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INFLUENCE OF CONNECTIONS MODELING ON THE SEISMIC BEHAVIOUR OF INDUSTRIAL BUILDINGS

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Abstract. *Many industrial buildings with a precast concrete structure have been designed in accordance with non-seismic provisions by considering only gravity loads or, in some other cases, in accordance with not adequate seismic detailing requirements .In particular, seismic events that occurred in the recent past have shown that the connections between the main structural elements play an important role on the dynamic and seismic behaviour of the precast concrete structure. Nowadays, the more advanced technical standards require mechanical connections between the precast elements; however codes do not furnish exhaustive information about the design and modelling of these connections neither in the seismic vulnerability assessment of existing buildings nor in the design of new constructions. In this work, the role of dowel connections between beams and columns elements in the seismic behaviour of one-storey precast concrete industrial building is investigated and a simple modelling procedure is proposed for both the seismic vulnerability assessment of existing buildings and the design of new constructions. The applicability of the proposed methodology is tested through linear dynamic analysis and non linear static analysis carried out on two real case studies. In particular, different finite elements models are developed for the benchmark cases, both considering or omitting the rigid floor constrain and both assuming rigid or flexible connections. The results show that the modelling strategy strongly influences the seismic response and thus the safety verifications of the structural typologies considered.*

1 INTRODUCTION

Since the second post-war period single-storey precast structures represent a structural typology widely used for the construction of industrial and commercial buildings. Nowadays, in the Italian and European context, a large part of these buildings are characterized by a outdated and inadequate structure compared to structures which meet current seismic criteria, resulting therefore exposed to high seismic hazard.

There are many evidences about the behaviour of precast structures during past earthquakes; the main causes associated to the damage of the precast structures were constituted by the failure of connections. In particular, the recent earthquake that hit Emilia in 2012 highlighted the importance of the role that the connecting systems play in the global behaviour of prefabricated structures [1]. Specifically, in the older precast buildings, collapses due to mere loss of seating among the elements were observed where the connections were entrusted only to the resources of friction.

Recent Italian regulations, in force until a few years ago, allowed the realization of friction based connections between structural elements [2] for areas classified non-seismic while the latest standards ([3], [4]) require that the connection is secured by mechanical devices properly dimensioned. However, serious damages due to inadequacy of the connections were also observed in more recent buildings, complying with the latest standard indications. In some cases, the cause can be traced to reduced resistance of the same, to the absence of the rigid diaphragm behaviour or abnormal rigid floor behaviour induced by particular configurations of connection between the structural elements (main and secondary) that generated forces concentrations in some connections determining the rupture. Therefore, these issues have highlighted a deficiency of the current rules that, although requiring the use of mechanical connections, do not furnish exhaustive information for designing [1] and modelling of the same.

Connections by means of pin or dowel are the most common connection system in the single-storey industrial buildings in Europe for their effectiveness and practicality of realization. The aim of this paper is to propose a procedure for modelling the dowel connections that is easy to use both for the evaluation of the seismic vulnerability of existing buildings and for the design of new buildings. Modelling aspects of dowel connections between beams and columns and between main beams and secondary elements are investigated with reference to single storey buildings with precast structure frame (beams, columns, and roof secondary elements). For these purposes, in a first step, the analysis of the state of the art is carried out, through investigation of the scientific literature and available codes, then the proposed method for modelling the dowel connections is applied to two real case studies consisting of buildings with single level precast structure with dowel connections. The structures are modelled by means finite element method both by considering and by omitting the rigid floor constraint. The procedure is tested through nonlinear static analysis and the main results are discussed. In particular, it is emphasized how the strategy for connection modelling affects the seismic response of the structure and, consequently, the results of the safety verifications.

2 BACKGROUND

The dowel or pinned connections sustain shear forces through a mechanism known in the literature as dowel action (Figure 1), discovered in the 30s [5].

The resistance of the connection can be governed by three different failure mechanisms: pure shear failure of the pin (steel shear failure), concrete splitting failure and yield of the bar and concrete crushing of the contemporary concrete around the dowel (combined steel and concrete failure).

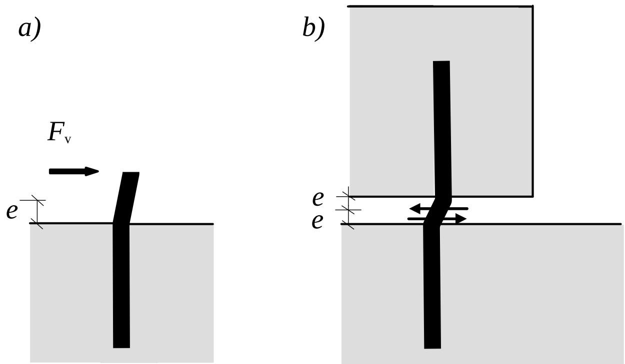


Figure 1: Transfer of shear force through the dowel action: *a*) one-sided dowel; *b*) double-sided dowel [7]

The shear failure is not very applicant; it can be determined by means the Von Mises criterion.

The concrete splitting failure is a brittle mechanism that can arise if the bar acts against the concrete cover characterized by a reduced thickness c and not provided with adequate confinement bars. Various authors conducted experimental investigations with the aim of estimating the splitting resistance. Vintzeleou and Tassios (1986) [6] provide expressions derived from analytical models and show that the failure may happen for side splitting or bottom splitting in relation to the diameter of the pin and to the ratio between the concrete cover c and the lateral cover b_{ct}, measured along the orthogonal direction of the acting force. The splitting failure can be avoided if adequate bars are foreseen and dimensioned by means strut and ties models, according to the fib bulletin n.43 document [7].

The combined steel and concrete failure mechanism is a ductile mechanism because it occurs together with the formation of plastic hinges in the pin. This mechanism was widely investigated by the scientific community since the beginning of the last century. In the first half of the previous century simple models based on linear elastic behaviour of materials were used [8]; however the linear model proved unsuitable for such purposes since the materials exhibit a plastic behaviour in conditions close to collapse. For these reasons, the beginning of the second half of the last century models based on the theory of plasticity were adopted. Rasmussen (1963) [9] was among the firsts to propose a plastic model; other authors investigated the same failure mechanism by calibrating the analytical model on the basis of experimental results, including [10]. Subsequently, Vintzeleou and Tassios [6], [11], applied the Brom's model [12] of free-headed pile in cohesive soil for the definition of the pin resistance and the response curve of the same. On the basis of the theory of plasticity, Engström (1990) [13] proposed further formulations for the determination of the parameters of dowel displacement. An exhaustive summary of the studies conducted on the connection strength governed by the pin yielding is available in fib bulletin n.43 document [7].

3 PROPOSED METHODOLOGY

3.1 Ultimate strength

The ultimate strength is assumed equal to the minor among the resistance values associated to rupture mechanisms discussed in the previous paragraph.

As previously reported, pure shear resistance of the pin is determined via Von Mises criterion whereas concrete splitting failure follows indications proposed by Vintzeleou and Tassios in [6]. This collapse mechanism can occur if the cover c of the pin is less than 6 to 8 times its diameter d. In particular, the resistance is determined as the smaller value between the side splitting resistance and the bottom splitting resistance, reported in this order in the Equation (1) where f_{ct} indicates the tensile strength of the concrete.

$$F_{V,cr} = \min\left(2b_{ct}df_{ct}; 5f_{ct}d\frac{c^2}{0.66+d} \right) \tag{1}$$

In the case of covers smaller than limits reported above and equipped with confinement bars, splitting failure can not be considered. Therefore, by considering that the precast elements are provided of bars in the connection areas, the connection rupture can be assumed as the minor value between the pin shear strength and the dowel yielding resistance. Resistance associated to the last collapse mechanism is determined through the following Equation, according to the requirements set out in [7],

$$F_{V,Rd} = \alpha_0\alpha_e d^2\sqrt{f_{cc}f_y} \tag{2}$$

where f_{cc} and f_y represent the materials strengths, while α_0 and α_e are the parameters evaluated respectively according to the Equations (3) and (4)

$$\alpha_0 = \sqrt{\frac{\beta_c}{3}}, \qquad \text{where} \quad \beta_c = \frac{f_{cc}^*}{f_{cc}} = 4 \tag{3}$$

$$\alpha_e = \sqrt{1+\left(3\frac{e}{d}\sqrt{\frac{f_{cc}}{f_y}}\alpha_0\right)^2} - 3\frac{e}{d}\sqrt{\frac{f_{cc}}{f_y}}\alpha_0 \tag{4}$$

in which f_{cc}^* indicates the concrete strength in the triaxial stress state.

3.2 Shear-displacement law

The force-displacement law is proposed on the basis of the information given in [7] and of the results of research performed in the literature. Such relationship is determined with reference to the ductile failure mechanism. In particular, the ultimate strength is evaluated from Equation (2) by assuming, in the case of double – sided dowel configuration, a concrete strength f_{cc} equal to the maximum value between the resistances of the two connected elements.

The yielding point is considered reached to a level of strength equal to half of the ultimate strength, while the displacement at yield is evaluated on the basis of the following formula

$$s_y = \frac{2F_{VRd}\beta_e\left(e\beta_e+1\right)}{E_c} \tag{5}$$

in which β_e is the stiffness parameter defined as

$$\beta_e = \left(\frac{E_c}{8E_sI_s}\right)^{0.25} \tag{6}$$

where E_c and E_s are the Young's moduli of the concrete and steel, respectively, while I_s is the moment of inertia of the dowel cross-section. Displacement corresponding to the ultimate plasticization of the pin $s_{\max}$ is evaluated by means the following expression:

$$s_{\max} = \theta_{cr}l_p \tag{7}$$

where θ_{cr} represents the critical rotation, which is assumed proportional to the curvature of the pin cross-section when this latter reaches the yield strength. This rotation is estimated as $1.75\varepsilon_{sy}/d$. The l_p parameter corresponds to the distance of the plastic hinge from the external edge x_0 in the case of one sided dowel configuration whereas, in the case of double sided dowel configuration, l_p represents the distance between the two plastic hinges. The distance from the outer edge of the element to the plastic hinge x_0 is determined via the following expression.

$$x_0 = \frac{F_{VRd}}{3\alpha_0^2 f_{cc} d} \tag{8}$$

Figure 2 illustrates an example of the force-displacement curve proposed.

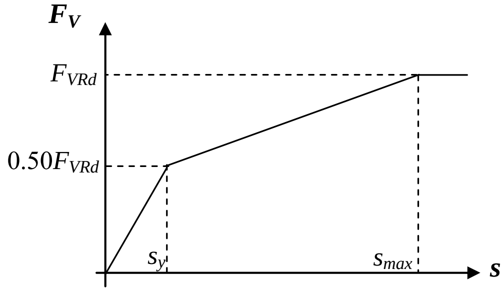


Figure 2: Example of the force-displacement curve proposed.

4 CASE STUDIES

In this section, the proposed procedure is illustrated by considering two real existing buildings that belong to the same industrial complex, but built in different years in Italy. The seismic action for the structures site is defined by the NTC08 soil type E (soil factor S = 1.524) spectrum [4], for a peak ground acceleration PGA = 0.174 g and period of return T_R = 475 years. The Figure 3 shows the seismic action for the selected case studies.

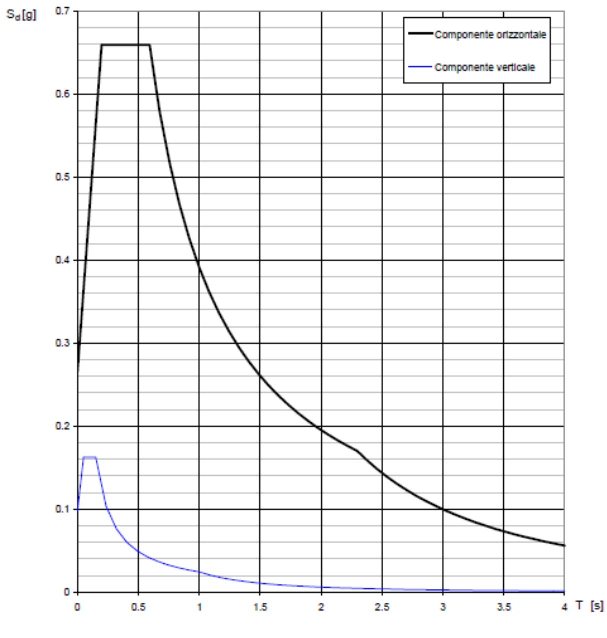
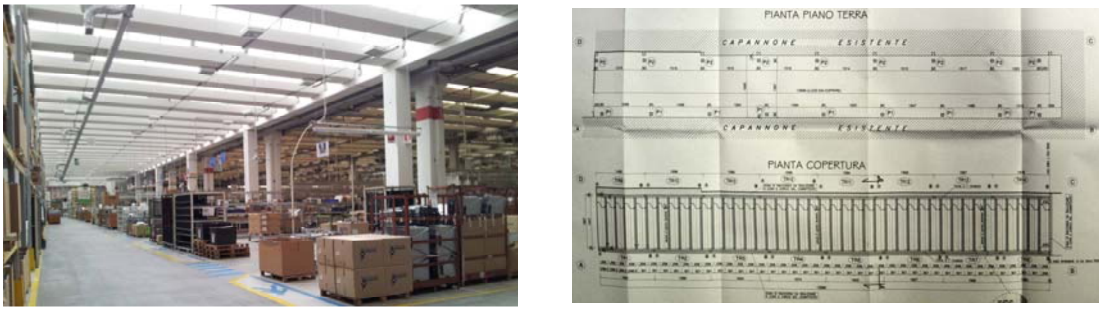


Figure 3: Vertical and horizontal elastic pseudo-acceleration response spectrum of the buildings site

4.1 Case study 1

The first case study concerns a structure with an elongated rectangular plan built in 2005; it consists of equal columns with square cross-section of 0.80 x 0.80m and main beams with an "inverted T" cross-section with a height of 1.65 m. These beams support secondary beams that complete the roof. These latter have the typical shed cross-section, largely diffused in Italy. Beam-column and beam to shed connections are dowel connections, made with Italian FeB44k steel. Figure 4 shows a photo of the building and an excerpt of the original project while Figure 5 shows the details of the connections.

First analyses are carried out by adopting dynamic linear models with rigid connections. Since the real degree of stiffness of the roof floor is not known, two global models of the structure are defined, with and without the rigid diaphragm constraint for the roof. Additionally, the analyses are conducted by adopting a confidence factor equal to 1.0 (due to the nature of the prefabricated structure) and 1.35 (given the lack of knowledge of the structure details). Structure factors of 1.5 and 3, as contemplated by the Italian current regulations for existing buildings [4], are chosen to take into account the dissipation capacity of the structure.



a) *b)*

Figure 4: Case study 1: a) photo taken from the inside; b) excerpt of the originate projects

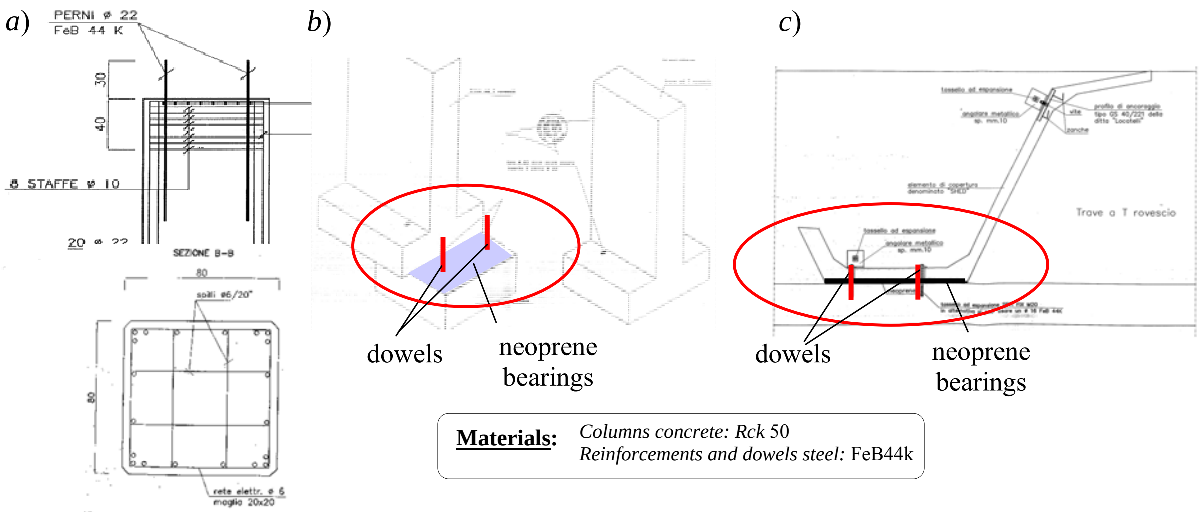


Figure 5: a) Detail of dowels; b) beam-column connections; c) beam-shed connections

It should be observed that the linear analyses are characterized by some limits. First off all the evaluation of the seismic vulnerability of the investigated structure carried out via linear analysis is excessively precautionary, since it is necessary to refer to the first element in which capacity is exceeded, thus in some cases the seismic action sustained by the building is very low. Additionally, it should be noted that the analyses are conducted by assuming over-resistant and rigid connections, i.e. not-deformable and able to allow the columns plasticization, and thus the adoption of the behaviour factors.

In order to investigate the behaviour of the structure beyond the elastic limit non-linear analysis are performed. Also for these analyses both the case of deformable deck and infinitely rigid roof are considered. Figure 6 shows the comparison between the elastic and inelastic spectra relative to a period of return T_R = 475 years and the capacity curve of the structure obtained by applying a distribution of forces proportional to the masses along the direction +x and +y (the results obtained by applying the same distribution along the negative direction and by assuming a rigid deck are similar and for reasons of synthesis are not reported). It is possible to observe that in both analyses the structure ductility (μ_c) is high (about 3) and greater than the value required to sustain the design earthquake (R_μ). Therefore, it can be concluded that safety verifications are satisfied because, in each case, the ductility demand is less than the available ductility. Also in this case, it should be emphasized that the results obtained are valid only in the event that the connections between the structural elements are over-resistant with respect to the columns. So, it is necessary to check the full strength of the dowels, by comparing the demand, expressed in terms of internal forces, with the resistance provided by the pins at each connection.

From non-linear static analysis a maximum shear force of 234 kN in beam-to-column connections and a maximum action of 232 kN in the beam-to-shed connections are obtained. Collapses for splitting does not take place because the cover thickness is greater than the limit of 6-8 d as shown by Figure 7. Therefore, the possible local collapse mechanisms can be induced by shear failure or flexural yielding of the dowel. The pure shear strength of the couple of dowels in the beam-column connection amounts to 203.8 kN. The flexural resistance of the connection, obtained by substituting in the Equation (2) the value of the materials resistance and by assuming α_0 = 1 and α_e = 0, is equal to 144.6 kN. From the results it can be noted that the resistance of the connection is governed by the flexural failure mechanism of the dowel and that the resistance of the connections is lower than the acting shear forces, thus they are not over resistant.

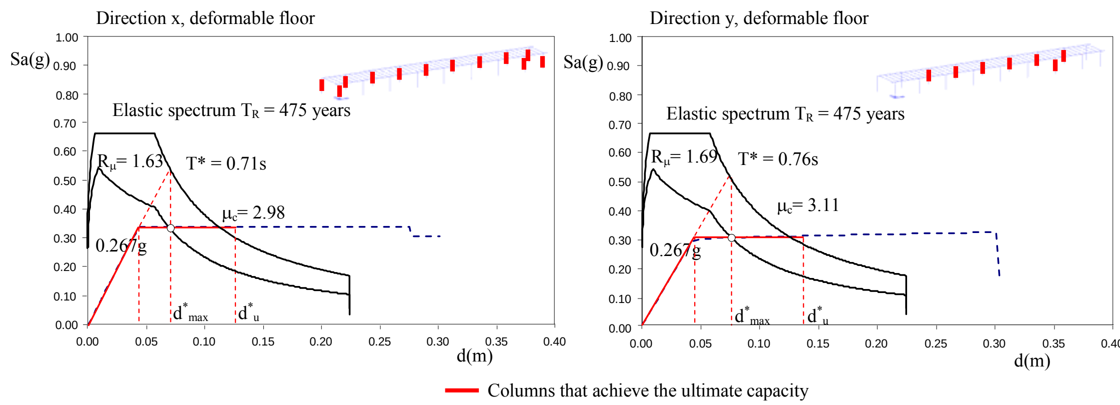


Figure 6: Curve capacity relating to the analysis of push-over (with forces proportional to the masses) both in the longitudinal direction x and transverse direction y

However, it should be noted that the deformability of the connections could result in a reduction of the forces acting on the same. Therefore, a proper assessment of the local demand on the connections and the comparison with their capacity can only be performed in terms of displacements in a model with deformable connections, as shown below.

Figure 7 shows the force-displacement law adopted to describe the behaviour of the connections in the non-linear model of the industrial building, without rigid diaphragm constraint.

Non-linear static analyses carried out both in the transverse (y) and longitudinal (x) direction have provided deformation levels much lower than those for which the dowels plasticization occurs, for both main beams-secondary beams and beam-columns connections.

In Figure 8 the comparison between the capacity curve and the demand curve is depicted. It is possible to note that, in both the directions, the earthquake sustained by the structure is higher than design one (period of return T_R greater than 475 years) and the risk index is greater than 1.

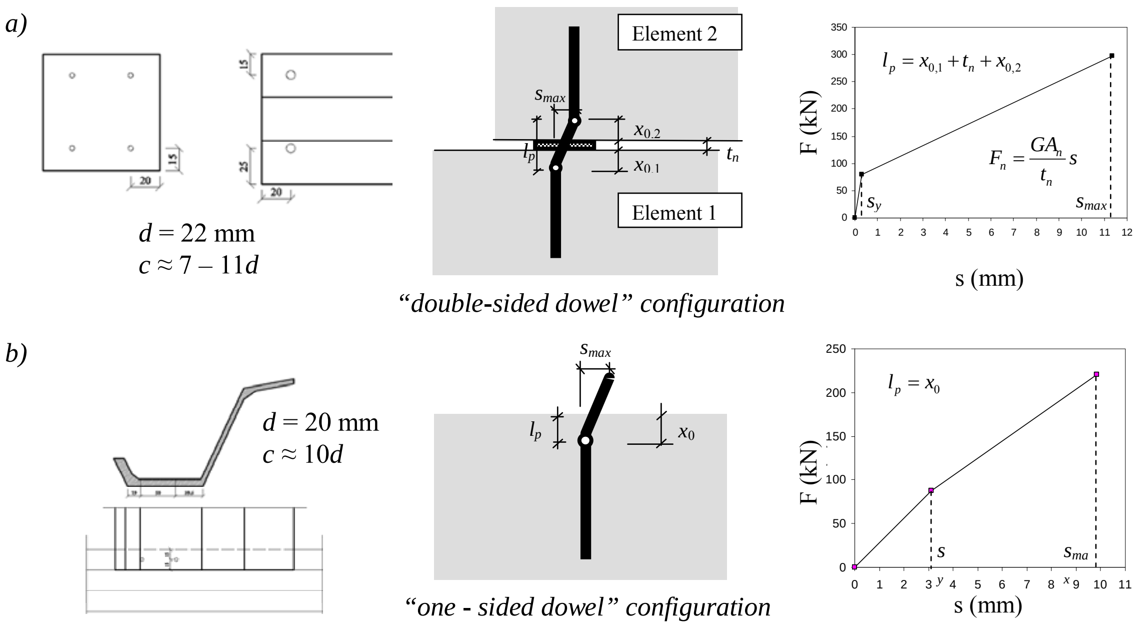


Figure 7: Shear-displacement relations of a) beam-column connections and b) beam to shed connections

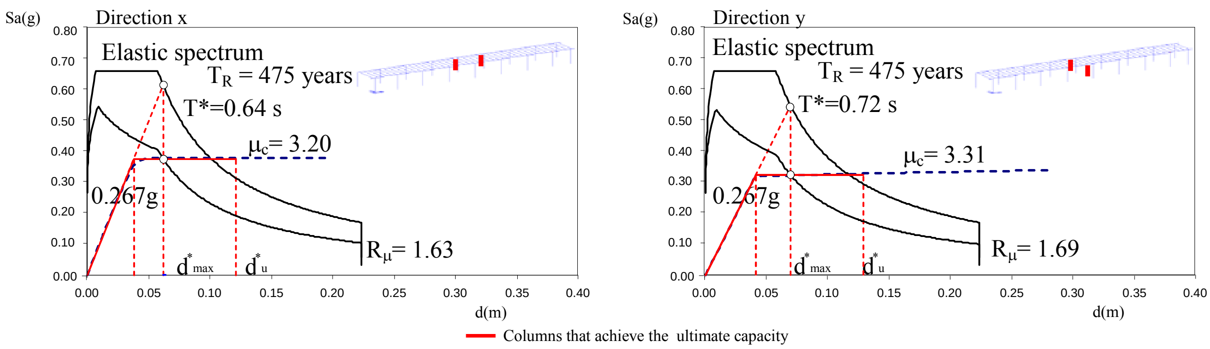


Figure 8: Push-over analysis results: comparison between demand and capacity curves in the ADRS plane for both the directions of analysis

4.2 Case study 2

The second case study concerns an irregular building in plan and elevation, built in 1981 and adjacent to the building previously analyzed. The building is characterized by precast columns of variable dimensions, from 0.90 x 0.90m to 0.30 x 0.40m, while the beams have "inverted T" and "Omega" cross-section. As regards the materials, the resistances are lower than those of the first case study. The beam-column connections and beams-shed connections consist of steel dowels, whose configurations are shown in Figure 9.

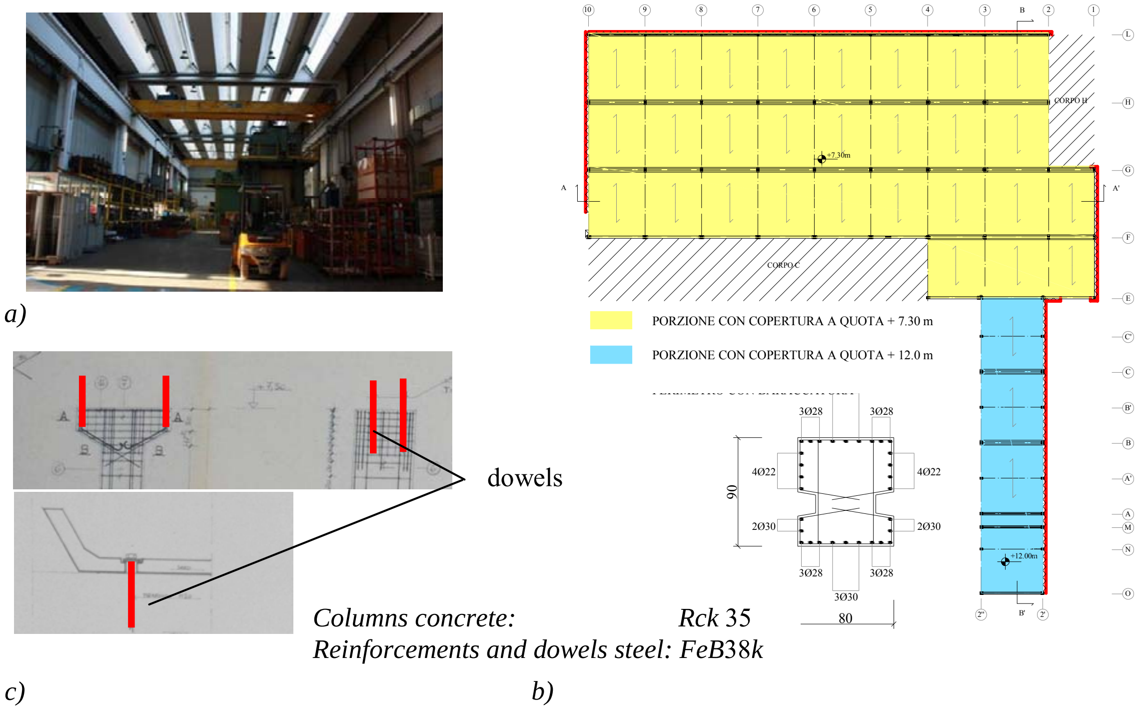


Figure 9: Case study 2: a) photo taken from inside; b) write-off of project; c) write-off of the executive details of beam-to-column and beam-to-shed connections

Also in this case dynamic linear analyses are carried out by adopting models with rigid connections and both by considering and by disregarding the rigid plane behaviour of the roof. Same confidence factors and behaviour factors employed in the previous case study are used for performing the safety verifications. In many cases the checks are not satisfied. Also in this case, since the linear analyses are considerably prudential non-linear static analyses are performed in order to take into account the behaviour of the structure beyond the elastic range. From the comparison between the elastic and inelastic demand spectrum and the structure capacity curve it is possible to observe that the verifications are satisfied in that, in each case, the demand in terms of ductility (R_{μ}) is less than the available ductility (μ_c). With the objective of checking whether the connections over strength hypothesis is satisfied, dowels forces are compared with the strengths. The analysis carried out in both directions have shown that the shear forces on the various types of beam-column connections were greater than the dowels resistances, calculated by assuming the same assumptions of the previous case study. Moreover, it was observed that the dowel resistance is governed by the combined steel and concrete failure. Splitting verification was not conducted because, despite the covers result smaller the limit of 6-8 *d*, they are equipped with confinement bars. Since, as explained in the previous case study, the analysis conducted by considering rigid connections lead to an overestimation of the forces, the same analyses are carried out by modelling the connections as deformable. The assumed force-displacement law is depicted in Figure 10.

Non-linear static analyses carried out both in the transverse (y) and longitudinal (x) direction have provided for both beams-shed connections and beams-columns displacement levels much higher, in some cases, than the displacement at which the ultimate plasticization occurs. However, the obtained displacements are always lower than the seating extension of the supported elements. Therefore, it is possible to state that important damages occur in the connections, but seating losses of the main and secondary beams do not occur.

In Figure 11 the comparison between the capacity curve and the demand curve is depicted; from the comparison it is possible to note that, in all cases, the earthquake sustained by the structure is higher than design one (with period of return T_R = 475 years) and the risk index is greater than 1.

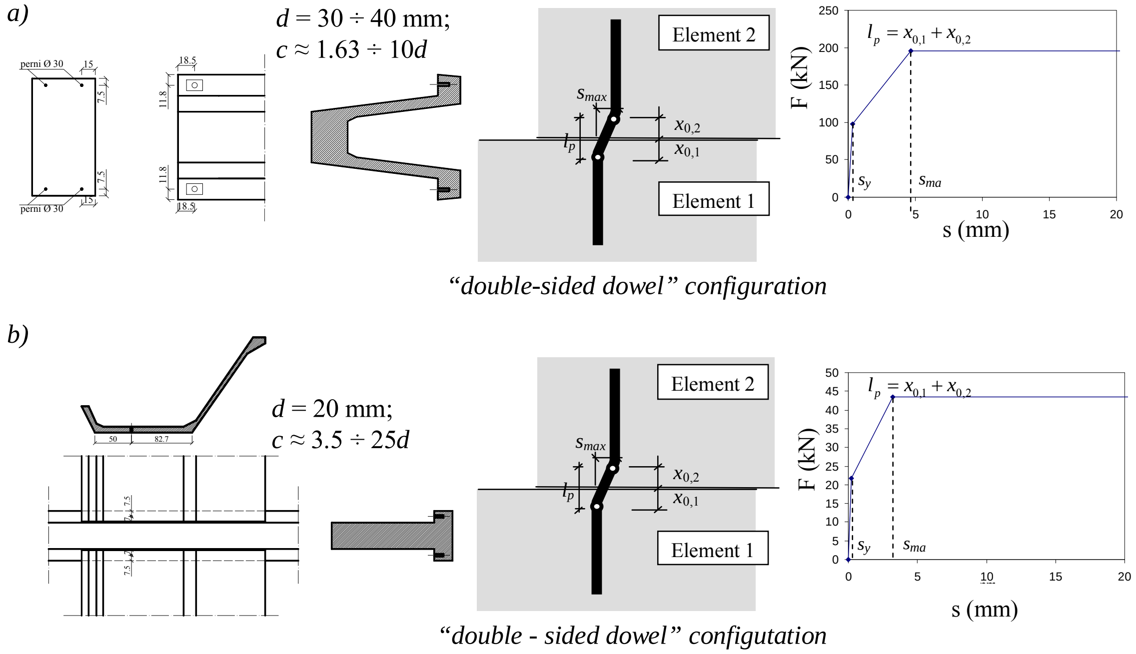


Figure 10: Force–displacement relations of a) beam-to-column connections and b) beam–to-shed connections

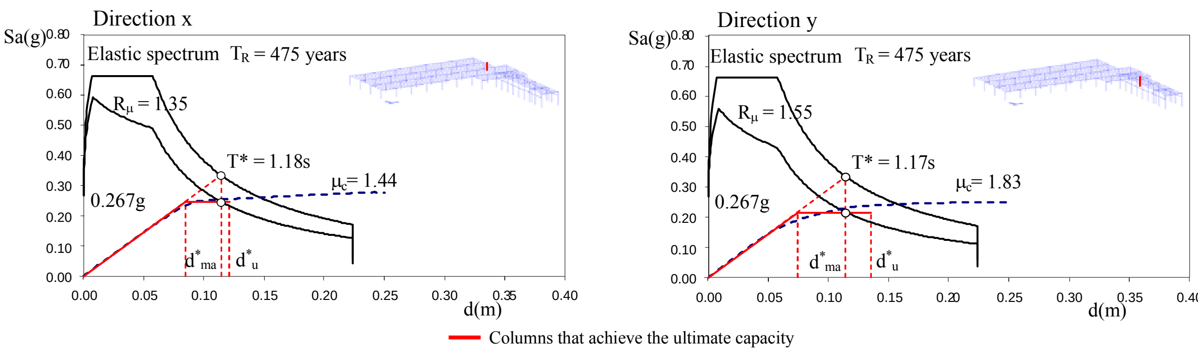


Figure 11: Push-over analysis results: comparison between demand and capacity curves in the ADRS plane for both the directions of analysis

5 CONCLUSIONS

In the present work a procedure for modelling the dowel connections between prefabricated elements of single level buildings was proposed for assessing the seismic vulnerability of existing buildings and for the new buildings design. The proposed shear force-displacement law refers to the ultimate flexural dowel resistance and its applicability is tested by performing non-linear static analysis conducted on two real industrial buildings.

The analyses are carried out both by considering the rigid and the deformable behaviour of the connections and both by omitting and by considering the rigid diaphragm constraint. The results have shown that the seismic response of the structure and thus the safety verifications are strongly influenced by the modelling strategy adopted for describe the connections. In particular, in both the study cases, it is observed that the shear forces on the beam-to-column connections and shed-beams connections reach significantly higher values in the models with rigid connections with respect to the models with deformable connections. Moreover, for both the study cases, the investigated structures are able to sustain the design earthquake only if elastic and inelastic connection behaviour is modelled, because the connections are adequately resistant and allow the formation of the dissipative mechanism in the columns.

Finally, it was noted that in the first case study, of more recent realization, the structure is able to withstand the design action without to undergo significant damage of the connections, while in the second case the dowels of many connections are plasticized undergoing elevated plastic displacements, but minor of the available seating length.

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