Structural Engineering and Mechanics, Vol. 39, No. 4 (2011) 599-620
DOI: http://dx.doi.org/10.12989/sem.2011.39.4.599

Static vulnerability of existing R.C. buildings in Italy: a case study

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(Received September 28, 2010, Accepted May 25, 2011)

Abstract. The investigation on possible causes of failures related to documented collapses is a complicated issue, primarily due to the scarcity and inadequacy of information available. Although several studies have tried to understand which are the inherent structural deficiencies or circumstances associated to failure of the main structural elements in a reinforced concrete frame, to the authors knowledge a uniform approach for the evaluation building static vulnerability, does not exist yet. This paper investigates, by means of a detailed case study, the potential failure mechanisms of an existing reinforced concrete building. The linear elastic analysis for the three-dimensional building model gives an insight on the working conditions of the structural elements, demonstrating the relevance of a number of structural faults that could sensibly lower the structure's safety margin. Next, the building's bearing capacity is studied by means of parametric nonlinear analysis performed at the element's level. It is seen that, depending on material properties, concrete strength and steel yield stress, the failure hierarchy could be dominated by either brittle or ductile mechanisms.

Keywords: static vulnerability; case study; failure mechanisms; reinforced concrete frames; elastic analysis; non-linear analysis; load multiplier

1. Introduction

In the last decades a number of structural failures unexpectedly hit our built environment, alerting public opinion and engineers on the urgent and thorny problem of structural safety for existing buildings. Excluding the collapses caused by earthquakes or by very rare events such as fire, gas explosions or exceptional weather conditions, still a conspicuous amount of so-called "spontaneous collapses" happen. Just to mention some of the most recent ones for Reinforced Concrete (R.C.) buildings in Italy we recall the failures of a 5 story building in Rome in 1998, the 6 story building collapse in Foggia (Palmisano *et al.* 2003) in 1999 and the failure of an 8 story apartment building in Naples in 2001 (Augenti 2003). Other recent examples worldwide are from Turkey (Kaltakci *et al.* 2007) and Israel (Michel *et al.* 2007).

The investigation on possible causes of failures and structural downfalls related to documented

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collapses is a complicated issue, primarily due to the scarcity and inadequacy of information available. One of the first attempts to collect and catalogue the failures of R.C. constructions and their causes may be found in the *Bibliography of Structural Failures 1850-1970* (Singh 1976) and in *Study and analysis of the first 120 failure cases* (Walker 1981). These first studies were integrated and re-elaborated by other researchers, among which (Gori and Muneratti 1995, Gori and Muneratti 1997, Melchers 1999). The necessary work of classifying failures according to predefined categories inevitably give raise to a certain degree of subjectivity, often obstructing researchers ability to understand the interrelations between the facts. Hence, it is understandable that the principal causes of failure are classified under different, even if similar, categories, and with varying order of importance. According to (Walker 1981) the major responsible of failure is 'inadequate appreciation of loading conditions or structural behavior' (43%), followed by 'inadequate execution of erection procedure' (13%), 'random variation in loading, structure, materials, workmanship' (10%), 'mistakes in drawings or calculations' (7%) etc. Gori and Muneratti (1995) assign major responsibility to 'errors in calculation' (34%), followed by 'incorrect choice or production of structural materials' (21%), 'construction errors' (19%), 'incorrect design of structural details' (16%) etc.

Independently from the relative percentages attributed to the single causes, it appears generally that the incorrect choice or error in the use of *design models*, *materials* inadequacy, *execution defects*, poor *details* may be responsible of serious structural diseases.

The failure mechanisms that could be activated in R.C. moment-resisting frame type buildings could be roughly subdivided into two categories: those interesting vertical structures, i.e., columns failing under excessive compression, and those involving horizontal structures, i.e., beams or floor slabs failing under excessive bending or shear or due to punching shear mechanism (for slabs). The former type of collapse is certainly the most hazardous, being brittle and involving potentially the entire building, especially if the structural system does not supply adequate redundancy (robustness). However, according to (Muneratti *et al.* 2006) the failure of horizontal structures is nearly two times more frequent than the failure of vertical ones and in some cases could trigger a collapse mechanism involving the entire building, as documented in King and Delatte (2004).

Although several studies have tried to investigate on the inherent structural deficiencies or circumstances associated to failure of vertical elements (Augenti 2003, Palmisano-Vitone 2003) or horizontal ones (Spence *et al.* 2005), to the authors knowledge a uniform approach for the evaluation a building *static vulnerability*, accounting both for potential failures in vertical and horizontal elements, does not exist yet.

In a recent paper (Polese *et al.* 2006) a simple method deemed to the classification of buildings based on their static vulnerability was presented. The procedure relies on very simple structural models. In particular, the building state is evaluated as a function of three different limit states at the element level (column, beam and floor slab). Being these former studies more focused to the formulation of a general assessment system, to be used in large scale applications, the performance evaluation at the element level was limited to linear safety-factor type analyses. However, when the potential failure mechanism of a building is to be investigated, it is important to consider the non-linear behaviour (Vecchio *et al.* 2004). In fact, although some type of failures, such as the column one, brittle, could be either investigated with linear or non-linear type analyses, other flexure-governed mechanisms allow anelastic re-distribution among adjacent elements and therefore could only be studied with non-linear analyses.

With the aim of a major comprehension of the potential failure mechanisms in a R.C. building under the sole gravity loads, this paper presents the results of a detailed case study for an existing building representative for gravity load designed structures in southern Europe. The structural faults that may contribute to the lowering of a building's safety are first introduced. The study for the building, then, is performed enlightening the role of some of these faults, that are present for the specific case.

In particular, the linear elastic analysis for the three-dimensional building model gives an insight on the working conditions of the structural elements. Next, the building's effective bearing capacity is studied by means of parametric nonlinear analyses performed at the element's level.

Differential displacements at the foundation level, that are other important causes of structural diseases in existing buildings, are not considered.

The results that will be discussed are undoubtedly affected by the singularity of the case study; nevertheless, they can be helpful in individuating the structural faults that are responsible for the lowering of the safety margin in R.C. constructions and on the element's failure hierarchy depending on material properties. Another study (Verderame *et al.* 2009) investigates on some key aspects related to material properties versus design strengths and modeling features with the aim of generalizing the considerations inherent to failure mechanisms and the related load multipliers.

2. Structural faults in gravity load designed R.C. buildings

This paragraph presents some issues related to design, materials and execution defects that were typical in R.C. constructions built in Italy during the twenty years immediately following the Second World War. Many characteristics of this building typology are common also for the R.C. constructions in other European countries (Bal *et al.* 2007, Kaltakci *et al.* 2007).

Regarding the *Design Models*, in old-type codes and design practices it was common to determine approximate design forces for the floor slabs, the beams and the columns with the use of analytical models that were developed at the level of the single element and not for the entire structure. Although the so-called *allowable stresses method*, that basically limited the maximum tensional levels in the steel and concrete of the R.C. members of a structure, generally provided structures with cautiously large safety margin, it could happen that approximation in the evaluation of loads or in the choice of the element model lead to unpredicted lowering of this margin.

Fig. 1 shows the simplified load transfer schematization for a gravity load designed reinforced concrete frame with unidirectional joist-slabs. A first possible simplification consists in the slab's and the beam's modeling with regard to the determination of flexural effects. In fact, the bending moment at the span edges (M_r-M_t in figure) or at mid-span (M_s at section s in figure) were sometimes calculated with simplified formulas (see point (1) in figure) only roughly accounting for the element's constraint scheme with assigned α-coefficients (Verderame *et al.* 2009). Moreover, it could happen that the structural continuity was neglected in transferring the load to the structural elements. In fact, it was not infrequent that the slab's shear and the relative loads transferred to the beams were determined based on the simple influence area rule (see point (2) in figure); in such a way it could happen that the beam loads was underestimated up to 25%.

Analogously, the approximation chain continues if the beam's continuity is neglected while transferring the loads to the columns (see point (3) in figure), leading to a possible further underestimation of the axial load on the vertical elements.

Sometimes, when combining the above-mentioned approximations with old code prescriptions for

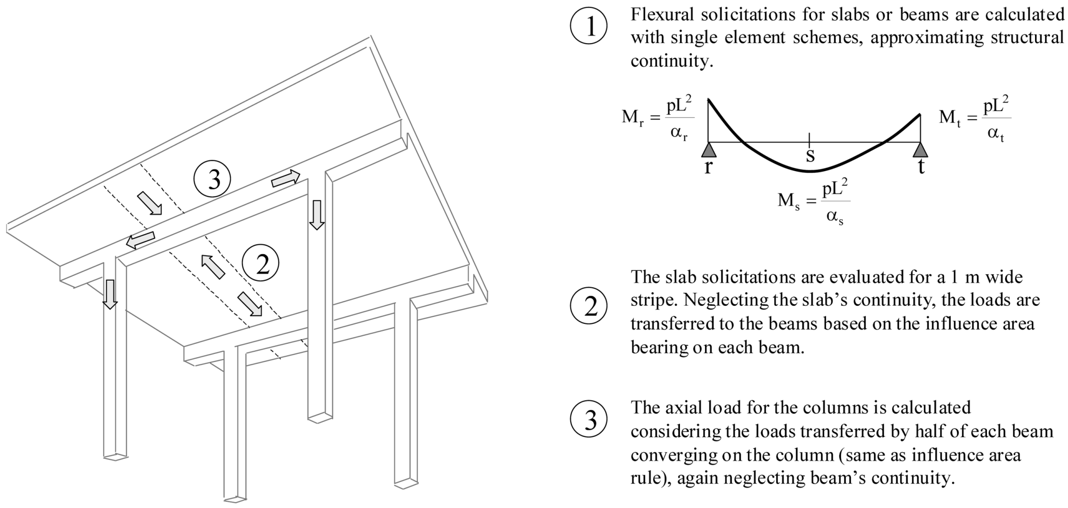


Fig. 1 Simplified load transfer schematization for a gravity load designed reinforced concrete frame with unidirectional joist-slabs

the design of R.C. elements, unsafe conditions could be obtained.

For example, with regard to the shear design of the beams, the code regulations prescribed that if the tangential stress τ did not exceed a threshold value τ_{co}, equal to 0.4MPa and 0.6MPa for the normal strength and high strength concrete, respectively, the shear resistance V_{res} was to be computed considering the sole concrete resisting section

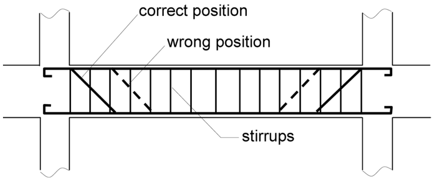
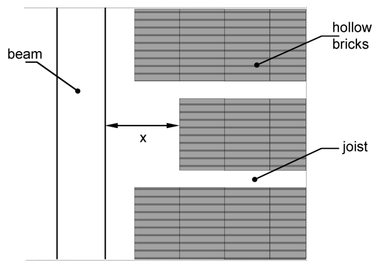
$$V_{res} = 0.9 \cdot d \cdot b \cdot \tau_{co} \tag{1}$$

with b the transversal section base and d the relative effective height. For such cases, just minimum transversal reinforcement were to be disposed; typical values were ϕ6 or ϕ8 mm stirrups every 20/25 cm or ϕ8 mm stirrups with spacing equal to min (33 cm, 0.8 d).

However, due to the above described approximate modeling, it could happen that the shear forces, and consequently the τ, were underestimated and, in the case of fallacious evaluation of the case $\tau \leq \tau_{co}$, beams with insufficient transversal reinforcement could be realized.

For what concerns *Materials*, it has to be considered that concrete and steel mechanical characteristics were generally poorer with respect to nowadays ones. Referring to concrete, it was admitted the use of compressive strength of 12 MPa for normal concretes or 16 MPa for high resistance ones; when preventively determined this value could raise up to 18-22.5 MPa. Being intrinsically a non homogeneous material, the concrete is characterized by an high variability of its mechanical properties; strengths lower than 10 MPa are rare, but possible, as confirmed by (Cosenza *et al.* 2006). For what concerns the steel, the commonly adopted bars type were smooth ones, denominated *quality type steels* Aq42, Aq50 or Aq60 depending on their ultimate strength. Whilst steel characteristics may be defined with much narrower bounds, this material could be affected by deterioration processes due to corrosion that can sensibly reduce the effective steel resisting area, hence lowering the safety margin. Although the severity of corrosion phenomenon is much more significant in weather exposed structures, such as bridge piles, sea quays, R.C. tanks or dams, still the corrosion effect could impact ordinary building constructions, as documented in (Kim *et al.* 2006).

Table 1 Typical execution defects in the elements of RC buildings

Improper placement of transversal bars	Not conforming stirrups spacing	Wrong positioning of the lightening bricks
		

Execution defects include all the defects depending on human error or fallacious work management. Table 1 summarizes some of the defects that may be found in the main structural elements of a R.C. frame.

One of the most common defects is the wrong positioning of shear reinforcement in the beams: the effective position of the diagonal shear resisting reinforcement could be easily mistaken, with the resulting inefficiency of the Ritter-Mörsch mechanism (see Table 1).

Regarding the columns, it could happen that the minimum code conforming stirrups spacing was not respected; moreover, the stirrups were often closed with 90° hooks of inadequate length. These circumstances could lead, for axial loads, to the buckling of the longitudinal bars with the consequent reduction of the concrete resisting section. Also the floor slabs may be subject to design or execution defects. Typical European reinforced concrete floors are unidirectional joist-slabs in which a thin layer of concrete on top of hollow bricks, generally 4 cm thick, has mainly a load repartition role. For such kind of elements, where the concrete resisting section is very thin, the shear resistance is entirely devolved to the concrete, whose transversal section should have suitable dimensions. Therefore, a common design solution consisted on removing the lightening bricks in proximity of the beams, with the purpose of enlarging the base for the resisting concrete section and obtaining a higher shear strength. However, the location of the section where to enlarge the joist's transversal section dimension could be easily mistaken, due to errors in the evaluation of the shear solicitation or, more frequently, to execution defects (see Table 1). As a consequence, their shear resistance could be lower than necessary.

3. Evaluation of the static vulnerability of existing buildings

The building object of this study is a 4 storey condominium, built in 1972 in Salerno, southern Italy; being Salerno classified as seismic solely in 1981, the building is Gravity Load Designed (GLD). The structure is deemed to be representative of typical mid-rise GLD residential buildings constructed in age '60 and '70 in Italy. In this period, the faulty belief that the slenderness of the structural elements could demonstrate technical skill, coupled with the tendency to build entire lots economizing on materials and workmanship, often lead to constructions characterised by slender columns, with poor reinforcement and inadequate lateral confinement (largely spaced stirrups);

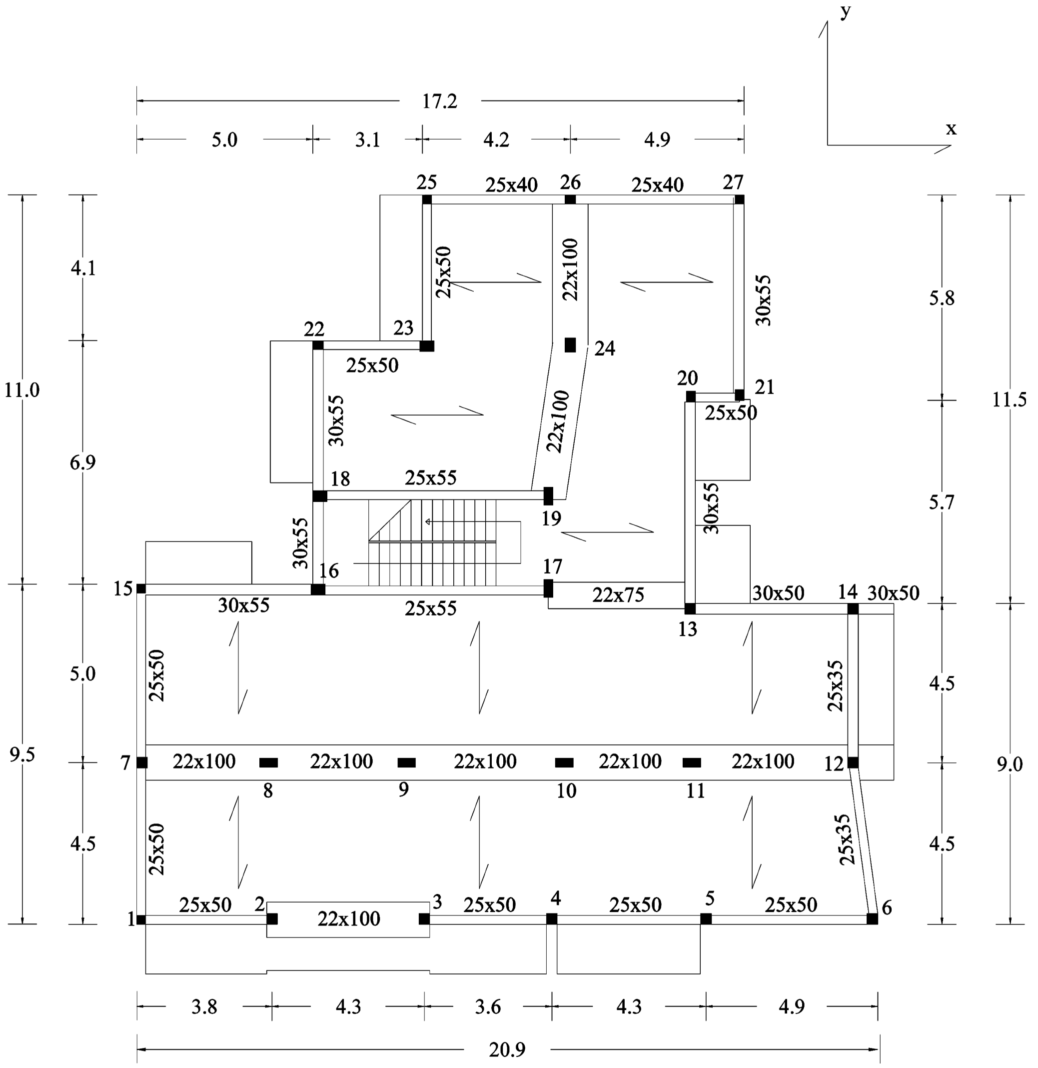


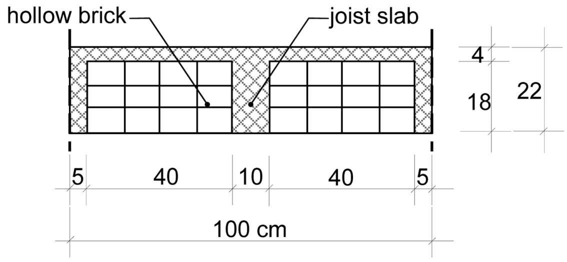
Fig. 2 First floor plan of the studied building

structural systems were generally organized so to comply with basic architectural needs not paying attention to structural redundancy (robustness).

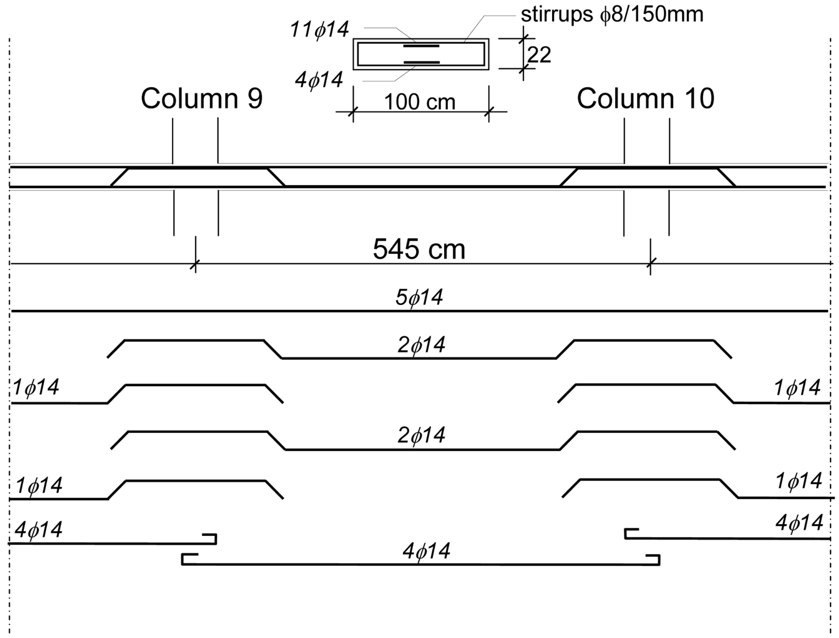
Fig. 2 shows the first floor plan of the studied building; as it can be seen, the footprint shape is that of an upside-down *T*, with global dimensions (circumscribing rectangle) 20.9 m along *X* and 20.5 m along *Y*.

The floor slabs, 0.22 m thick, are cast in place unidirectional joist-slabs, lightened with hollow bricks; Fig. 3(a) shows the scheme of a portion of the slab with its main constituents and the relative dimensions.

The structural system, as typical in GLD frames (Masi 2003, Bal *et al.* 2007), is provided only with directly loaded frames (those orthogonal to the slab's way) and the perimeter ones. Moreover, the central beams have the same thickness of the horizontal slab (embedded beams), while



(a)



(b)

Fig. 3 (a) Section of the slab, (b) longitudinal section of an embedded beam

perimeter ones are generally higher (emergent beams). Fig. 3(b) shows the longitudinal section of an embedded beam.

Being dimensioned based on gravity loads, the columns transversal section vary in a range from [25 × 25, 25 × 50] cm^2 at the first level, decreasing gradually to [20 × 25, 25 × 35] cm^2 at the upper story, the fourth. Column's dimensions and reinforcement are listed in Table 2.

Allowable stresses, as well as design loads and element schemes and basic rules adopted in design were inferred from the original design report and from design drawings. In particular, the allowable stresses for steel and concrete, were 180 and 7.5 N/mm^2, respectively, latter value decreasing to 6.0 for elements subject to pure axial load. According to codes in force at the time of construction, these values should correspond to concrete whose (cubic) compressive strength should be greater or equal than $R_c = 3\sigma_{ca} = 22.5$ MPa and steel type Aq50, that is characterized by a yield stress greater or equal than $f_y = 2\sigma_{sa} = 360$ MPa.

Design loads are 4 kN/m^2 and 2 kN/m^2 for permanent and live loads, respectively, while the load

Table 2 Columns typology

Column	Storey			
	1st	2nd	3rd	4th
1	25X25 4Φ12	20X25 4Φ12	20X25 4Φ12	20X25 4Φ12
2	30X30 4Φ16	25X25 4Φ14	20X25 4Φ12	20X25 4Φ12
3-4	30X30 4Φ14	30X30 4Φ14	25X25 4Φ14	20X25 4Φ12
5	30X30 4Φ14	25X30 4Φ14	20X25 4Φ12	20X25 4Φ12
6	30X30 4Φ14	25X25 4Φ14	20X25 4Φ12	20X25 4Φ12
7	30X30 64Φ14	25X30 6Φ14	25X25 4Φ14	20X25 4Φ14
8-9-10-11	25X50 6Φ14	25X45 6Φ14	25X40 4Φ14	25X35 4Φ14
12-13	30X30 4Φ14	30X30 4Φ14	25X30 4Φ14	25X25 4Φ12
14	30X30 4Φ14	30X25 4Φ14	25X25 4Φ12	20X25 4Φ12
15	25X25 4Φ14	20X25 4Φ12	20X25 4Φ12	20X25 4Φ12
16-18	30X40 4Φ18	30X30 4Φ16	30X30 4Φ14	30X30 4Φ14
17-19	25X50 6Φ14	25X45 6Φ14	25X40 4Φ14	25X35 4Φ14
20	25X30 4Φ14	25X30 4Φ14	25X25 4Φ12	20X25 4Φ12
21	30X25 4Φ14	30X25 4Φ14	20X25 4Φ12	20X25 4Φ12
22	25X30 4Φ14	25X30 4Φ14	25X25 4Φ14	25X20 4Φ12
23	30X40 4Φ16	30X35 4Φ16	30X30 4Φ14	25X30 4Φ14
24	30X40 4Φ14	30X35 4Φ14	30X30 4Φ14	30X30 4Φ14
25-27	25X25 4Φ14	25X25 4Φ14	20X25 4Φ12	20X25 4Φ12
27	30X25 4Φ14	30X25 4Φ14	25X25 4Φ12	25X25 4Φ12

due to infills, applied on perimeter beams, was assumed equal to 6 kN/m. In addition, a permanent load of 2.25 kN/m accounting for partition walls was added to the beam loads. Structural elements, slabs, beams and columns, were designed according to allowable stresses method, utilizing many of those approximations as listed in § 2.

Floor slabs and beams were basically designed for bending actions.

In particular, for the horizontal slabs simplified single-element schemes were adopted, evaluating the bending moment at the extremities and at mid-span as

$$M = \frac{(g_k + q_k) \cdot L_s^2}{\alpha} \tag{2}$$

with L_s the slab's span length, g_k and q_k the permanent and live characteristic loads (unit loads acting on a 1.00 m wide slab stripe) and α a coefficient roughly accounting for the element's constraint scheme (α = 14, 16 or 10 for positive moments at mid-span, negative moments at the perimeter span edges or at the interior span edges, respectively).

The bending moments for beams were evaluated based on a continuous beam scheme.

The column area A_c was designed for centered axial load N, the latter being determined based on influence area bearing on the R.C. element

$$A_c = \frac{N}{\bar{\sigma}_{ca} \cdot (1 + n\rho)} \tag{3}$$

In the (3) $\bar{\sigma}_{ca}$ is the allowable stress for concrete under pure axial load (6.0 kN/m^2), that, because the whole concrete section participates theoretically in the absorption of the axial load, is cautiously reduced by codes with respect to the allowable stress for sections subject to flexure and axial load. Moreover, in order to consider the presence of longitudinal bars reinforcement, the allowable stress was multiplied by a factor ($1 + n\rho$) with n, the homogenization coefficient, assumed equal to 10 and ρ, the reinforcement percentage, corresponding to minimum code requirement (0.8%). Note that $n = 10$ is an intermediate value between instant homogenization coefficient and long term one, as suggested by codes, in order to account for creep effects.

The columns and beams shear reinforcement was not designed and minimum shear reinforcement was disposed. In fact, according to the design report, the tangential stress τ resulted to be always lower than τ_{co}.

In particular, 2 braces $\phi 6$ stirrups every 200 mm were disposed for emergent beams and for columns, while 2 braces $\phi 8$ stirrups every 150 mm in embedded beams (ϕ represents the bar diameter in mm). It has to be noted that the embedded beam's base varies between 80 and 120 cm; in such cases the two braces stirrups could be inefficient in the shear resisting mechanisms.

3.1 The elastic analysis

The elastic analysis performed on the three-dimensional building model evidenced that some of the elements suffer even for the sole static loads. The columns, beams and floor slabs safety factors were computed via *allowable stresses method* assuming, as declared in the design report: concrete allowable stress $\sigma_{ca} = 7.5$ N/mm^2 for sections subject to flexure and axial load, $\sigma_{ca} = 6.0$ N/mm^2 for pure compression, $\sigma_{sa} = 180$ N/mm^2 for the steel allowable stress and an homogenization coefficient, that nominally represents the ratio of steel to concrete Young modulus, n = 10.

Concerning the structural modeling for elastic analysis it has to be précised that the analyses and verify for the various typologies of horizontal slabs that are present in the building are performed autonomously from the rest of the structure; the slabs are modeled with continuous beams schemes (see § 3.1.3), while for the columns and beams a three-dimensional elastic model is considered.

The values for permanent and live loads to be applied on the slabs (unit loads for square meter) are assumed equal to those declared in the design report. Coherently with design practices in force at the time of construction of the considered building, the live load was considered to be contemporarily applied on all the slab's span, as well as on balconies.

The loads applied on the beams, in the three-dimensional model, are those deriving from the analysis of adjacent slabs. Beam element's weight is added to these loads, as well as the weight of the infills for perimeter frames.

In general, these loads do not coincide with those applied on the beams in the original design. In fact, having adopted a continuous beam scheme for the slab's analysis, the loads acting on internal frame's beams result to be increased while those for perimeter beams are decreased, though this latter effect is mitigated by the presence of balconies and of infills.

The forces and moments obtained with the three-dimensional model are used for the verify of columns and beams, while the actions on slab are derived by the continuous beam schemes that are specialized for the proper geometric configuration.

The resisting characteristics (axial load, bending moment and shear) are evaluated starting from the geometry and reinforcement of the structural elements, as inferred from original drawings.

3.1.1 Columns safety factor: axial load

In principle, the column elements are verified for a state of bending and axial load, with bending moment M and axial load N deriving from the linear elastic analysis on the three-dimensional model. The bending solicitation is generally modest, apart from perimeter columns; moreover the M to N ratio decreases more than linearly from upper to bottom storey. The columns characterized by the lower resistance/solicitation ratio are those of the first level, where the axial load solicitation largely prevails with respect to bending one, negligible. With little approximation, then, the columns' safety factor SF_c is computed for pure compression state as the ratio of the axial resistance N_{Ra} for the generic column section versus the corresponding axial solicitation

$$SF_c = \frac{N_{Ra}}{N} = \frac{\overline{\sigma}_{ca} A_c \cdot (1 + n\rho)}{N} \tag{4}$$

where A_c and ρ are the concrete cross section and the longitudinal steel ratio of the generic column and N is the axial load solicitation determined with the three-dimensional linear analysis. In the evaluation of N_{ra} the stress transfer from concrete to steel, due to creep, is neglected; however this phenomenon may become important only in very highly stressed or lightly reinforced slender concrete columns as evidenced in (Samra 1995, Kaltakci *et al.* 2007).

The bar diagram in Fig. 4 represents the Safety Factors for the columns at the first level. As it can be noted there are some columns that are not verified, being their SF_c lower than one. This circumstance is due the fact that, differently from N, the axial load used in the original design was determined as a function of permanent and live design loads and on the area of influence of the generic column, neglecting the structural continuity. This is a typical "structural fault" as evidenced in the (§ 2).

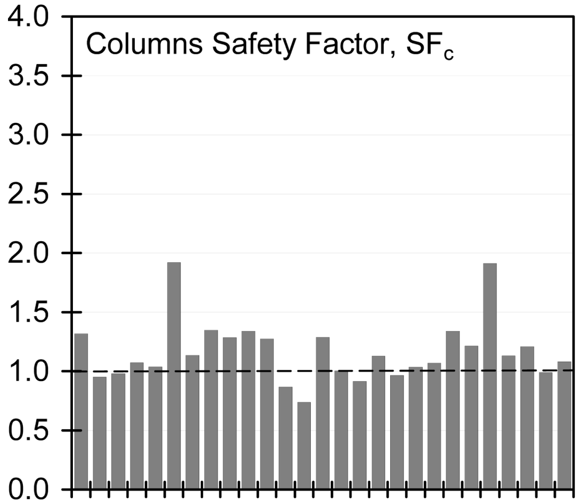


Fig. 4 Columns safety factors at the first level

3.1.2 Beams safety factors: bending and shear

The beams safety factors were determined for the bending and shear conditions.

The beam bending Safety Factor SF_{bb} is computed for either the central and edge sections of each beam (subjected to higher flexural solicitation) and is defined as the ratio of the resisting moment M_{Ra} versus the corresponding solicitation M

$$SF_{bb} = \frac{M_{Ra}}{M} \tag{5}$$

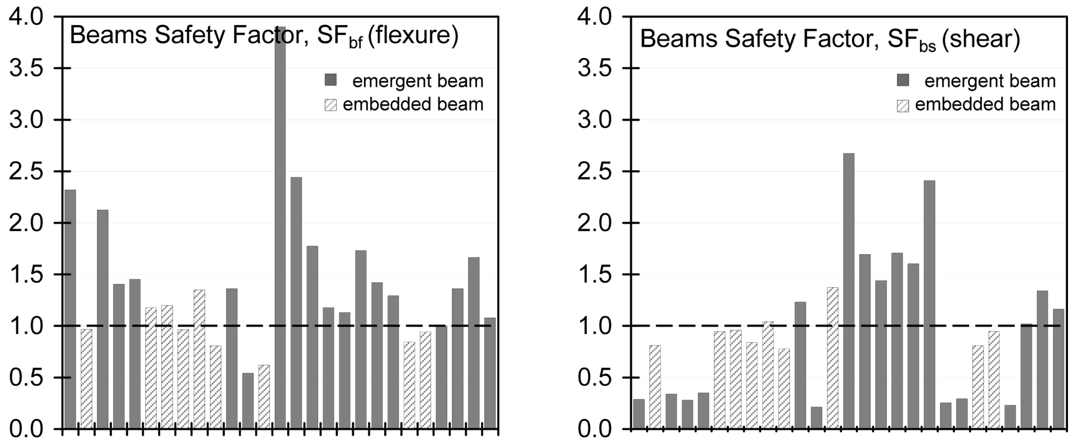
where

$$M_{Ra} = \min\{\sigma_{ca} b \cdot 0.5x \cdot d^*;\ \sigma_{sa} \cdot \rho A_c \cdot d^*\}$$

with d^* the flexural level arm, x the neutral axis deepness, ρ the longitudinal steel percentage (in tension), A_c the concrete section and M the flexural solicitation determined with the three-dimensional linear analysis.

Fig. 5(a) represents SF_{bb} at the first level. As it can be seen some of the beams are not verified ($SF_{bb} < 1$). This other negative outcome is again due to the incorrect evaluation of design loads: in fact, although in the original design the bending moments for beams were evaluated based on a continuous beam scheme, that theoretically should have furnished higher moments with respect to those evaluated on a three-dimensional scheme, still the unit loads transferred by the floor slabs to the beams were determined based on the simple influence area, neglecting the slab's continuity. This approximation could lead to more than 25% underestimation of the actual forces. As a confirmation of this aspect, it can be noted that most of the not-verified beams ($SF_{bb} < 1$) are embedded ones, that are present in internal frames, where the neglecting of the slab's continuity in design phase leads to an underestimation of the loads. The sole emerging beam characterized by a safety factor lower than one (the lowest SF for beams in flexure) is characterized by the presence of a balcony, whose load was not properly considered in the design phase (design error).

The beam shear safety factor SF_{bs}, for each of the edge sections, is computed as



(a) (b)

Fig. 5 Beams safety factors (a) flexure, (b) shear

$$SF_{bs} = \frac{V_{Ra}}{V} \tag{6}$$

where V_{Ra} is the shear resistance in the considered section and V is the corresponding solicitation. It has to be evidenced that the transversal reinforcement in all the beams is just a minimum amount (see § 2) and was not designed to absorb shear forces. In fact, in compliance with the allowable stresses method, the shear resistance could be entirely devolved upon the concrete if the transversal section was subject to low values of tangential stress τ, and upon the Ritter-Mörsch mechanism, involving the shear reinforcement, if a threshold τ value (τ_{co} = 0.6 N/mm^2) was exceeded.

In the first case, V_R was computed as the shear resistance of the un-cracked concrete section, see Eq. (1).

If $\tau > \tau_{c0}$ the shear solicitation should bear upon the shear reinforcement and was evaluated as

$$V_{Ra} = 0.9d \cdot \frac{A_{sw}}{s} \cdot \sigma_{sa} \tag{7}$$

where A_{sw} is the transversal steel area and s is the stirrup spacing.

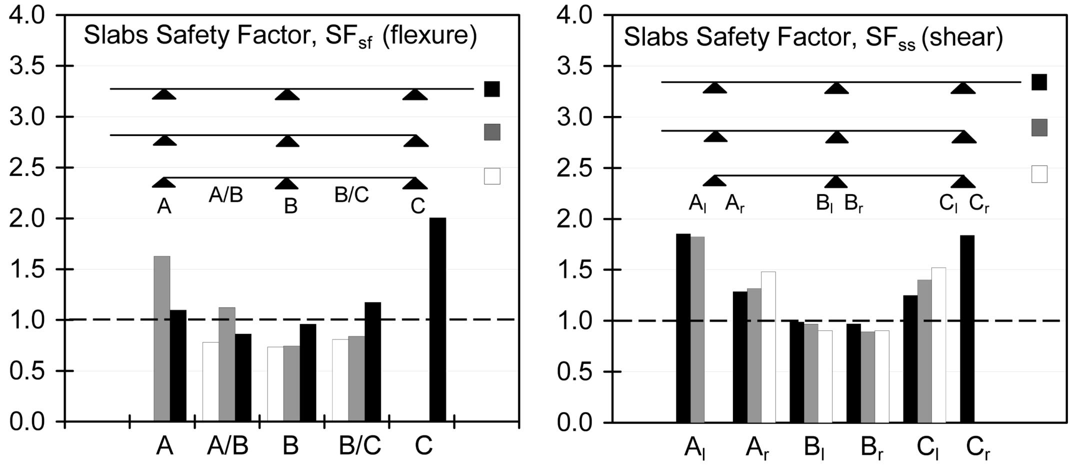
Some clarifications are needed for the embedded beams. In fact, differently from the design phase, in the verify of the beams the tangential stress is determined referring to an effective base and not to the nominal one, b; the effective base is determined as $b_{eff} = b_c + 2s$ with b_c the transversal dimension of the column intersecting the beam and s the slab's height. Moreover, the shear resistance for embedded beams is always evaluated as for elements not reinforced in shear; in fact, the two braces of the existing stirrups are always external with respect to the effective base. As a consequence, the shear safety factor for embedded beams could be evaluated as the ratio of τ_{co}/τ with τ the tangential stress determined from the analysis.

Fig. 5(b) represents SF_{bs} at the first level; as it can be seen great part of the elements are characterized by safety factors lower than one, meaning that they are far under the safe condition. However, it has to be evidenced the different behavior of embedded beams with respect to emergent ones. The former, in fact, are characterized by a lower variability of the safety factor; in such case, the negative outcome of the verify is imputable to both the diversity of the acting loads (slab's continuity) and of the analysis model (three dimensional frame instead of continuous beam). On the other hand, the emergent beams are characterized by a high variability of the safety factor; this is due to the different way to compute the shear resistance depending on the tangential stress. In fact, because the shear reinforcement was not properly designed, the shear resistance in the post-cracking state (7) is sensibly lower with respect to the one of a pre-cracking condition (1). As an example, the beam (3-4), having transversal section (250 × 500)mm and transversal reinforcement 2 braces ϕ6/200 mm stirrups, is characterized by a pre-cracking shear resistance $V_{Ra} = 63.45$ kN and a post-cracking one $V_{Ra} = 21.32$ kN. This circumstance determines an abrupt lowering of the shear resistance for tangential stresses higher than τ_{co}. In fact, the emergent beams for which $\tau > \tau_{co}$ are characterized by a safety factor significantly lower than one (minimum $SF_{bs} = 0.20$), while those with $\tau \le \tau_{co}$ have $SF_{bs} \ge 1$.

3.1.3 Slabs safety factors: bending and shear

For what concerns the floor slabs, three model schemes were adopted in order to compute the bending and shear forces: first model corresponds to the case of two balconies at the slab ends, as is for the slab adjacent to beam (1-7-15) in Fig. 2, second model corresponds the case of one balcony at the slab ends and the third one corresponds to the case on no balconies. The safety factors were, again, computed as the ratio of bending or shear resistances versus the relative external actions as determined with the proper model scheme.

The bar chart in Fig. 6(a) represents the floor slab bending safety factors SF_{fs} for the three considered schemes; a different bar hatching is associated to each scheme. SF_{fs} at slab edges (A, B, C) and at mid-span (A/B and B/C) were computed, as it is shown with respect to the model schemes sketched in figure. Fig. 6(b) shows the minimum shear safety factors SF_{ss}, corresponding to a shear resistance computed via Eq. (1), where the base b is equal to 0.20 m for a 1.00 m wide slab strip and the effective height d is equal to 0.20 m (0.22 m minus the 0.02 m bar cover). Al, Ar, Bl, Br, Cl, Cr in figure indicate left and right position with respect to A, B, C, respectively, for the



(a) (b)

Fig. 6 Slab safety factors (a) flexure, (b) shear evaluation of slab shear safety factor in each of the considered scheme. Obviously in the third scheme Al and Cr are not considered, as for Cr in the second scheme.

As it can be seen from Fig. 6(a) and Fig. 6(b), both the bending and shear safety factors for the slabs are, in some cases, lower than one. For what concerns bending, the lowering of the safety factor is due to the adoption of simplified models in the design phase, that often underestimate real forces. Regarding shear, again, the neglecting of the floor slab continuity in the design phase, lead to an underestimation of the effective shear solicitation with the consequent design un-conservativeness.

3.2 The nonlinear analysis: a parametric study

In order to investigate on the building's effective bearing capacity a number of nonlinear analyses were performed at the element's level. Applying basic principles of plasticity theory to the beams, the slabs and the columns of the structural system for different failure mechanisms (flexure and shear for the beams and slabs, axial load for columns) the minimum live loads multiplier (permanent ones remaining unchanged) that brings each element to failure is determined.

In particular, for the slabs, the live load acting on the elements q_k is directly inflated by a factor λ up to the attainment of the failure condition. The initial permanent and live loads, the latter to be amplified by the factor λ, acting on the beams and on columns are determined starting form the results of the linear elastic analysis; initial loads on the beams derive from the slab's elastic analysis, while columns loads are those of the three-dimensional elastic analysis. Plastic redistribution on slabs and on beams is not accounted for.

In such a way it is possible to evaluate, in an approximate manner, the live load multiplier λ for

Table 3 Parametric nonlinear analyses: combinations of concrete compressive strength f_c and steel yield stress f_y

Case #	Concrete strength, f_c (MPa)	Steel, f_y (MPa)
1		325
2	5	370
3		415
4		325
5	10	370
6		415
7		325
8	15	370
9		415
10		325
11	20	370
12		415
13		325
14	25	370
15		415

each element, independently from the rest of the structure; the multiplier for the entire structure is evaluated as the minimum among the one calculated for all the elements.

In the following, the live load multiplier are evaluated for each element typology with varying values of the material strengths (concrete and steel) and the minimum value for the entire structure are shown. Compatibly to the values of test samples extracted from existing buildings, whose mean value of cylindrical strength is approximately $f_c = 15$ MPa (Cosenza *et al.* 2006), the concrete cylindrical strength f_c is considered to vary in the range (5-25) MPa. For the steel, the considered yield values are f_y = (315, 370, 415) MPa for Aq50 steel typology, that is the steel used in the studied building. Considering different combinations of f_c and f_y, a number of 15 analyses are performed, as listed in Table 3.

It can be observed that, according to the prescriptions of the time codes on minimum ratios of material strengths versus design tensions, the live load multipliers for $f_c = 20$ MPa and $f_y = 370$ MPa approximate the minimum condition for code conforming materials.

The failure condition for the columns corresponds to the attainment of the maximum compressive strength N_R of the column. In particular, the live load multiplier λ^c corresponding to failure condition is defined for the load equating N_R

$$N_G + \lambda^c N_Q = N_R = f_c A_c + f_y \rho A_c \tag{8}$$

Hence

$$\lambda^c = \frac{N_R - N_G}{N_Q} \tag{9}$$

In the (7), (8) N_G and N_Q, evaluated with the linear elastic analysis, are the amount of axial load due to permanent and live loads respectively, A_c and ρ are the concrete cross section and the longitudinal steel ratio, f_c and f_y are concrete compressive strength and steel yield stress respectively.

The minimum live load multiplier for beams in flexure is determined in the hypothesis that the plastic mechanism forms with two hinges at the beam ends and one hinge in the span, whose position corresponds to the section of maximum bending moment as evaluated with the linear analysis. The live load multiplier λ_{fl}^b for the plastic mechanism formation is such that (see Fig. 7)

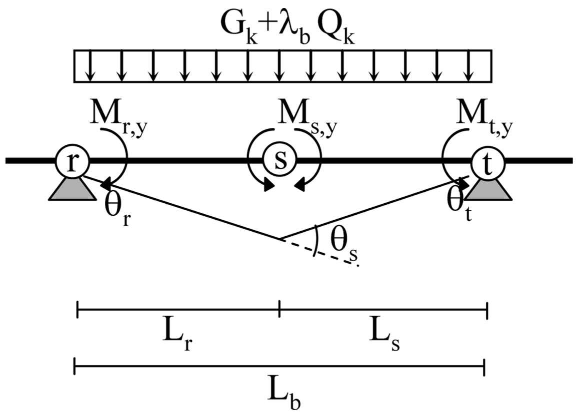


Fig. 7 The beam scheme used to compute λ_b

$$(G_k+\lambda_{fl}^b Q_k)\left(\frac{L_r^2}{2}+\frac{L_r L_t}{2}\right)\cdot\theta=\left[M_{r,y}+M_{t,y}\frac{L_r}{L_t}+M_{s,y}\left(1+\frac{L_r}{L_t}\right)\right]\cdot\theta \tag{10}$$

G_k and Q_k are permanent and live loads acting on the beam, $M_{r,y}$, $M_{s,y}$ and $M_{t,y}$ are the yielding moments in section r, s, t respectively, θ is the cinematic rotation of the rigid beam in the plastic-mechanism and L_r and L_t are the distances of section s with respect to the beam edges r and t, respectively. The yielding moment for the generic section i, $M_{i,y}$ is determined with usual sectional analysis approach (Priestley *et al.* 2007); alternatively simplified expressions may be used, as in (Fardis 2009). Eq. (10) is evaluated in the hypothesis of an EPP behavior for the section's moment curvature relation; in particular, initial stiffness is the secant stiffness to yielding point; this way it is hypothesized that the section failure, corresponding to ultimate curvature, is attained after the plastic mechanism development (see Fig. 7).

Hence

$$\lambda_{fl}^b=\frac{1}{Q_k}\left[\frac{2}{L_r^2+L_r L_t}\left(M_{r,y}+M_{t,y}\frac{L_r}{L_t}+M_{s,y}\left(1+\frac{L_r}{L_t}\right)\right)-G_k\right] \tag{11}$$

If $L_r = L_t = (L_b/2)$ with L_b the beam's length, the (11) may be specialized in the following simpler expression

$$\lambda_{fl}^b=\frac{1}{Q_k}\cdot\left[\frac{4}{L_b^2}\cdot(M_{r,y}+M_{t,y}+2M_{s,y})-G_k\right] \tag{12}$$

Fig. 8 plots the minimum live load multipliers λ due to beams flexural mechanism and columns crushing for the 15 analysed cases. The failing elements, those having the lower λ, are column 13 and beam (11-12), see Fig. 2. As it can be observed the prevailing failure type, the one corresponding to the lower λ value between the considered ones, changes with varying material properties: the columns are those failing for low concrete strength and, vice versa, the beams are the weaker elements for concretes of higher strengths. As it appears evidently from the structure of the

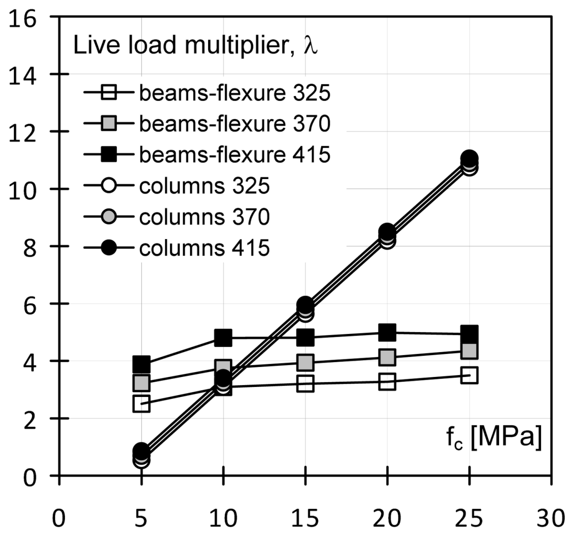


Fig. 8 Live load multiplier λ due to beams flexural mechanism and columns crushing

(8)-(9) and (12) the concrete strength f_c has a large influence on the load multiplier for the columns λ^c, while it is far less important for those of beams λ^b_{fl}, that are mostly dependent on f_y (through the yielding moment $M_{j,y}$ in the generic j section).

The shear failure condition for beams corresponds to the attainment of the shear resistance V_R and the corresponding live load multiplier λ^b_{sh} is defined for the load equating V_R

$$(G_k + \lambda^b_{sh} Q_k)\frac{L_b}{2} C_b = V_R \tag{13}$$

with symbols already defined. Hence

$$\lambda^b_{sh} = \frac{1}{Q_k} \cdot \left[\frac{2}{C_b L_b} \cdot V_R - G_k\right] \tag{14}$$

In particular, it is hypothesized that the shear diagram is characterized by an invariant null point, that is the same of the one computed from elastic analysis. In such case, in order to obtain the minimum load multiplier, the maximum shear force, corresponding to the generic load $(G_k + \lambda^b_{sh} Q_k)$, should be inflated by a beam's continuity coefficient $C_b \geq 1$, that allows determining the shift from the mid-span of the zero shear section with respect to the emi-symmetric condition; theoretically, for non-sway frames, the continuity coefficients can assume a value up to 1.25.

Concerning the shear resistance V_R the same considerations of the elastic analysis apply. The shear resistance for embedded beams is evaluated as for beams not reinforced in shear, while for the emergent beams the existing transversal reinforcement is considered. Hence, for the emergent beams the shear resistance V_R is determined as suggested in (EC2 2004)

$$V_R = \min(V_s, V_c) \tag{15}$$

with

$$V_s = 0.9 \cdot d \cdot \frac{A_{sw}}{s} \cdot f_y \cdot \cot\theta \tag{16}$$

and

$$V_c = 0.9 \cdot d \cdot b \cdot 0.5 f_c \cdot \cot\theta / (1 + \cot^2\theta) \tag{17}$$

In the (16) and (17), written for the case that the transversal bar inclination with respect to the horizontal axis is 90°, d and b are the beam effective depth and base, A_{sw} and s are the transversal steel area and spacing, θ is the inclination of the concrete struts with respect to horizontal axis, with the limitation ($1 \leq \cot\theta \leq 2.5$).

In general, the $\cot\theta$ is computed imposing the contemporary crisis of the struts in compression and the yielding of the transversal reinforcement, i.e., equating the (16) and the (17). If the latter condition is attained for $\cot\theta > 2.5$, that means for low mechanical percentages of the reinforcement, the crisis is governed by the sole transversal reinforcement.

Vice versa, for embedded beams, the shear resistance may be calculated adopting formulations corresponding to elements that do not have shear reinforcement (Nehdi and Greenough 2007, Song and Khang 2010). Here the proposal of Eurocode 2 is adopted (EC2 2004)

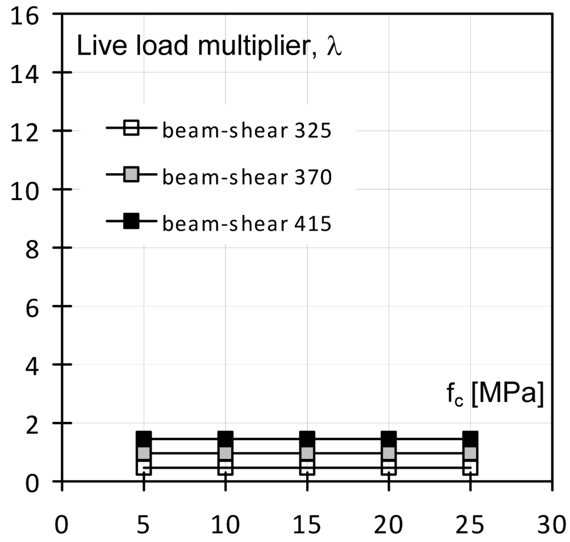


Fig. 9 Live load multiplier λ for the beams shear failure condition (minimum shear reinforcement)

$$V_R = [0.18 \cdot k \cdot \sqrt[3]{100\rho f_c}] \cdot b \cdot d \geq v_{\min} \cdot b \cdot d \tag{18}$$

where

$$k = 1 + \sqrt{200/d} \leq 2 \tag{19}$$

$$v_{\min} = 0.035 \cdot k^{3/2} \cdot f_c^{1/2} \tag{20}$$

with symbols already defined.

Fig. 9 shows the load multipliers λ due to beams shear failure: these values are computed considering the minimum transversal reinforcement as inferred by design drawings. It is important to note that such low values are basically due to the threshold type design that was performed with the allowable stresses method: elements characterized by a tangential stress lower than a given threshold value ($\tau \leq \tau_{co}$) were practically not designed in shear, and this could lead to low shear resistances according to nowadays codes. It is evident, from Fig. 9, that the minimum multiplier, defined by the crisis of emerging beam (15-16), is invariant with the concrete strength, while it grows with the steel yield stress and is always lower than 2. It is interesting to observe that for $f_y = 370$ MPa, approximately two times the allowable stress utilized in the design phase, the live load multiplier is about 1. It can be immediately verified that the multiplier for the beam (15-16), that was characterized by an allowable shear resistance V_{Ra} such to give a safety factor 0.2 (the minimum) in the linear elastic analysis, is inflated 5 times, considering both the minimum ratio (=2) of yielding to allowable stress for the steel, and the $\cot\theta = 2.5$. In other words, for a load condition corresponding to a service condition (studied with the linear analysis) the shear failure is attained, with the yielding of the transversal reinforcement.

Indeed, if the shear design was performed, the load multipliers would be significantly higher. Fig. 10 shows the live load multiplier for the case of transversal reinforcement designed with the

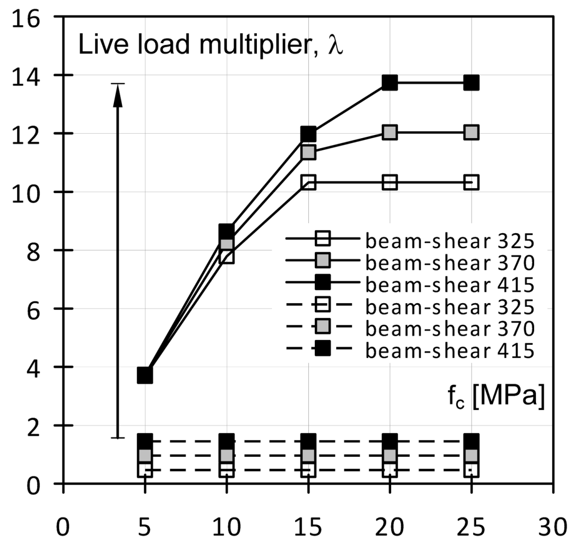


Fig. 10 Live load multiplier λ for the beams shear failure condition (designed shear reinforcement)

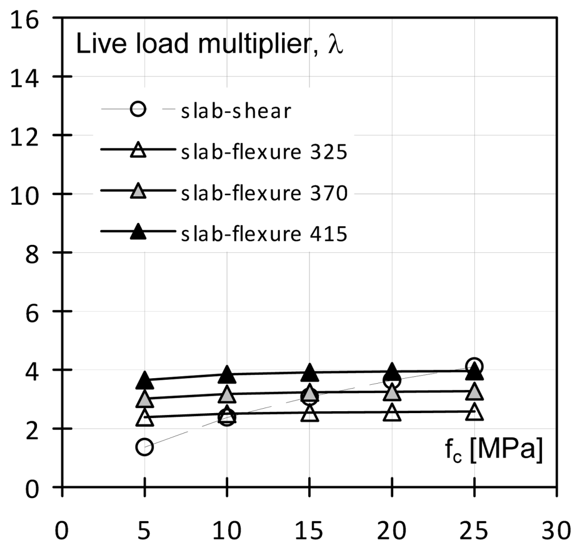


Fig. 11 Live load multiplier for the slabs: continuous lines are λ in flexure for varying f_y; dashed line is the λ for shear failure condition

allowable stress method: the stirrups area and spacing for this case is dimensioned so to bear the entire shear solicitation, independently from the threshold value τ_{co}. In such a case the load multiplier shows a completely different trend with respect to the previous case, growing both with the concrete strength and the steel yield stress. Only for higher strength concretes the load multiplier is independent from the concrete, reaching values up to 7 times higher with respect to beams not designed in shear (design error).

The horizontal slab load multiplier, λ_{fl}^{s} in flexure and λ_{sh}^{s} in shear, are found with the same approach described above for the beams, with the sole difference that the shear resistance is calculated with a formula corresponding to elements that do not have shear reinforcement (see Eqs. (18)-(20)).

Considering the expression (18) of the shear resistance for the slab it is evident that V_R does not depend on the steel yield stress f_y, while it is influenced by the concrete strength f_c. Fig. 11 shows the flexure and shear live load multiplier for the slabs.

Fig. 12, finally, show the minimum load multiplier among the considered failure mechanisms. In order to have an interesting confrontation among the different cases, the shear mechanism for the beams, that are affected by an evident design error, is excluded. In particular, considering the different combination of concrete strength and steel yield stress, a different failure mechanism hierarchy is attained, as shown by the dashed thick line in figure.

As it can be seen, for the case of low strength concrete a brittle type failure for excessive compression in the columns prevails in all cases. In general, for all the considered steels the failure hierarchy changes for concrete strengths between 5 and 10 MPa. In particular, for the 325 steel (f_y = 325 MPa) the type of crisis gradually passes from the first one to a brittle one due to slab's shear failure, to a ductile one (slab's in bending) for concretes characterized by f_c greater than 10 MPa. For 370 steel the behavior is analogous, with a larger f_c interval relative to the shear failure in

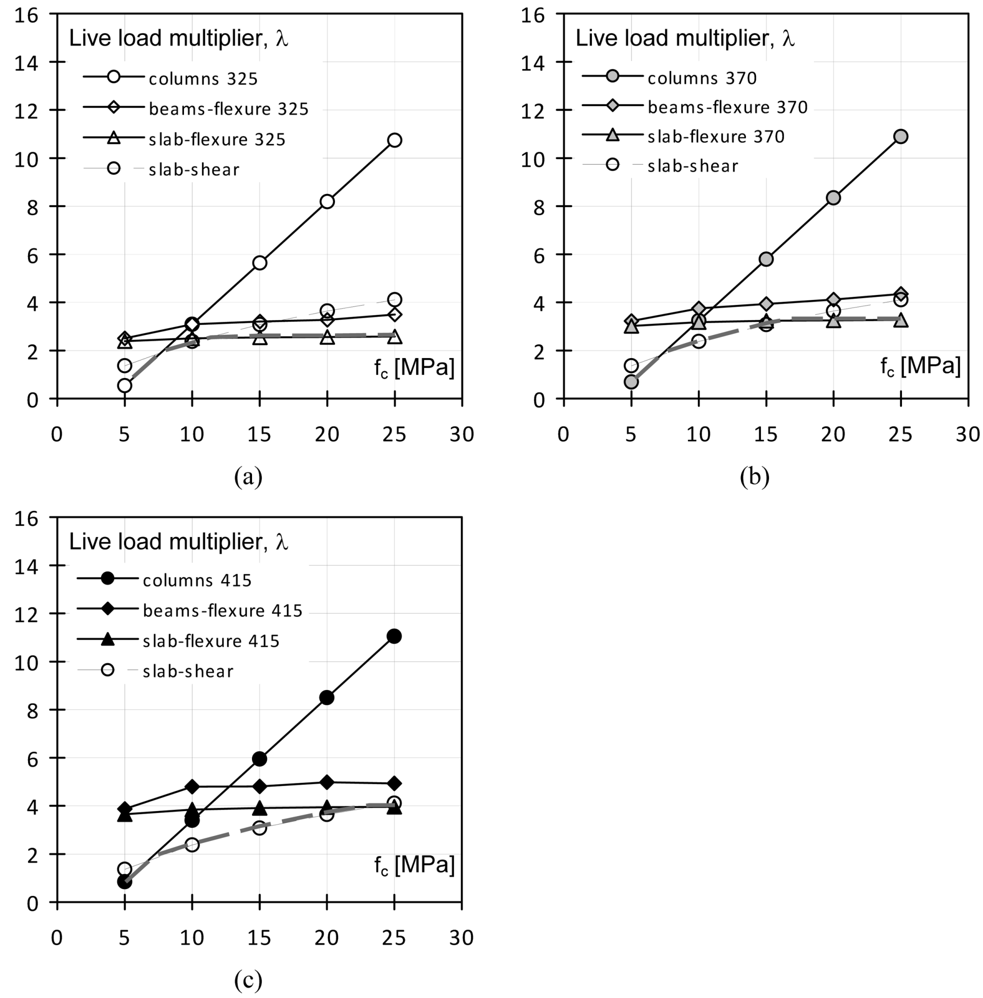


Fig. 12 Minimum λ values for the considered mechanisms with (a) f_y = 325 MPa, (b) f_y = 370 MPa, (c) f_y = 415 MPa

the slabs. Finally, for 415 steel, the type of failure is brittle for nearly all the f_c range, having the shift to a ductile type of failure (slab's failure) only for f_c around 23 MPa.

4. Conclusions

There are a number of structural faults that may be responsible for the lowering of the safety margin in R.C. constructions, such as inadequate or incautiously approximate design models, poor materials or execution defects.

This paper exemplifies the possible effects of these structural faults on the lowering of the safety margin by means of a detailed study of a an existing R.C. building, representative for mid-rise gravity load designed structures in southern Europe.

The linear elastic analysis helped locating the critical elements and allowed making a breakthrough in the understanding of potential causes of structural diseases: some elements, designed with simple element models, may suffer because of incorrect evaluation of bending/axial forces or for inadequate shear design.

Moreover, nonlinear parametric analysis was performed at the element's level, evaluating the minimum live load multiplier that brings beam, columns and slabs to failure for varying values of concrete strength and steel yield stress.

The nonlinear parametric analyses showed that, depending on material properties, the failure hierarchy is dominated by either brittle or ductile mechanisms. In particular, independently from the steel type, if the resistance of the brittle failing elements is not adequately supplied these elements dominate the failure mechanism through all the material properties range, as it happen with the beams that are not adequately reinforced in shear. Indeed, if the latter circumstance is avoided, the failure mechanism changes as a function of material properties. In general, the column brittle failures due to excessive axial load prevail for low strength concretes. For concretes of a higher strength, it is observed that, with increasing steel yield strength, the failure mechanism changes from a brittle one to a ductile one, with higher values of the load multiplier.

In order to generalize these observations it is necessary to further investigate on typical structural modeling simplifications and on the ratio of material properties versus design strengths.

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