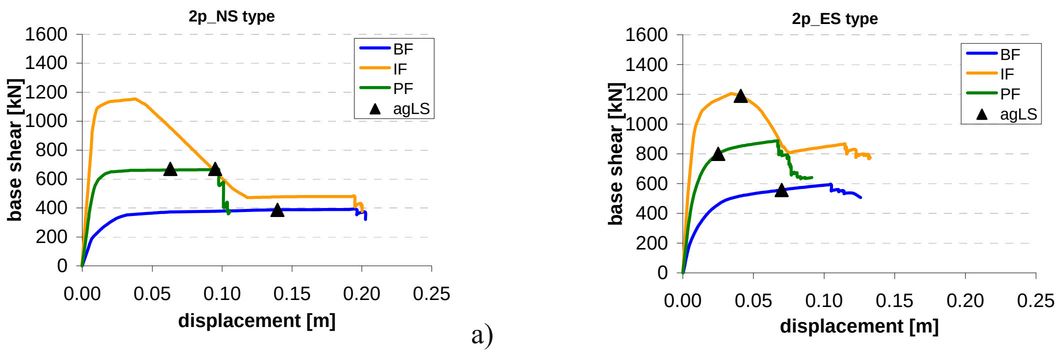
As a consequence of the in-plan position of infill panels (arranged only in the external frames along the Y direction), the main differences among BF, IF and PF types have been found in the Y direction,

while along the transversal direction (X direction) results are practically coincident.
Results of linear dynamic analyses show that the presence of the infill panels, particularly when they are regularly arranged along the building height (IF type), reduces the variations caused by to the presence and position of staircase. Generally, in IF type the vibration modes are similar for all considered types (NS, CS, ES). Further, considering the same number of storeys, the period of vibration T in the Y direction for IF types is nearly constant among all considered types, with values in the ranges 0.23-0.26 and 0.45-0.50 for 2s and 4s type, respectively (Table 4.2). The higher values of T are relevant to BF types, while intermediate values have been found in PF types.

**Table 4.2.** Periods of vibration T and related modal masses M* along the Y direction for all types under study

| | | BF | | IF | | PF | |
|---|---|---|---|---|---|---|---|
| | | T[sec] | M* [%] | T[sec] | M* [%] | T[sec] | M* [%] |
| 2s | ES | 0.59 | 59.5 | 0.26 | 91.3 | 0.44 | 66.5 |
| | CS | 0.41 | 89.7 | 0.23 | 92.4 | 0.35 | 97.2 |
| | NS | 0.66 | 86.1 | 0.25 | 93.1 | 0.46 | 99.1 |
| 4s | ES | 1.18 | 55.0 | 0.50 | 85.1 | 0.71 | 74.5 |
| | CS | 0.81 | 84.0 | 0.45 | 86.3 | 0.58 | 93.9 |
| | NS | 1.35 | 82.0 | 0.48 | 86.3 | 0.72 | 97.2 |

Figure 8 shows the push-over curves determined for NS (a) and ES types (b) with two storeys (2s) considering BF, IF, PF configurations.
With reference to IF configuration, masonry infills significantly increase stiffness and strength. In particular, comparing IF and BF types a stiffness increment in the range 150-160% has been found in the buildings without staircase (NS type). Similar results have been found by comparing the stiffness increase due to staircase (i.e. comparing CS and NS types in BF configuration). Similarly, the increase due to masonry infills is in the range 75-100% as for maximum base shear values, with variations lower than those deriving from the presence of staircase members.
It is worth noting that in terms of maximum base shear the influence due to the in-plan position of staircase is negligible when IF configuration is considered.
Generally, stiffness and maximum base shear values in PF types are intermediate between bare (BF) and infilled (IF) types.



a) b)

**Figure 8.** Performance of the (a) NS and (b) ES types with two storeys considering BF, IF, PF infill configurations

**Table 4.3.** Values of $a_{g,LS}$ for all considered types

| | $a_{g,LS}$ [g] | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | BF | | | IF | | | PF | | |
| | NS | CS | ES | NS | CS | ES | NS | CS | ES |
| 2p | 0.39 | 0.32 | 0.23 | 0.63 | 0.35 | 0.31 | 0.45 | 0.22 | 0.19 |
| 4p | 0.32 | 0.065 | 0.12 | 0.43 | 0.075 | 0.07 | 0.39 | 0.05 | 0.06 |

Table 4.3 reports the $a_{g,LS}$ values for all the examined types.

$a_{g,LS}$ values evaluated on IF types are higher than those of BF types. In particular, the presence of regularly arranged infill panels significantly improve the seismic performances of NS types, while $a_{g,LS}$ values slightly increase in CS types.
$a_{g,LS}$ values of CS and ES types with IF configuration are similar. Indeed, infill panels reduce the eccentricity between the centres of mass and stiffness, therefore lower effects due to the rotational response have been evaluated in ES type.
Generally, $a_{g,LS}$ values of PF types with staircase are significantly lower than those of IF types, while they are slightly lower than $a_{g,LS}$ values in BF types.
PF configuration has $a_{g,LS}$ values greater than those obtained in BF type considering the structures without staircase (NS type).
The failure mechanisms of regularly infilled structures (IF) are the same with respect to those observed for bare structures (BF). While, in NS type with 2 storeys and pilotis frames (PF type) a premature soft storey failure has been found.

## 5. CONCLUSIONS

An extensive parametric analysis on structural types representative of the Italian building stock designed only to vertical loads has been carried out. Specifically, structural types with a stiff staircase structure and masonry infill walls have been analysed.
The role of staircase structure has been considered varying its position in order to analyse different values of in-plan eccentricity. In particular, central (CS) and eccentric (ES) staircase types have been considered. Results have been compared with structural types without staircase (NS type), i.e. buildings where the staircase contribution to the global stiffness and strength can be neglected.
As typical in RC existing buildings without earthquake resistant design, frames are arranged only along one direction, that is the longitudinal one (X direction). As a consequence, the analysed structures have shown different performances in the two principal in-plan directions. Along the X direction the brittle failure of the beams leads to low values of the ground acceleration determining the Limit State of Life Safety ($a_{g,LS}$ equal to about 0.05g). Further, negligible differences of $a_{g,LS}$ values have been found for all considered types.
On the contrary, in transversal direction (Y direction) seismic response is remarkably affected by the presence of staircase, whose very rigid inclined members are arranged along this direction. With respect to NS regular type, the presence of staircase determines large increments of lateral stiffness (110-150%) and base shear (100-130%). With respect to the failure mechanism, a brittle failure of the columns of staircase has been found, therefore CS and ES have $a_{g,LS}$ values lower than those of NS type, whose elements suffer a ductile failure.
Depending on the number of storeys, different performances have been found with increasing irregularity. In 2 storeys structures $a_{g,LS}$ value in ES type is lower than in CS type. On the contrary, $a_{g,LS}$ value in ES type with 4 storeys is unexpectedly higher than in CS type. Indeed, this can be explained with the different failure mechanisms of the column members in the frames more distant from the centre of stiffness and with the reduction of demand on the rigid side of the building due to the rotational response. In the structures with 2 storeys, whose columns suffered a ductile failure, $a_{gLS}$ is equal to 0.23g and 0.32g, respectively for ES and CS type. On the contrary, in the structures with 4 storeys, whose columns showed a brittle behavior, $a_{g,LS}$ value is equal to 0.12g for ES type while it decreases up to 0.065g in CS type.
With respect to the role of infill walls, three types have been considered varying their presence and position along height, that is BF (Bare Frame), IF (Infilled Frame), and PF (Pilotis Frame) types. Generally, in IF types $a_{g,LS}$ values are higher than in BF types. Moreover, masonry infills reduce in-plan eccentricity due to staircase and, consequently, reduce the difference of performances between CS and ES types. In PF type $a_{g,LS}$ value is lower than those evaluated in BF and IF types, both for CS and ES type.
Future developments of the study should be devoted to: (i) extend the set of structural types to be analysed and (ii) more accurately define the adopted models. As regards the first point, high-rise buildings, other kinds of in-plan irregularity present in the Italian building stock (e.g. irregular plan shapes like L, C, I shapes), and structures with low earthquake resistant design level should be

analysed. As regards the second point, considering the remarkable influence of brittle failure on seismic performances, more accurate shear capacity models should be adopted based on the results of experimental tests performed on specimens representative of real existing buildings. Finally, in assessing structural performances also the capacity of beam-column joints should be included.

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