SEISMIC RELIABILITY OF EXISTING RC BUILDINGS WITH SHORT COLUMNS

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ABSTRACT :

This paper deals with the problem of evaluating seismic reliability of existing RC frame structures including short elements. In the first part of the work a study of effectiveness of some models concerning evaluation of the shear capacity of RC short elements is developed, based on comparison with experimental data available in literature; this study has allowed to identify a model characterized by a good approximation in predicting the monotonic response of shear-critical elements. In the second part of the paper, the selected model has been used to evaluate the role of shear mechanisms in seismic response of sample frames representing structural configurations usual in the Italian building context. Performed analysis has shown that short elements induce a significant increase in vulnerability of the examined structures, particularly in the case of the sample frame characterized by a squat configuration, typical of industrial buildings.

KEYWORDS: RC structures, short elements, shear, theoretical models, dynamic analysis.

1. INTRODUCTION

It is well known that short elements, that is elements having a shear-span to section depth ratio (*a/d*) less than the conventional threshold of 2-2.5, represent a critical factor for RC frame structures due to their tendency to develop crisis mechanisms governed by shear. For this reason, modern codes for earthquake-resistant design (EC8, 2003) (OPCM 3274, 2003) advise against use of such element type, especially when the design procedure is based on a ductile behaviour. On the other hand, seismic vulnerability of existing buildings is an important issue. Concerning the Italian situation, most of existing RC buildings have been realized in the absence of any aseismic criterion; in addition, these buildings often incorporate short elements, particularly in the case of industrial constructions. This fact has resulted in renewed interest, within the scientific community, towards investigating behaviour of shear-critical elements. Several studies have been made on this subject in the past years. They mostly concerned development of theoretical models for the evaluation of shear strength under monotonic loads; lower efforts were devoted to response under cyclic load condition. Despite the large amount of experimental and numerical investigations available on this topic, the problem of modelling behaviour of short elements in cyclic shear is not completely resolved yet; this is due to the numerous variables affecting their inelastic response and, among them, the complex interaction between flexural and shear mechanisms.
The current work is aimed at evaluating seismic reliability of structural types including short columns, extensively used in Italy in the case of industrial buildings. It is divided into two parts, referred to different scales of observation. The first one is dedicated to validation, based on experimental data, of some of the most known procedures for evaluating shear strength of RC short elements. This study has allowed to identify a model characterized by a good approximation in predicting response of short elements under different mechanical and geometrical conditions. In the second part of the work, the same model has been used within a procedure aimed at evaluating seismic response of RC frame structures with short elements. In particular the problem of reduction in structural reliability of RC buildings due to shear-critical elements has been studied. Two sample structures having an opposite configuration and representing, respectively, a residential and an industrial building, have been assumed. Numerical analyses pointed out that the frame characterized by a squat configuration, typical of industrial buildings, resulted to be affected by shear mechanisms much more than the slender one ones.

2. THEORETICAL MODELS FOR SHEAR STRENGTH OF SHORT ELEMENTS

In this section some of the most important models to evaluate the ultimate shear of RC short elements are examined, namely the model by Shohara and Kato (1981), by Umehara and Jirsa (1984) and by König et al. (1993). These models have been tested through a comparison with experimental data available in literature.

2.1. Shohara and Kato model

The model proposed by Shohara and Kato assumes that the shear transmitting mechanism of a short column can be considered as the superposition of two different contributions: the beam mechanism and the diagonal strut mechanism (Figure 1). The element is assumed to be made by a concrete core absorbing part of shear through the compressed strut, and by two concrete external layers including longitudinal and shear reinforcement, according to the Mörsch theory. The model provides a shear-axial force domain consisting of three different regions: a central region, corresponding to the shear collapse, and two lateral regions, where the element experiences a flexural failure under axial load (tensile or compressive). The width of the central region depends on the amount of longitudinal and transversal reinforcment and on the *a/d* ratio.

2.2. Umehara and Jirsa model

The model by Umehara and Jirsa assumes that the shear strength of the short element can be obtained from the summation of three different contributions: the first one is due to concrete, the second one is due to transversal reinforcment and the third one is related to the axial load. The model has been set for values of the *a/d* ratio ranging between 1.0 and 2.5, and for medium-low values of axial load ($n = N/bdf_c \le 0.4$). In the following equation, the simplified expression assumed by CEB (1996) is reported:

$$Q_{UJ} = 1150\left(1 - 0.28\, a/d\right) A_c \sqrt{f_c} + [\min(0.2N\,;1400A)]/(a/d) \quad [kN] \tag{1}$$

where A and A_c are, respectively, the gross area and the area of the concrete core in m^2, f_c is the cylindrical concrete strength in *MPa* and N is the axial load in *kN*.

2.3. König, Graham and Tang model

This analytical model allows to obtain the monitonic shear-deformation curve for the element, which is assumed to be subjected to constant shear and anti-symmetric bending moment. The shear-deformation curve is found through an iterative procedure, based on the analysis of the flexural behaviour of the short element which is supposed to consist of a central uncracked region and two cracked triangular portions at the element ends (Figure 2).

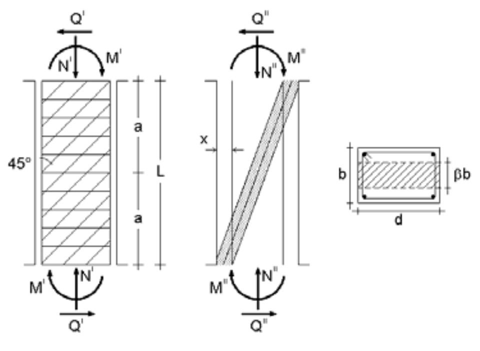


Figure 1. Shohara and Kato model: shear mechanisms.

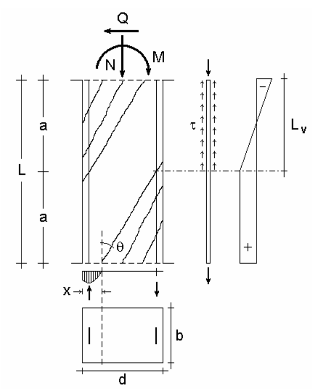
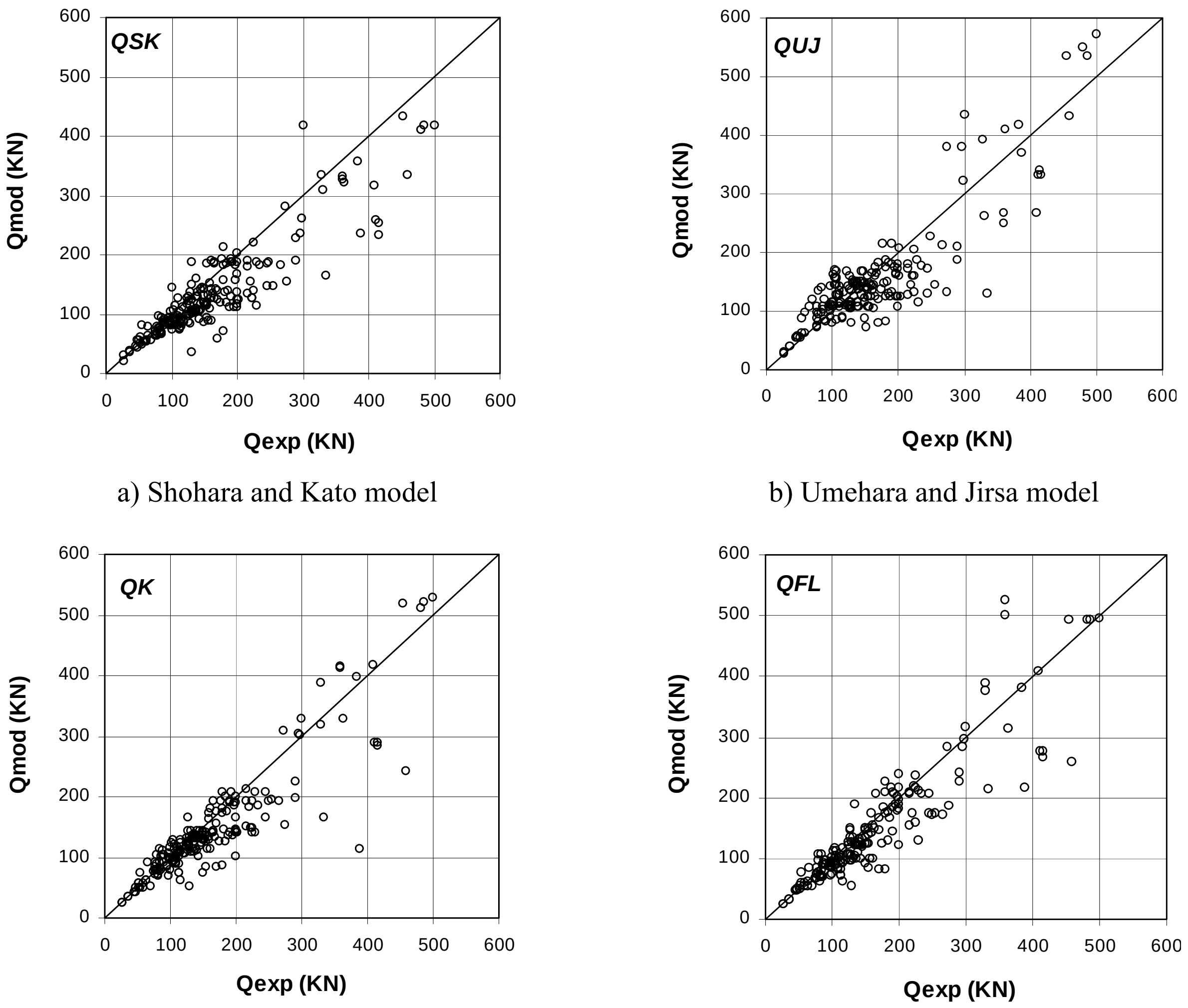


Figure 2. König et al. model.

The Kent and Park relationship (1971) is assumed for concrete, while the reinforcing steel is assumed to have an elastic-perfectly plastic behaviour, with a maximum tensile stress related to the bond properties. The transversal reinforcement is taken into account only for its confining effect on the concrete core. The collapse condition is assumed to be reached at the achievement of the yield tensile deformation of the rebars (ductile collapse) or at the achievement of the peak deformation of the compressed concrete (brittle collapse). The element response depends on geometrical properties, on the level of axial load, on amounts of transversal and longitudinal steel, on material strengths and on the bond relationship.

2.4. Comparison between theoretical and experimental results

Effectiveness of the above described models in estimating shear strength of short elements has been tested by means of a set of experimental data available in literature; these data refer to tests performed on 220 RC specimens having different geometrical and mechanical characteristics, with a predominance of short elements ($a/d \leq 2.5$). In particular, tests performed by the following institutions or authors have been considered: Building Research Institute of Japan (1978), Umehara and Jirsa (1982), Watanabe (1984), Tanaka et al. (1988), Tegos and Penelis (1988), Moretti and Tassios (1998), Papanikolau (1998), Liu et al. (2001), De Stefano and Nudo (2005). Results of comparison between analytical and experimental values are reported in the Figure 3, with reference to the assumed models.



a) Shohara and Kato model

b) Umehara and Jirsa model

c) König et al. model

d) Flexural model

Figure 3. Analytical (Q_{mod}) versus experimental (Q_{exp}) results.

In order to estimate contribution of flexural mechanisms in defining response of short elements, the ultimate shear of considered specimens has been also evaluated on the basis of a purely flexural crisis condition, corresponding to the achievement of the ultimate bending moment at the end sections of the element (it is assumed to be under constant shear and antisymmetric bending); respective results are shown in the Figure 3d.
Among examined models, the König model evidenced a better approximation with respect to experimental values; it was characterized by an average percentage difference (*apd*) equal to –8.6%. The flexural model gave also good results (*apd* = –9.2%), so testifying that contribution of flexural mechanisms is considerable, also for short elements; as expected, the accuracy of flexural model decreased with the *a/d* ratio. The Shohara and Kato model showed a worse approximation with respect to experimental data (*apd* = –15.4%), particularly for low values of the *a/d* ratio. The Umehara and Jirsa model, based on a simplified formulation, gave unreliable results owing to large approximations in evaluating shear capacity of short elements.

3. SHORT COLUMNS AS VULNERABILITY SOURCE IN INDUSTRIAL BUILDINGS

In Italy, most of the RC industrial constructions were built in the Sixties and the Seventies, that is before issuing the first italian seismic regulations (1974). Therefore a large percentage of these buildings was realized without any seismic protection; in addition, many of them have short columns. On 2000, the Seismic Department of the Regione Toscana started a research program aimed at evaluating seismic vulnerability of RC constructions belonging to productive areas (2004). Table 3.1 presents the statistic of industrial buildings investigated in some zones of Toscana subjected to significant seismic risk (maximum PGA of about 0.2 *g*; exceedance probability of 10% in 50 years).

Table 3.1. Industrial buildings in some Toscana areas subjected to seismic risk.

Zone	Total of RC industrial buildings			Industrial buildings with short elements			Percentage
	precast	cast in situ	total	precast	cast in situ	total	
Casentino	266	119	385	7	17	24	6.2%
Garfagnana	102	63	165	1	18	19	11.5%
Lunigiana	113	57	170	10	8	18	10.6%
Senese	245	162	407	23	50	73	17.9%
Valtiberina	182	122	304	15	27	42	13.8%
Total	908	523	1431	56	120	176	12.3%

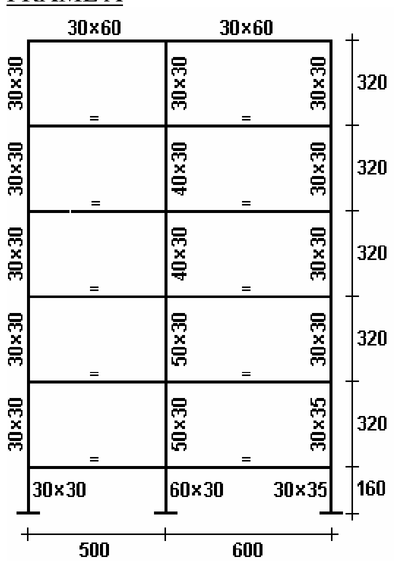
Figure 4. Typical industrial building with short elements.

As it can be seen, on a total of 1431 industrial buildings, about 12% of them presents short columns, so denoting the importance of the problem under a social and economical point of view. Such structural elements are mostly due to narrow windows positioned at the top of the external walls (Figure 4).

3.1. Seismic behaviour of RC structures with short elements

A non-linear dynamic analysis has been performed in order to evaluate the incidence of short elements in defining the inelastic response of different frame configurations including short columns. To this end, two sample frames representing typical structural configurations largely adopted in Italy in the pre-normative period, have been considered (Figure 5). The first sample structure (Frame A) is a six-storey, double-span frame representative of a residential building. Short columns are formed since the frame is supposde to be laid on a stiff RC wall in order to shape the basement windows. The frame has been proportioned not taking into account seismic actions and assuming, as regards materials, mechanical properties usual in the '70 years; namely, a mean compressive strength f_{cm} equal to 21 MPa for concrete and a steel class *Fe B 32k* (f_{yk} equal to 320 MPa) for reinforcement have been assumed. The second sample structure (Frame B) is a one-storey, 12-spans frame, extrapolated from an existing factory building monitored within the resarch program of the Regione Toscana. It is a typical example of industrial building which is characterized, in general, by a long rectangular plan and a consistent roof mass; short columns are positioned at the top of peripheral frames and come from two lines of beams containing narrow windows. In this case the actual material properties have been assumed, that is a mean strength of 16.6 MPa for concrete and a steel class *Fe B 32k* for reinforcement. The seismic response of the sample structures has been determined by performing a non-linear dynamic analysis with reference to a set of 20 ground motions (SAC project, 1997) whose mean spectrum matches with a good approximation the elastic spectrum provided by EC8 for the soil type C (Figure 6).

FRAME A



FRAME A
Beam load: 35 kN/m
Concrete strength: f_{cm}= 21 MPa
Steel strength: f_{yk}= 320 MPa
Beam reinf.: 6 (up) + 4 (down) *d*18 mm for L = 6.0 m; 5 (up) + 4 (down) *d*18 mm for L = 5.0 m ([1])
Column reinf.: cross-sections 50×30 cm and 60×30 cm: 3+3 *d*16 mm; other cross-sections: 2+2 *d*16 mm
Stirrups: *d*6 mm, *s* = 20 cm for beams and columns

FRAME B
Beam loads: 11 kN/m (roof beams); 4 kN/m (lower beams)
Concrete strenght: f_{cm}=16.6 MPa
Steel strength: f_{yk}= 320 MPa
Beam reinf.: cross-sections 30×50 cm: 3+3 *d*16 mm; cross-sections 45×60 cm: 4 (up) +3 (down) *d*16 mm ([1])
Column reinf.: 3+3 *d*16 mm for all the columns
Stirrups: *d*6 mm, *s* = 18 cm for beams and columns

([1]) Steel reinforcement at the end cross-sections

FRAME B

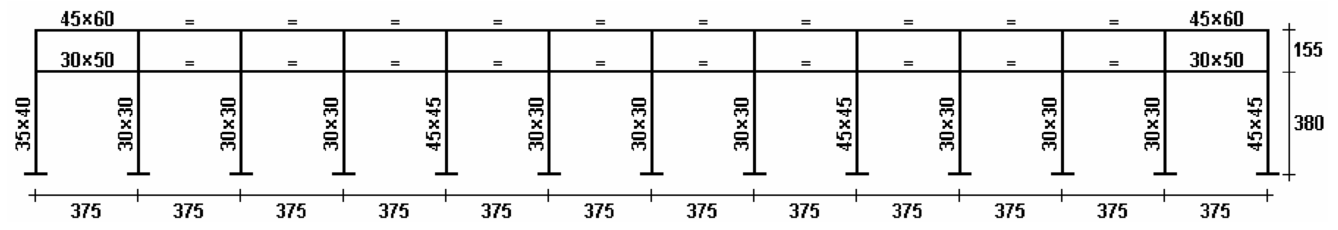


Figure 5. Sample structures and their properties.

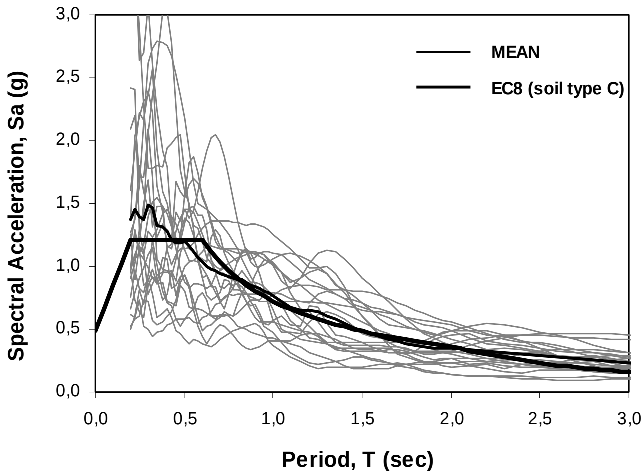
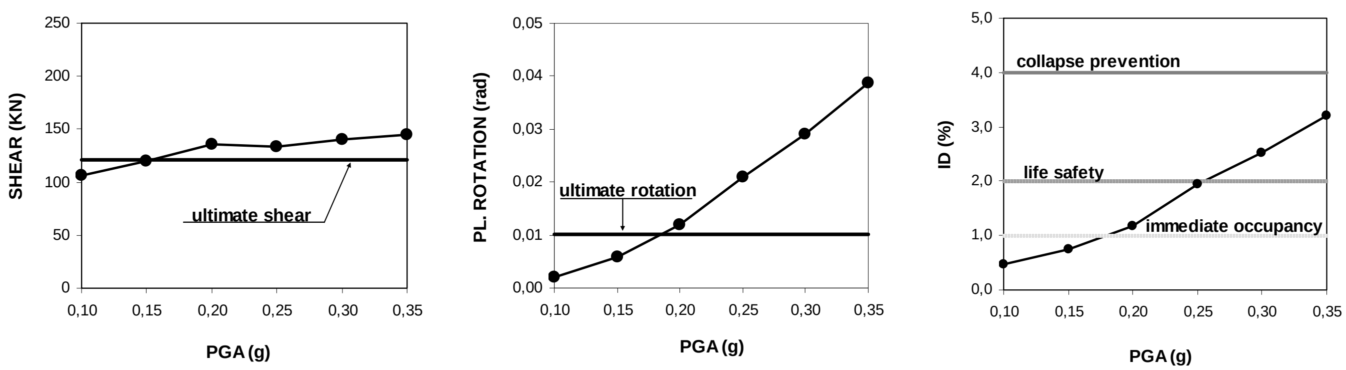


Figure 6. Elastic spectra of the ground motions assumed for the analysis.

In the analysis the assumed ground motions have been scaled in order to represent a seismic input having different intensities, varying between 0.10 *g* and 0.35 *g*. The following response parameters have been monitored: shear force (Q) within the short columns, plastic rotation (Φ_p) at the ends of beams/columns and interstorey drift (*ID*); they have been calculated, for the different earthquake intensities, as the average of the maximum values provided by each ground motion of the ensemble. The assumed control parameters are representative of different conventional crisis conditions: the ductile collapse, attained when the ultimate plastic rotation is reached within a cross-section of a beam/column ($\Phi_p = \Phi_{pu}$); the brittle collapse, attained when the shear strength, evaluated by the Köenig model, is reached within a beam/column ($Q = Q_u$); loss of service, attained when interstorey drift overcome conventional threshold corresponding to the "Immediate Occupancy" (*IO*) limit state provided by FEMA (2001). In this way the aptitude of the examined structures to develope a given collapse mechanisms can been evaluated.

3.1.1. Results of the dynamic analysis: frame A

Results of the dynamic analysis performed on the frame A (slender frame configuration) are illustrated in the Figure 7. In particular, the Figure 7a shows the maximum shear, averaged over the 20 assumed ground motions, as a function of seismic intensity; maximum values are achieved, as expected, within short columns at the base of the frame. It can be seen that the limit value of shear is overcome for a PGA of about 0.15 *g*. As regards plastic rotations (Figure 7b), it was found that limit values are achieved in short columns for a value of PGA between 0.15 *g* and 0.20 *g*. Concerning interstorey drift, results reported in the Fig. 7c show that the *IO* limit state is exceeded for a value of PGA included between 0.15 *g* and 0.20 *g*; the "Life Safety" limit state is achieved for a PGA of about 0.25 *g*. The first sample structure, therefore, has attained the different crisis conditions for similar values of PGA.



a) Maximum shear force within a short column

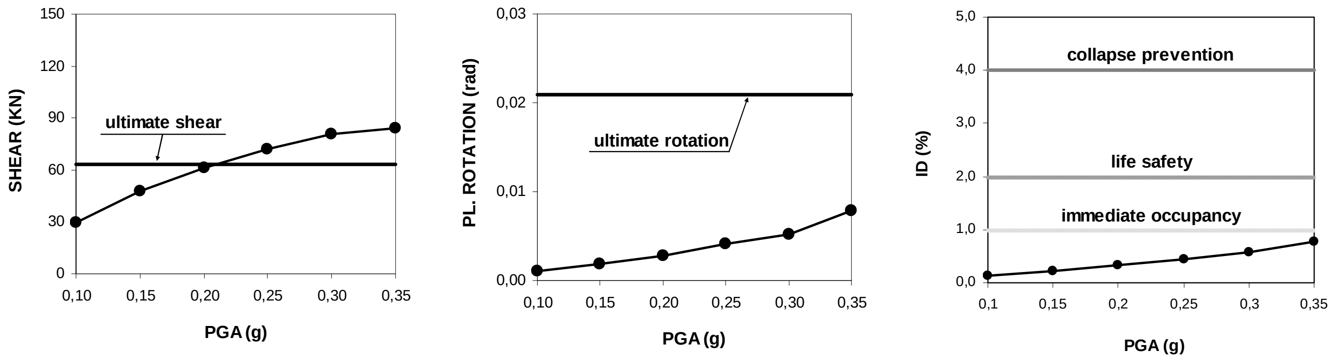
b) Maximum plastic rotation at the end of a short column

c) Maximun interstorey drifts

Figure 7. Frame A: results of non-linear dynamic analysis referred to the assumed crisis conditions.

3.1.2. Results of the dynamic analysis: frame B

Results of the dynamic analysis performed on the frame B (squat frame configuration) are illustrtated in the Figure 8. Highest values of shear force have been found within short columns having the smaller cross section (30×30 cm). From the diagram shown in the Figure 8a it can be observed that the ultimate shear is overcome for a value of PGA of about 0.20 *g*. Concerning plastic rotations, it resulted that the most involved elements are the slender columns at the first level of the structure; in all the cases the obtained values of plastic rotation are well below the respective ultimate value (Figure 8b). The Figure 8c shows that the maximum interstorey drifts obtained from the analysis are lower than the limit value corresponding to the *IO* limit state. In this structure, the presence of short column and the related influence of shear concentration, induced a considerable increase in the vulnerability levels. Therefore, obtained results indicate frame B as very sensitive to shear mechanisms. It follows that this kind of structure needs a specific attention and further studies.



a) Maximum shear force within a short column

b) Maximum plastic rotation at the end of a slender column

c) Maximun interstorey drifts

Figure 8. Frame B: results of non-linear dynamic analysis referred to the assumed crisis conditions.

4. CONCLUSIONS

In the first part of the paper some of the most important models providing the shear capacity of short elements have been tested on the basis of experimental results present in literature. Among the examined models, the one proposed by König et al. showed the best approximation at the varying of the main parameters affecting the inelastic response of short elements (*a/d* ratio, axial load level, amount of longitudinal and transversal reinforcement). The same model has been applied in the second step of the work, aimed at evaluating seismic reliability of RC frames including short elements. A non-linear inelastic analysis has been performed on sample structures representing typical constructions of the Italian context before issuing the first seismic regulations. The seismic behaviour of the examined structures has been checked by adopting, as response parameters, the plastic rotation at the element ends, the shear within short columns and the interstorey drift. From the performed analysis, it was found that the first sample structure, a two-span, tall frame, attained the different crisis conditions for similar values of the seismic intensity. The second sample structure, a multi-span, squat frame typical of industrial buildings, exhibited a very marked aptitude towards achievement of brittle collapse owing to excess of shear in short columns. This outcome points out an important vulnerability scenario due to the great incidence of this kind of structural configuration within the Italian building heritage, particularly as regards industrial constructions.

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