Analytical simplified procedure for the evaluation of the RC buildings.

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SUMMARY:
Numerous existing buildings are located in seismic prone areas and many of them were not designed considering adequate seismic provisions. Thus, for safety reasons, it is necessary to assess their seismic vulnerability and to provide strengthening interventions if needed. To reduce gradually the seismic vulnerability, a prioritization list for the intervention is required. A new simplified mechanical procedure for the assessment of the seismic resistance of reinforced concrete structures is herein presented (FIRSTEP-RC). The types of structures that may be assessed are various: one-way frame, three-dimensional frame, shear walls, mixed frame and shear walls, precast frames. The procedure related to the study of the frame structures is herein presented and discussed. An example underlines the effectiveness and the simplicity of application of the proposed procedure.

Keywords: Seismic vulnerability, Reinforced concrete, Existing buildings.

1. INTRODUCTION

A large portion of Italy is considered to be seismically active, but the seismic classification has been updated only after seismic events. Since the issue of the OPCM 3274 (2003), the seismic classification has been extended to the whole national territory with varying degrees of hazard. Due to these updates, the changes are significant, in fact, some areas that were previously considered to have low seismicity have now been classified medium to high hazard (Manfredi and Masi, 2009). As a consequence, several existing buildings are located in areas that were not classified as seismically active at the time of construction and so they were built without seismic criteria. Thus, the vulnerability of the existing buildings, in particular the public sector, has to be studied. A detailed analysis of all existing constructions, however, requires very high economical resources and so the problem may not be solved in the short term. The hugeness of the public building stock requires the adoption of expeditious methods that allow the identification of potentially critical buildings. It is necessary to define a prioritization list for the strengthening interventions so to allow a gradual reduction of the seismic vulnerability and to increase the social safety.

The available methods for the assessment of the seismic vulnerability of buildings are various and are based on different principles, in function of the result to be obtained. The most common methodologies are the "index-score" methods that associate a score to each building, in function of some parameters that describe the ability of the structure to survive to earthquakes. Among these methods, the most well known is that proposed in FEMA154 (2002), as well as those methods inspired to the Hazus procedure (FEMA 366, 2003). In these methodologies, a displacement capacity curve is associated to each building, according to the structural typology, the number of floors and the year of construction. The ratio between the capacity and the demand (in term of spectral displacement) represents the ability of the structure to support the expected earthquake and allows to have an index of comparison among structures. Other simplified mechanical procedures allow to estimate the resistance of the structure and so they are suitable to achieve a priority list based on a quantitative parameter. The simplified mechanical methods developed until now for the evaluation of reinforced concrete structures are limited to some structural types (plane frame, shear walls). Further, the number of data required for the assessment is too high. Among these methodologies, the “VC” procedure (Dolce e Moroni, 2007) allows to quantify the base shear resistance of RC frame structures, assuming a shear-type behaviour for each column and a “soft storey” collapse.

In this paper, a new simplified mechanical procedure is proposed. This method is called "FIRSTEP-RC", that is the acronym of First Step Evaluation Program for Reinforced Concrete structures (Franceschinis, 2012). It was developed to assess the seismic resistance of all the types of reinforced concrete structures that may be found in the built heritage: one-way frame, three-dimensional frame, shear walls, mixed walls and frames, frames with infill walls. In FIRSTEP-RC the assessment of the structure takes into account the various types of failures that may occur: shear collapse, combined flexure and axial force collapse, local collapses. At the end of the process, the peak ground resisting acceleration of the building, for each principal direction (a_{ux}, a_{uy}), is carried out. The lesser one (a_u) is taken to define the seismic safety index of the structure (I_s), in relation to the seismic acceleration expected (a_{exp}):

$$I_s = a_u / a_{\exp} \; . \tag{1.1}$$

This procedure was developed during the course of the Assess Project (2008-2011), financed by the Italian Region Friuli Venezia Giulia and finalized to define a prioritization ranking to make interventions in school buildings so to reduce the seismic risk.

2. GENERAL PROCEDURE

Various structural types of reinforced concrete buildings may be assessed with the FIRSTEP-RC procedure: one way frame, three dimensional frame, precast frame, shear walls, mixed frame and shear walls, frames with masonry infill walls. The purpose is to develop a procedure characterized by rapidity in application, in which the number of data required for the evaluation is limited. Thus, some analytical simplifications are needed. The most important simplification is to study each building with reference to the resistant elements at the ground floor only. To make reliable this simplification and to identify the most suitable schematizations of the various structural types, in terms of stiffness and resistance so to take into consideration the influence of the upper levels on the behaviour at the ground floor, some specific studies and parametric analysis were performed.

The results found with this procedure are reliable for all the buildings in which the section of the elements does not vary significantly from one floor to the others, and the sections of the beams are the same at each level of the structure. The FIRSTEP-RC procedure, thanks to these simplifications, requires the graphic input of the resisting element at the ground level only, the beams and the floors that pertain to each vertical resisting element. A graphic interface was implemented in order to speed up the input of the needed data starting from a CAD geometry. These data allow the numerical procedure to evaluate the influence area and the load that pertains to each vertical element, and the centroid of the building. The steel percentage in each section is evaluated by a simulated design procedure in function of the influence area of the element and the historical code provisions. The beam sections are defined in function of the type found during the survey to the building: as for example deep beam or shallow beam. The dimensional ratio that characterizes each type of section was obtained through a parametric study.

The FIRSTEP-RC procedure concerns two parts that may be summarized in the two functional blocks in Fig. 2.1. The former refers to the analysis of the structure and the latter to the evaluation of the resistance of members. In the analysis, a unitary force, applied on the centroid of the building, is distributed among the resisting elements at the ground floor in function of their stiffness and position (f_{ie}). A dedicate procedure allows to define the stiffness of each element. In case of structures made of frames and shear walls, a methodology to accurately estimate the interaction between the frame and the equivalent shear wall was studied. These interactions become much important at the increase of the number of floors of the building. In particular, the procedure imposes that the displacement at each level be the same for the frame and the equivalent wall and evaluates the fraction of base shear force that pertains to the frame ($F_F = \alpha \cdot F_{TOT}$) and to the walls ($F_W = (1-\alpha) \cdot F_{TOT}$). The coefficient α allows to estimate the stiffness of the frame (K_F), with respect to the total stiffness of the structure ($K_F + K_W$), taking into consideration the interaction with the equivalent wall. Thus, it permits to establish how to modify the stiffness of the columns of the frame (K_F^{MOD}) and of the walls (K_W^{MOD}) in order to correctly assess the distribution of the base shear force. The distribution of the force made with the procedure takes into consideration the presence of a twisting effect in case of eccentricity between the stiffness centre and the centroid. Furthermore, the procedure allows to considerate, at the ground floor, the presence of squat columns. These irregularities may lay, in fact, to a relevant eccentricity.

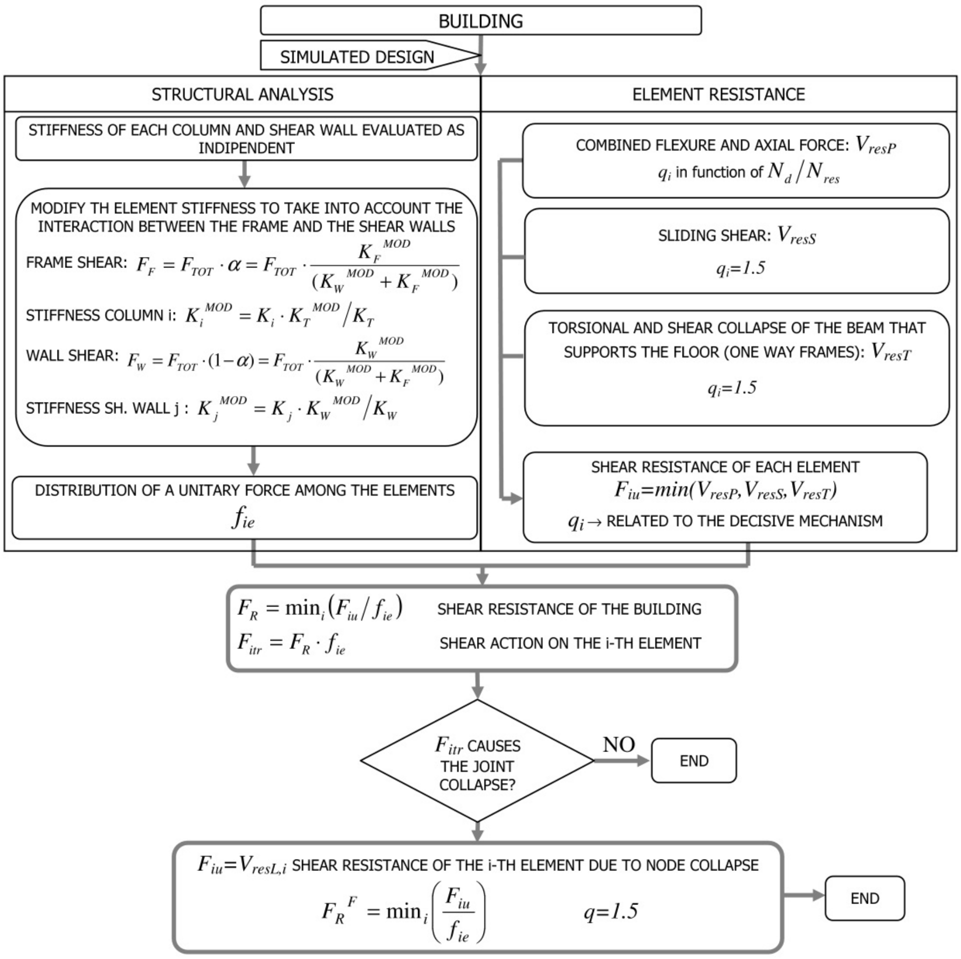


Figure 2.1: Flowchart of the general procedure of FIRSTEP – CA.

The second functional block refers to the evaluation of the resisting shear force of each element (F_{iu}). This coincides with the minimum of the shear resistance force related to the most common type of collapse mechanisms: combined flexure and axial force collapse (V_{resP}), sliding shear collapse (V_{resS}) and local collapse due to combined twisting and shear in the beam that supports the floor, in case of one way frame analysed in the weak direction (V_{resT}). For each principal direction, the total base shear resisting force of the building (F_R) coincides with the base shear force that causes the collapse of the first element that reaches its ultimate limit state:

$$F_R = \min(F_{iu} / f_{ie}). \tag{2.1}$$

A behavior factor is defined for each vertical element (q_i), in function of the dominant collapse mechanism. The value of the behavior factor is defined in the following paragraphs for the cases of combined flexure and axial force, sliding shear and local brittle mechanisms. For each principal direction, the behavior factor of the building q, coincides with the lower value that derives from the ratio between the structural factor q_i of each vertical element and its load rate:

$$q = \min_i \left[q_i \cdot F_{iu} / (F_R \cdot f_{ie}) \right]. \tag{2.2}$$

At the end of this procedure, the shear force that pertains to each element is known ($F_R \cdot f_{ie}$). The following step is to evaluate if these forces may lead to the brittle collapse of one or more beam-column joints. If it is the case, the resisting force of the building has to be reduced (F_R^F) and the behavior factor is taken equal to 1.5 (brittle collapse - NTC 2008). The resisting acceleration is evaluated through the equation:

$$a_u = \frac{F_R{}^F \cdot q \cdot g}{W \cdot S \cdot F_0} \tag{2.3}$$

in which a_u is the peak ground resisting acceleration at the base of the building, q is the behavior factor, W/g is the mass of the building, S is the soil factor, F_0 is the building amplifier factor of the seismic action. For brevity, only the aspects related to the study of the frame structures are presented the following.

3. STRUCTURAL ANALYSIS: SCHEMATIZATION OF THE FRAMES.

To analyze the real structure, the stiffness of each vertical element is separately evaluated in each of the two principal directions. In this step, the behavior of the element (column or shear wall), the base restraint (fixed or hinged), the grade of restraint at the upper end of the column at the ground floor and the number of levels of the considered element are taken into account. The models adopted to analyze the frame structures refer to the portal method (Fig. 3.1a). This method assumes that, for a group of horizontal forces applied to the frame, the diagram of the bending moment has his contraflexure points in the middle of each column and of each beam. Placing hinges in these points, the structure, initially hyperstatic, may be studied as a sum of isostatic substructures. The generic column, in function of its stiffness, has to support a rate ψ of the total base shear force (Fig. 3.1b).

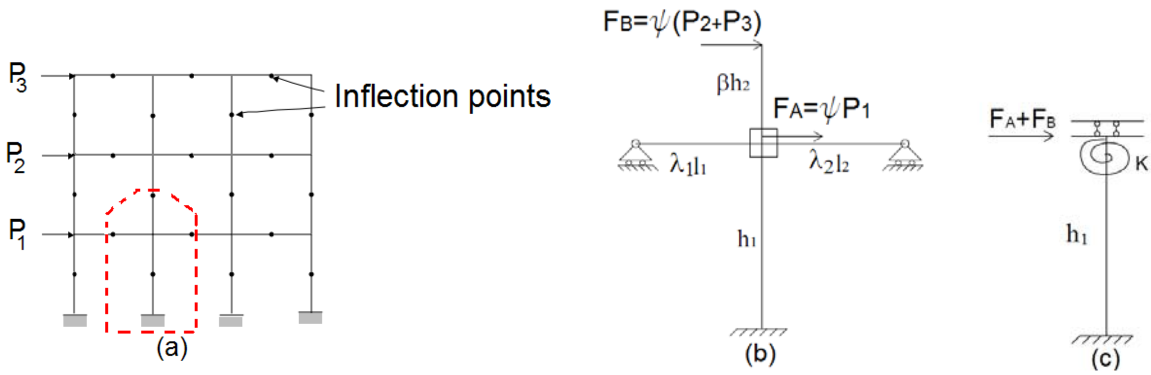


Figure 3.1: Portal method: (a) Multi-level frame with contraflexure points; (b) forces and significant dimensions on the generic column; (c) structural simplification for the column with fixed end at the base and with an elastic rotational restraint at the top.

In the FIRSTEP-RC procedure, the position of beam hinges ($\lambda_1 l_1$, $\lambda_2 l_2$) and column hinge at the first floor (βh_2) are not placed in the middle of the elements, as in the portal method, but result from parametric analyses performed on frame structures with different number of floors, length and sections of the beams and floors (Fig. 3.1b). The influence of the horizontal beams and the presence of the upper floors are taken into account through a spring with K rotational stiffness, which leads to the definition of the simplified schematization in Fig. 3.1c. In Fig. 3.1a and Fig. 3.1b, the force F_A corresponds to the quantity $\psi \cdot P_1$ of Fig. 3.1b, instead F_B is equal to $\psi(P_2+P_3)$. For each element, to evaluate the stiffness K of the rotational spring, a distribution of horizontal loads concentrated at each floor and varying in proportion to the height, is considered. The interstory height and the mass of the floor are assumed constant at each level. A unitary total force is applied to each element ($F_{tot}=1$). Thus, the force at the *i*-th level (F_i) is assumed proportional to the number n_i of the plan: $F_i=Cn_i$, with C constant. The value of the constant C is equal to:

$$C = 2 \cdot \left[(n_{tot} + 1) \cdot n_{tot} \right]^{-1}, \tag{3.1}$$

where n_{tot} is the total number of floors. With these assumptions, the force F_A (Fig. 3.1b) is equal to C, instead the force F_B is equal to the sum of the forces due to the upper floors ($F_B=1-C$).

This procedure allows to model all the restraint configuration that may characterize the columns at the ground floor of a real frame (Fig. 3.2): (a) fixed end at the base, elastic rotational restraint with free translation at the top, (b) hinge at the base, elastic rotational restraint with free translation at the top, (c) fixed end at the base and hinge with free translation at the top. The first case (a) represents, in the plane, the configuration of frames that has a sound foundation and bearing beams at the upper floors. The second case (b) concerns the cases with a weak restraint at the foundation level (weak footings or plane of action perpendicular to the direction of the beam foundation) and a bearing beam or a one way slab at the upper levels. The last case (c) represents the configuration of inverse pendulum structures (prefabrication or columns with strong foundation but very weak beams at the top. The rotational spring at the top of the element in the cases (a) and (b) of Fig. 3.2, represents the rotational inertia guaranteed by the beams that pertain to the joint; it takes into account the degree of restraint offered by the beams and the number of levels of the vertical element.

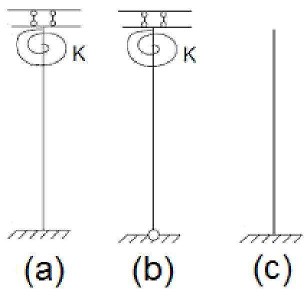


Figure 3.2: Representative models of actual cases.

3.1. Stiffness of a column fixed at the base and with an elastic rotational restraint at the top.

The problem concerns the model of the column in Fig. 3.1c. In this figure, $\lambda_1 l_1$, $\lambda_2 l_2$ and βh_2 are respectively the lengths of the horizontal beams and of the column at the first level up to the point of zero moment (hinge). The horizontal loads acting on the structure are characterized by the rules described in the previous chapter ($F_i=Cn_i$, with C constant). The stiffness of the rotational spring K is obtained imposing the same displacement at the top of the systems evidenced in Fig. 3.1b and Fig. 3.1c:

$$K = \left[(K_{l1} + K_{l2}) \frac{h_1^2}{2} - (1-C) \cdot h_1 \cdot \beta \cdot h_2 \cdot \frac{K_{h1}}{4} \right] \cdot \left[\frac{h_1^2}{2} + (1-C) \cdot h_1 \cdot \beta \cdot h_2 \right]^{-1} \tag{3.2}$$

$$K_{l1} = \frac{3EI_1}{\lambda_1 l_1}, K_{l2} = \frac{3EI_2}{\lambda_2 l_2}, K_{h1} = \frac{4EI_{h1}}{h_1}, \tag{3.3}$$

where I is the second moment of area of the cross section and E is the Young modulus, the subscripts 1 and 2 identify the beam at the left and right of the joint, respectively. On observe that the value of the stiffness K depends on the stiffness of the beams and the load distribution along the height. The stiffness of the simplified element in Fig. 3.1 is equal to:

$$K_{element} = \frac{12EI_{h1}}{h_1^{\,3}} \cdot \left[-2 + \left(3h_1 + \frac{6EI_{h1}}{K} \right) \cdot \left(\frac{K}{Kh_1 + EI_{h1}} \right) \right]^{-1} . \qquad (3.4)$$

The bending moment diagram on the column at the ground floor is linear and the point of zero moment (h_F) is evaluated with the relationship:

$$h_F = \left[\left[-2 + \left(3h_1 + \frac{6EI}{K} \right) \cdot \left(\frac{K}{Kh_1 + EI} \right) \right]^3 + 2 \right] \cdot \frac{h_1}{6} . \qquad (3.5)$$

3.2. Stiffness of a column with a hinge at the base and an elastic rotational restraint at the top.

The modelling is similar to that in Fig. 3.1 with a hinge at the base. The procedure for the evaluation of the rotational spring K is the same adopted in the previous chapter. This modelling refers to all the frames with a weak restraint at the base. The resistant mechanism to horizontal actions which occurs in this case is guaranteed by the slab resistance to bending and by the combined shear-torsional strength of the bearing beam. The FIRSTEP-RC procedure takes into account both the flexural stiffness K_{flex}, due to the beams l_1 and l_2 that represent the one way slabs, and the torsional stiffness of the beam that supports the slabs (K_{glob}) (Fig. 3.3a). The two rotational springs are in series and thus the equivalent stiffness becomes:

$$K = \left(\frac{1}{K_{glob}} + \frac{1}{K_{flex}} \right)^{-1} . \qquad (3.6)$$

To evaluate the torsional stiffness provided by the beam in global terms (K_{glob}), a uniform distribution of torques (m_t) on the beam was assumed, which represents the contribution of the floor joists. The constant distribution of torsional moment is not the real one, but leads to a good approximation of the torsional stiffness of the beam. The strategy is to impose, for the two systems in Fig. 3.3b, the same deformation energy due to the torsional moments:

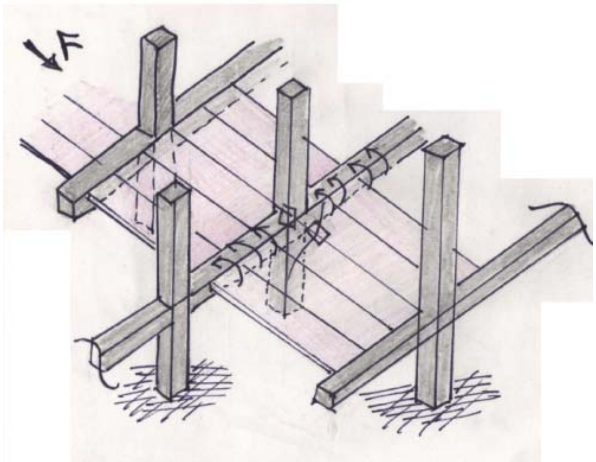
$$\int_0^a [m_t \cdot z]^2 \cdot \frac{1}{GI_p^*} dz = \left(M_i^* \right)^2 \cdot \frac{1}{K_{glob}} , \qquad (3.7)$$

where M_t^* is the torsional moment at the fixed end, $G{\cdot}I_p^*$ is the torsional stiffness of the beam in the uncracked state of the section. The distribution of the torque is assumed to be constant, thus the torsional stiffness of the spring is:

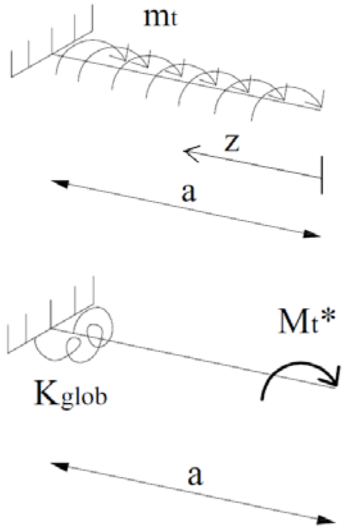
$$K_{glob} = 3 \cdot \frac{GI_p^*}{a} , \qquad (3.8)$$

where a is half of the beam length (Fig. 3.3b).

In this configuration, the bending moment diagram on the column at the ground floor results to be triangular with zero value at the base. The shear action V (constant) and the maximum bending moment on the element (M_{max}), are linked by the relationship: $M_{max} = h_F\ V$, where h_F, in this case, coincides with the height of the column (h_1).



(a)



(b)

Figure 3.3: (a) Stiffening effect of the slabs and the beam that supports them in respect to the action of horizontal forces; (b) schemes to evaluate the torsional moment on the beam.

3.3. Stiffness of a column fixed at the base and hinged at the top.

In the case of precast frame buildings with columns fixed at the foundation and hinged at the floor levels (inverted pendulum), the following changes with respect to the model in Fig. 3.1 were done. In particular, it is assumed that the hinges are located at the ends of the beams. The horizontal loads acting on the structure are characterized by the same rules adopted in the previous chapters ($F_i=Cn_i$, with C constant). The translational stiffness of the element is evaluated by the following relationship:

$$K = \frac{F_{tot}}{y(h_1)} = \left(\frac{M_{tot} \cdot h_1^3}{3 \cdot E \cdot I_{h1}} + \frac{h_1^3}{3 \cdot E \cdot I_{h1}} + \frac{h_1 \cdot \chi}{G \cdot A_{h1}} \right)^{-1}, \tag{3.9}$$

$$M_{tot} = C \cdot \sum_{i=2}^{n_{tot}} \left(h_i \cdot \sum_{j=i}^{n_{tot}} n_i \right). \tag{3.10}$$

In the equations, h_i is the interstorey height of the i-th level, A_{h1} is the cross section of the element, G is the shear modulus of concrete, χ is the shear factor of the section, F_{tot} is the sum of the forces applied on the element at each level ($F_{tot}=1$), M_{tot} is the overturning moment on the top of the column at the ground floor derived from the forces applied at the upper floors.

In this configuration, the position of the contraflexure point (h_F) in the equivalent element is assumed to be equal to that obtained by relating the maximum moment at the base of the element with the shear action in the same section:

$$h_F = \frac{M_{\max}}{T_{\max}} = C \cdot \sum_{i=1}^{n_{tot}} \left(h_i \cdot \sum_{j=i}^{n_{tot}} n_i \right). \tag{3.11}$$

4. BASE SHEAR RESISTANCE OF THE BUILDING

With the procedure described in the left block of the diagram in Fig. 2.1 the quote of the unitary shear force that pertains to each resisting element can be evaluated. To assess the resisting shear force of the building it is necessary to evaluate the resistance of each element in terms of shear action. The resisting shear force of each element is the minimum among those relative to the collapse due to the following mechanisms (Fig. 2.1): (1) combined flexure and axial force collapse (V_{resP}); (2) collapse due to sliding shear (V_{resS}); (3) collapse due to combined twisting and shear in the bearing beam that supports the slab, in case of one way frame considered in the weak direction (V_{resT}). The possibility of activation of a local collapse mechanism in beam-column joints (V_{resL}) is studied by the procedure through a check *a posteriori*.

4.1. Combined bending moment and axial force resistance.

The resistance of the element related to the collapse mechanism of combined bending moment and axial force is evaluated trough the study of the strain fields, defined in function of the axial force on the element. The position of the neutral axis and the resisting moment of the section (M_{res}), in the principal directions, are evaluated by the procedure in function of the axial force, the percentage of reinforcement in the cross section and the strain distribution at the ultimate limit state. The resisting shear force associated to this resisting moment is $V_{resP}=M_{res}/h_F$. The distance h_F correlates the bending moment acting on the element and the action of shear, it is evaluated in a different way for the three basic configurations in Fig. 3.2, as described in the previous Sections. The behavior factor associated to this type of collapse ("ductile") is linearly variable from 1.5 to 3, depending on the stress level of concrete (NTC 2008). In particular, for the *i*-th element, the behavior factor q_i is evaluated in function of the ratio between the axial force on the member N_{Edi} and the maximum axial resisting force N_{Resi}:

$$q_i = 3 - 1.5 \cdot N_{Edi} / N_{\mathrm{Re}si} \ . \tag{4.1}$$

4.2. Sliding-shear resistance.

The resistance related to the sliding shear mechanism of the column (V_{resS}) is evaluated without considering the stirrups effect, which may be assumed negligible for existing RC buildings built before seventies. The shear strength per unit area is taken equal to the maximum between v_{rd1} and v_{rd2}, as recommended by the NTC (2008):

$$v_{rd1} = 0.18 \cdot k \cdot \frac{\left(100 \cdot \rho_l \cdot f_{ck}\right)^{\frac{1}{3}}}{FC \cdot \gamma_c} + 0.15 \cdot \sigma_{cp} \, , \quad v_{rd2} = \left(0.035 \cdot k^{1.5} f_{ck}{}^{0.5} + 0.15 \cdot \sigma_{cp}\right), \tag{4.2}$$

where k is the lesser between $1+(200/d)^{0.5}$ and 2; d is the effective depth of the concrete section in mm; ρ_l is the geometrical ratio of longitudinal reinforcement; FC is the confidence factor; f_{ck} is the characteristic compressive resistance of concrete; γ_c is the partial security factor of concrete; σ_{cp} is the average stress in the concrete section. This type of collapse is brittle and thus the relative behavior factor is equal to 1.5 (NTC 2008).

4.3. Shear resistance related to the torsional collapse of the beam that supports the floor.

In one way frames, the weakest direction is the transversal one. The resistant mechanism in this direction takes in consideration the bending resistance of the floors and the torsional resistance of the bearing beam that supports the floors. In general, the dissipative capacity of floors is greater than the beam and thus the attention is focused on the beam. In fact, the bearing beam is subjected to shear and torsion, thus it is necessary to verify the capacity relative to the combination of both actions (NTC 2008):

$$\frac{T_{Ed}}{T_{Rcd}}+\frac{V_{Ed}}{V_{Rcd}}\leq 1. \tag{4.3}$$

In the equation T_{Ed} is the design value of the torsional moment acting on the beam, T_{Rcd} is the torsional resisting moment related to the collapse of concrete, V_{Ed} is the shear action due to gravitational loads and V_{Rcd} is the resistance related to the mechanism of shear and compression of concrete. The resisting torsional moment of the bearing beam (T_{Ed}^{res}) is obtained through the Eqn. 4.3, after the evaluation of V_{Ed}, V_{Rcd} and T_{Rcd}. The shear action on the column (V_{resT}) is evaluated at the presence of that torsional moment on the beam, using the simplified models above described (Fig. 3.2). This value of shear action is equal to the resisting shear of the column related to the torsional collapse of the beam. This type of collapse is brittle and so the behavior factor is equal to 1.5 (NTC 2008).

5. BEAM-COLUMN JOINT COLLAPSE.

For the vertical elements with unconfined joint in one or both principal directions, the maximum shear that the element may support without the collapse of the node is evaluated. In the procedure FIRSTEP-RC this type of control is performed at the end of the process. In particular, the shear action on the node has to be lower than the resisting shear force of the node. The shear action on each node V_{nEd} is evaluated through the node equilibrium. In Fig. 5.1 the actions on the node are displayed: V_i is the shear force on the element at the ground level, and it is estimated on the basis of the shear capacity of the structure F_R, that is evaluated taking into consideration the possible activation of the collapse mechanisms described above ($V_i = F_R \cdot f_{ie}$). Moreover, F_A is the quote of the external shear force on the element, applied at the first floor (F_A=$C \cdot V_i$, with C constant (Eq. 3.1)), while F_B is the quote of the shear action on the element due to the horizontal forces applied at the upper levels (F_B=F_A-V_i). M^R and M^L are the bending moments on the beams at the right and at the left of the node, C^R, C^L and T^R, T^L form the couples (compression and tension forces) in the beams, related to the presence of M^R and M^L. M_i and M_o are the flexural moments at the summit of the column at the ground level and at the bottom end of the column at the second level, respectively. The shear resistance of the node V_n is evaluated taking into account both the compression failure and the tension failure of concrete (NTC 2008). If the shear resistance of the node V_n is lower than that required (V_{nEd}), the demand has to be reduced by a factor equal to the ratio between V_n and V_{nEd}, and the shear resistance of the vertical element (V_{resL}) has to be reduced by the same factor. In this case the shear resistance of the column is F_{iux}=V_{resL}, so that the action on the node is within the limits of verification. The factor *Rid* is evaluated with the Eq. 5.1. If this factor is less than one, the brittle collapse mechanism of the node is decisive and the shear resistance of the structure has to be reduced for this factor. In this case the behavior factor is equal to 1.5 (brittle collapse – (NTC 2008)).

$$Rid = \min_i (Rid_i) = \min_i \left[F_{iux}/(F_{Rx} \cdot f_{iex})\right]. \tag{5.1}$$

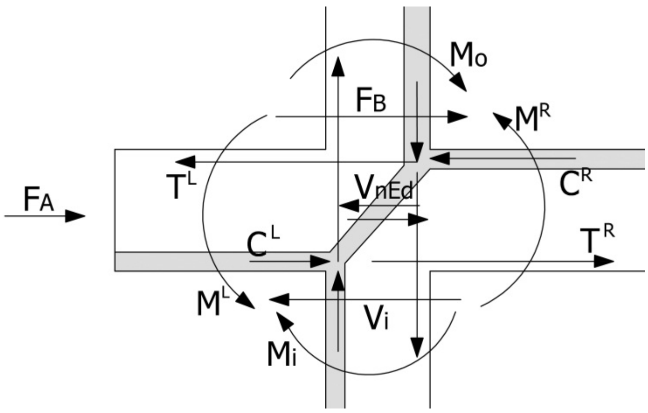
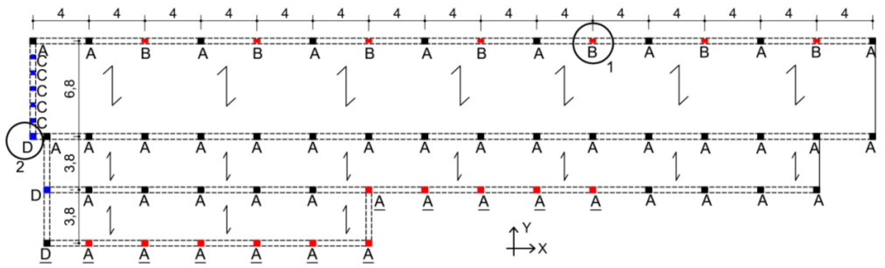


Figure 5.1: Equilibrium of an internal node.

6. WORKED EXAMPLE.

An example of the application of the procedure is shown in the follow (Franceschinis, 2012). The analysed building (1958) is a one way frame structure with beam foundation and deep bearing beams in the longitudinal direction. The building has 4 levels above the ground, the interstorey height is 3 m. It has a rectangular shape, of about 60 x 15m (floor plan in Fig. 6.1). The red elements in Fig. 6.1 are squat column in X direction, while blu elements are squat columns in Y direction (free height equal to 1 m). The strength of materials and the steel percentage were evaluated in function of the year of construction (concrete: f_{cm}=20 MPa, steel: f_{ym}=230 MPa, FC=1). The base shear resistance of the building was carried out through the FIRSTEP-RC procedure and through a linear static analysis using a finite element program. The comparison of results in Table 6.1 underlines the good accuracy of the simplified procedure with respect to a linear analysis; the most critical elements are the same for both procedures. The failure is due, in both directions, to sliding shear collapse of a squat column.



Section:	Dir X [m]	Dir Y [m]	Steel [%]
A, A, D, D	0.40	0.40	0.78
B	0.40	0.20	0.77
C	0.25	0.25	0.98

Figure 6.1: Plan of the ground level of the school building considered in the example.

Table 6.1: Peak ground resisting acceleration.

		FIRSPEP-RC	Linear-Analysis	Collapse
PGA a_u	X direction	0.045*g*	0.047g	Sliding shear q=1.5 (El.1)
	Y direction	0.015*g*	0.016g	Sliding shear q=1.5 (El.2)

6. CONCLUSIONS.

A new simplified mechanical procedure for the seismic assessment of reinforced concrete buildings in terms of peak ground acceleration was presented (FIRSTEP-RC). The few data required for the assessment may be gathered during a rapid survey of the building. A worked example puts in evidence the reliability of the methodology. The presented procedure was applied to a large number of buildings during the Assess Project, finalized to the prioritization of strengthening interventions on school buildings in Friuli Venezia Giulia Region in Italy.

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