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ESTIMATION OF EFFECTIVE STIFFNESS OF INFILL PANELS AT DAMAGE LIMITATION LIMIT STATE FOR LOW- AND MID-RISE INFILLED RC FRAMES

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Abstract. *A growing attention has been addressed to the influence of infills on the seismic behavior of Reinforced Concrete buildings, also supported by numerical and experimental analyses, and by the observation of damage to infilled RC buildings after recent earthquakes (e.g. L'Aquila 2009, Lorca 2011, Emilia 2012). In this paper, the attention is focused on the Damage Limitation Limit State (DL LS), defined as dependent on displacement capacity limit directly related to the damage to infill panels. Although inelastic (post-cracking) displacement demand in RC members at this LS can be quite low, infill behavior can be already strongly non linear, if a diffuse state of cracking has begun to arise in infill panels. Thus, based on the results of several non linear dynamic analyses, a reduction factor of the initial elastic stiffness of infills is estimated, in order to obtain an effective stiffness of infill panels to be used in a linear analysis in order to obtain a displacement demand at DL LS (i.e., maximum Interstorey Drift Ratio) which is approximately equal to the demand evaluated through non linear dynamic analysis on the same model. To this end, Incremental Dynamic Analyses (IDAs) are performed on four- and eight-storey infilled frames, designed for seismic loads according to the current Italian technical code and for gravity loads only according to an obsolete technical code. Influence of number of storeys and design typology on the obtained effective stiffness is also analyzed.*

1 INTRODUCTION

The observation of damage to infilled RC buildings after recent earthquakes (e.g. L'Aquila 2009, Emilia 2012) has demonstrated that the presence of infills in Reinforced Concrete (RC) buildings cannot be disregarded. Infills are usually assumed as partition elements without any structural function, but they have a significant influence on the increase in lateral stiffness and consequently on the reduction in period of vibration, on base shear capacity, on possible brittle failure mechanisms in joints and columns due to local interaction between panels and the adjacent structural elements, and on the building collapse mechanism.

The issue was investigated during last years supported by numerical ([1]-[9]) and experimental analyses ([11]-[19]), regarding both the influence of infills on the global seismic performance of infilled structures and "local" effects due to the interaction between infill panels and surrounding RC frames. The effect of mechanical properties of infills and RC influencing the seismic capacity of infilled RC structures has been investigated also through sensitivity analyses ([6], [8], [10]). Such analyses have shown that the rotational capacity of RC columns and concrete compressive strength have the highest influence on the seismic capacity at high level of displacement demand; whereas, as far as seismic capacity at low level of displacement demand is concerned, e.g. at Damage Limitation (DL) Limit State (LS), mechanical characteristics of infill panels have the highest influence on the response of the uniformly infilled frames ([8], [10]).

Thus, the contribution of infills to the lateral seismic response in terms of strength and stiffness significantly changes with the displacement demand: displacement and drift demand significantly decrease if infills are explicitly taken into account in the numerical model with respect to the corresponding bare frame, but such a reduction depends on strength and stiffness properties of infills and it is more significant at DL LS than at Ultimate LS (when infills are already extensively damaged). At DL LS the presence of infill panels produces a reduction in displacement capacity compared with a bare structure, due to the assumption of displacement capacity limits accounting for the damage affecting these elements; if such detrimental reduction is not counterbalanced by the beneficial increase in stiffness and strength provided by infill panels, a reduction in seismic capacity can be observed [9].

In this paper, a procedure is proposed for the estimation of an effective stiffness of infill panels to be used in linear analysis for seismic assessment at DL LS, thus allowing to explicitly include these elements also within this kind of analysis approach, which nowadays is still a widespread method, especially for seismic design of new structures. To this end, results from non linear Incremental Dynamic Analyses (IDAs) are used as a reference, and a principle of equivalence between linear and non linear analyses in terms of Interstorey Drift Ratio (IDR) demand is applied – being the demand parameter used for seismic assessment at this performance level. The influence of the number of storeys and of the design typology is investigated, too.

2 STATE OF ART AND CODE REVIEW

According to Eurocode 8 [20], the design and the construction of a structure should guarantee that, under DL LS seismic action, the costs due to the damage and the corresponding limitation of use of the structure should not be disproportionately high with respect to the cost of the whole structure. Provisions for seismic assessment of existing buildings [21] indicate that the damage occurring at this LS has to be easily and economically repaired.

As far as the infills are concerned, it can be assumed that the requirement corresponding to such a criterion is that the infill panel has not achieved the inter-storey drift corresponding to its maximum resistance, as some experimental results pointed out ([15], [17]), although at this stage some cracks have already occurred. In literature, different assumptions can be found regarding the diffusion of this kind of damage (that is, the proportion of panels reaching the maximum resistance displacement) to be assumed as corresponding to the attainment of DL LS. As a matter of fact, according to different authors this LS occurs when the maximum resistance displacement is reached in the first infill panel ([15], [17], [22]), in 50% of the infill panels at one storey ([23]), or in all of the infill panels at one storey [4]. Based on the results of experimental studies presented in literature ([15]-[19]), it is observed that the displacement corresponding to such maximum resistance condition is strictly dependent on the mechanical properties of both RC frame and infill panel, leading to an IDR ranging from 0.1 to 0.8%.

Code provisions for seismic assessment at DL LS regarding the specific role of infills are present, but – even if some recommendations for the prevention of damage to infill panels are provided ([20], [21], [24]-[28]) – an effective design procedure has not yet been proposed. Some additional measures for masonry infilled frames are introduced at higher level of seismic demand, in order to take into account the effects of the irregularity in plan or elevation produced by the infills, or the possible local detrimental effects due to interaction between infills and the surrounding frame, and to avoid possible brittle failures or out-of-plane collapse of slender panels. In order to effectively account for the damage occurring to infill panels, a seismic assessment at DL LS should be carried out using a model that explicitly takes into account these elements, and assuming a displacement limit directly correlated to their damage.

However, when assessing seismic capacity at DL LS, technical codes ([20], [21], [24]) allow to take into account the presence of infills – if a bare numerical model, as usual, is used – by assuming a fictitious displacement capacity limit (e.g., 5‰ IDR). The Italian "Circolare Esplicativa" [25] allows to limit the maximum IDR in a RC structure to values related to masonry (e.g., 3‰ for unreinforced masonry) if infills are explicitly taken into account in the numerical model. The conservatism of such a provision – depending on the number of storeys and the design typology – have been analyzed in N2 framework and some attempts to identify an equivalent displacement limit to be adopted for a bare model at DL LS depending on the design typology, the number of storey and, above all, on the mechanical properties of infill panels have been performed [9]. Other literature studies [15] suggest to use a conservative lower bound value of IDR limit, i.e. 3‰. However, further efforts to extend the concept of drift limitations to obtained some comprehensive design criteria for RC structures, depending on strength and stiffness properties of infill panels, should be produced.

If a linear analysis approach is adopted, the choice of the stiffness assumed for structural and non-structural elements is not trivial. In linear analyses at Serviceability LSs, it should take into account the influence of first cracks which occur also for low level of seismic demand. Generally speaking, the estimation of such "effective" stiffness should depend on displacement demand on the structural elements, and thus it should be different for each element in the structure, depending on the level of displacement demand involving each of them. However, this kind of – necessarily iterative – procedure should require a computational demand which seems to be not compatible with the level of approximation that implicitly characterizes a linear analysis. Hence, for a RC structure, when an effective stiffness is employed for a linear analysis, a unique value of degradation of the elastic stiffness (pre-cracking) is usually applied to RC members.

As far as RC structural members are concerned, several indications are provided. Some authors (e.g., [29]) suggest to use the secant-to-yielding stiffness in order to predict displacement and forces demand with a good approximation with respect to a non linear analysis, as far as ultimate limit states are concerned. EC8-part 1 [20] prescribes an effective stiffness equal to one-half of the corresponding elastic one, often overestimating the secant-to-yielding stiffness [30]; such an overestimation obviously implies a non-conservative underestimation of the displacement demand. Thus, in EC8-part 3 [21] the adoption of a secant-to-yielding stiffness is explicitly suggested for checks in terms of displacements. ASCE SEI 41/06 – Supplement I [26] suggests to use the secant-to-yielding stiffness, too, and the value of the effective stiffness for RC members depends on the kind of element and the axial load ratio. New Zealand standards [27] prescribe a value of effective stiffness which not only depends on the kind of element and the axial load ratio, but also on the considered LS, on the ductility capacity at Ultimate LS, and on the steel yield strength.

When infills are modeled for a linear analysis, the evaluation of their stiffness is a key point, but, compared with RC members, literature and code provisions regarding this issue are much more limited. Such "effective" stiffness should ideally account for stiffness degradation due to first cracks and detachments between the infill and the surrounding frame. It should be intermediate between the initial elastic (pre-cracking) stiffness and the secant-to-maximum stiffness, provided that such point is assumed as corresponding to the limit displacement capacity at DL LS. If an equivalent strut approach is adopted, the problem of evaluating the stiffness assumed for the infill panel is translated into the determination of the strut width. EC8-part 1 [20] does not provide a specific value of width-to-length ratio for the equivalent strut. Paulay and Priestley (1992) [31] suggest a value of this ratio equal to 0.25 in order to estimate the secant stiffness corresponding to a lateral load equal to 50% of the maximum load capacity of the infilled frame; in Fardis (2009) [30] a width-to-diagonal length ratio equal to 0.20 is proposed at DL LS, whereas a lower value (namely, 0.10-0.15) is suggested at Significant Damage LS.

3 NUMERICAL MODELING

3.1 Case study frames

The case study structures analyzed in this paper are infilled RC planar frames with five equal-length bays, with a bay length equal to 4.5 m and an inter-storey height equal to 3.0 m. The analyzed frames are extracted from a 3-D structure symmetric in plan, both in longitudinal and in transverse direction, with five bays in longitudinal direction and three bays in transverse direction. Slab way is always parallel to the transverse direction; dead load is equal to 7 kN/m^2 for the all stories; live load is equal to 2 kN/m^2.

Starting from different design typologies and number of storeys, four case study frames are extracted and analyzed:

- two gravity load designed (GLD) frames, a four-storey and an eight-storey frame, defined by means of a simulated design procedure according to code prescriptions and design practices in force in Italy between 1950s and 1970s ([32], [33]);
- two seismic load designed (SLD) frames, a four-storey and an eight-storey frame, designed for seismic loads according to the current Italian code [24] in Ductility Class High; hence, the principles of the Capacity Design are applied.

In each frame, infill panels are uniformly distributed along the height (see Figure 1). Panel thickness is equal to 200 mm, corresponding to a double layer brick infill (120+80) mm thick, which can be considered as typical of a non-structural infill masonry wall [34]. Presence of openings is not taken into account.

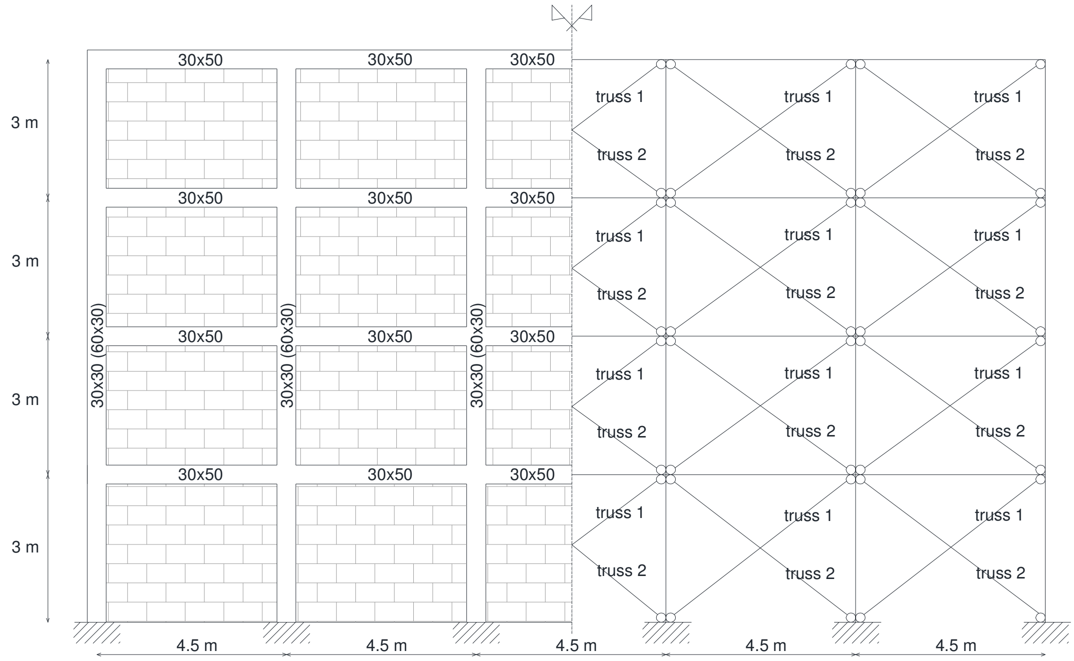

Figure 1: Four storey case-study infilled RC frame; member dimensions for GLD (and SLD) are provided.

3.2 RC and Infill modeling

Nonlinear response of RC elements is modeled by means of a lumped plasticity approach: beams and columns are represented by elastic elements with rotational hinges at the ends. A three-linear envelope is used, with cracking and yielding assumed as characteristic points. Section moment and curvature at cracking and yielding are calculated on a fiber section, for an axial load value corresponding to gravity loads. The behavior is assumed linear elastic up to cracking and perfectly-plastic after yielding. Rotation at yielding is evaluated through the formulations given in [35]. It is worth noting that displacement demand in RC members is typically low at the investigated LS, and in the analyses carried out in this paper no yielding of RC members was observed (for this reason the behavior after yielding is represented by a dashed line in Figure 2(a)). As far as the hysteretic behavior of RC members is concerned Figure 2(a)), no pinching of force and deformation is introduced, no damage due to ductility and energy, and degradation in unloading stiffness based on ductility are taken into account. Strength deterioration becomes an important factor when the structural response approaches the collapse limit state; at earlier steps of inelastic behavior, both deteriorating and non-deteriorating systems exhibit similar responses [36]. Since the inelastic demand on RC member is expected to be very low at the investigated LS, these hypotheses are not reasonably expected to introduce any significant lack of generality. Moreover, in new structures, with detailing of members for ductility, cyclic degradation of strength appears to be negligible [30].

Infill panels are modeled by means of equivalent struts. Modeling infills through single compressed struts allows to investigate the effects of the panels on the global behavior of the analyzed structure, consistently with the purpose of this study. The adopted model for the envelope curve of the force-displacement relationship is the model proposed by Panagiotakos and Fardis [37]. This force-displacement envelope is composed by four branches, as shown in Figure 2(a). The first branch corresponds to the linear elastic behavior up to cracking; the slope of this branch is the elastic stiffness of the infill panel k_{el}, and it can be expressed according to Equation (1), being A_w is the transversal area of the infill panel, G_w the shear elastic modulus and h_w its clear height. The assumption of such a model for the elastic (pre-cracking) stiffness of infills has demonstrated to provide a good agreement between numerical analysis and dynamic behavior of infilled RC buildings [38]. If τ_{cr} is the shear cracking stress, the shear cracking strength F_{cr} can be obtained according to Equation (2).

$$K_{el} = \frac{G_w A_w}{h_w} \tag{1}$$

$$F_{cr} = \tau_{cr} A_w \tag{2}$$

The second branch continues up to the maximum strength F_{max}, which can be calculated according to Equation (3). The corresponding displacement Δ_{max} is estimated according to the hypothesis that the secant stiffness up to maximum is provided by Mainstone's formulation [39], assuming that the width of the equivalent truss b_w depends on the height and the diagonal length of the panel, h_w and d_w respectively, and on the parameter λ_h (Equation (4)); the latter parameter depends on the elastic Young modulus of the infill panel E_w and of the surrounding concrete E_c, the diagonal slope of the equivalent truss to the horizontal θ, the infill thickness t_w, the moment of inertia of the adjacent columns I_c (Equation (5)). Secant stiffness up to maximum, k_{sec}, is provided by the expression shown in Equation (6).

$$F_{max} = 1.30 \cdot F_{cr} \tag{3}$$

$$b_w = 0.175 \left(\lambda_h h_w \right)^{-0.4} d_w \tag{4}$$

$$\lambda_h = \sqrt[4]{\frac{E_w t_w \sin(2\theta)}{4 E_c I_c h_w}} \tag{5}$$

$$K_{sec} = \frac{E_w b_w t_w}{\sqrt{L^2 + H^2}} \cos\theta \tag{6}$$

The third branch of the envelope is a degrading branch up to the achievement of a constant branch. Due to the definition itself of DL LS, the field of behavior of the infill trusses after the peak is never reached (thus the behavior after the peak is represented by a dashed line in Figure 2(b)).

Starting from this model and noting that the infill panels are all equal, the IDR corresponding to the peak strength is about equal to 1.4‰.

As far as the response of the equivalent masonry strut due to cyclic loading is concerned, no strength and stiffness cyclic degradation is considered, as shown in Figure 2(b), basically due to lack of data leading to high uncertainties and modeling difficulties ([40]-[42]). However, further studies should also consider this degradation investigating on the influence of hysteresis rules on the seismic behavior at the analyzed LS.

The viscous damping is represented by a mass and current stiffness proportional Rayleigh model, assuming 2% damping for the first and the third modes.

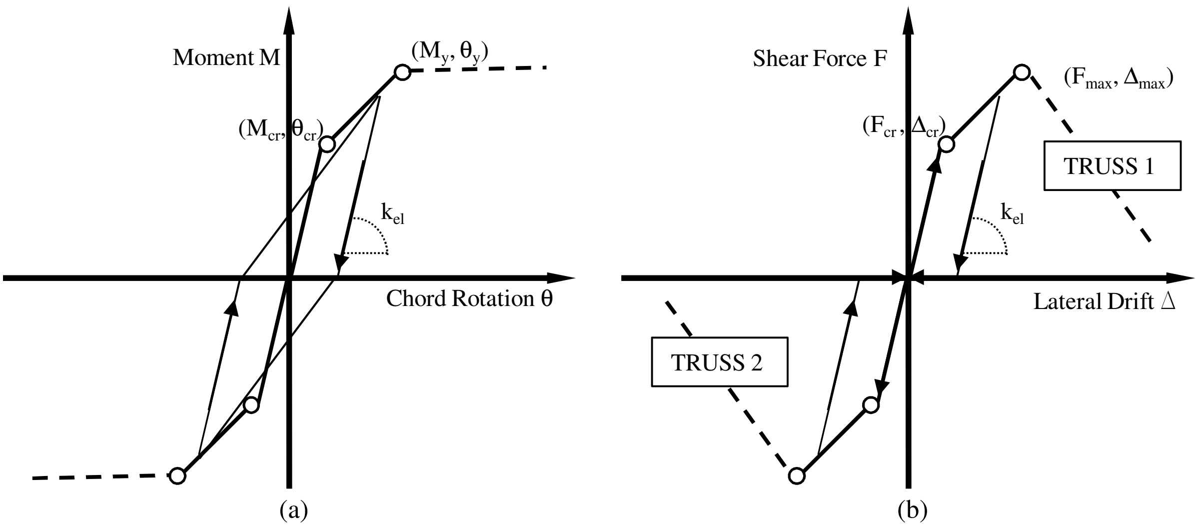

Figure 2: hysteretic behavior of RC members (a) and “simple hysteresis rule” [40] of infill panels (b)

A summary of the mechanical properties of both RC and infills is reported in Table 1.

	Mechanical property	GLD	SLD	References
RC	Concrete compressive strength, f_c	25.0 MPa	36.0 MPa	[43],[45]
	Steel yield strength, f_y	369.7 MPa	550.0 MPa	[44],[46]
Infill	Shear elastic modulus, G_w	1240 MPa		[47]
	Young elastic modulus, E_w	4133 MPa		[47]
	Shear cracking stress, τ_{cr}	0.33 MPa		[47]
	Softening-to-elastic stiffness ratio, α	1%		[37]
	Residual-to-maximum stress ratio, β	1%		[37]
	“Peak” IDR, IDR_{DL}	1.4‰		from model [37]

Table 1: Mechanical properties of RC and Infill

4 ANALYSIS METHODOLOGY

The methodology applied to search the effective stiffness for infill panels starts from Incremental Dynamic Analyses (IDA) ([48], [49]): the non-linear structural model is investigated through time history analyses under the action of a set of ground motion records; the non-linear time-history analysis is repeated increasing the scale factor of the record, for each record, thus obtaining a relationship between a ground motion intensity (spectral acceleration $S_a(T_1,5\%)$) and an engineering demand parameter (maximum IDR) for the structural model. Then, linear time-history analyses were performed, assuming a linear behavior for both RC members and infill panels.

Structural modeling and numerical analyses are performed through the “PBEE toolbox” software [50], combining MATLAB® with OpenSees [51], modified in order to include also infill elements ([52], [6]). A solution algorithm has been introduced in PBEE toolbox in order to solve non-converging problems, trying different possible solution algorithms, or reducing integration time step, or reducing tolerance as suggested in [53]. Moreover the “hunt and fill” procedure suggested in ([48], [49]) is adopted to trace IDA curves.

The definition of DL LS has to be considered still an open issue (see Section 2). In this study, this LS is conservatively assumed to occur when the maximum resistance displacement is reached in the first infill panel, thus starting to degrade. However, in the case study structures analyzed herein the clear span of each infill panel is almost the same, and all the infills at the same storey have the same displacement capacity, thus reaching their maximum resistance at same time. However, this issue could deserve further discussion in the future.

4.1 Ground motion record selection

Natural records are selected and scaled to different level of seismicity in order to obtain IDA curves. The record selection has been performed by using REXEL software [54], from the European Strong-motion Database, between earthquakes characterized by a magnitude which ranges between 6 and 7, with a source-to-site distance ranging between 0 and 30 km, and recorded on soil class A.

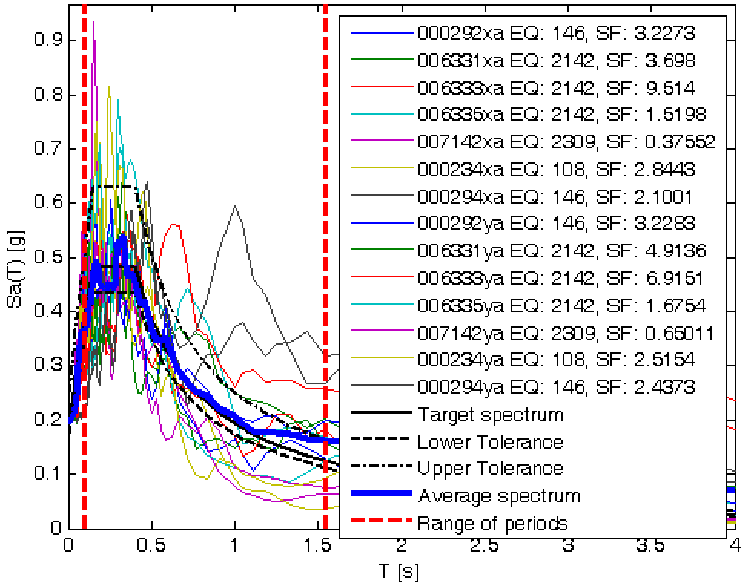

Figure 3: Selected ground motion records [54]

The average spectrum of the selected records matches the target spectrum in the range between 0.1s and 1.5s, with a lower tolerance of 10% and an upper tolerance equal to 30%. The target spectrum is the EC8 spectrum-Type 2 related to soil class A.

A summary of the main properties of the two components of the seven selected records is reported in Table 2. On the whole, 14 records are used in the dynamic analyses.

Earthquake Name	Date	M_w	Fault Mechanism	Epicentral Distance [km]	PGA_X [m/s^2]	PGA_Y [m/s^2]	EC8 Site class
Campano Lucano	23/11/1980	6.9	normal	25	0.5878	0.5876	
South Iceland	21/06/2000	6.4	strike slip	22	0.5130	0.3860	
South Iceland	21/06/2000	6.4	strike slip	28	0.1994	0.2743	
South Iceland	21/06/2000	6.4	strike slip	15	1.2481	1.1322	A
Bingol	01/05/2003	6.3	strike slip	14	5.0514	2.9178	
Montenegro	24/05/1979	6.2	thrust	30	0.6669	0.7541	
Campano Lucano	23/11/1980	6.9	normal	26	0.9032	0.7783	

Table 2: Selected ground motion records

4.2 Procedure: research of the effective stiffness of infill panels

First of all, the non-linear structural model is investigated through time history analyses under the action of the selected set of ground motion records, thus obtaining an IDA curve for each record in terms of peak IDR versus elastic spectral acceleration $S_a(T_1,5\%)$, where T_1 is the fundamental period of the infilled frame.

The attention is focused on the DL LS defined as dependent on a displacement capacity limit directly related to the damage to infill panels and, in particular, to the IDR corresponding to the achievement of the maximum strength of the infills, IDR_{DL}. As reported in Table 1, in the cases investigated in this paper, IDR_{DL} is equal to 1.4‰. Thus the median value of $S_a(T_1,5\%)$ corresponding to this IDR limit, $\eta_{Sa(T1,5\%)|IDR}$, can be evaluated from the previously obtained set of IDAs. This value of $S_a(T_1,5\%)$ is the Intensity Measure (IM) capacity at DL LS, $S_a(T_1,5\%)_{DL}$.

Then, Linear Time-History (LTH) analyses are performed, assuming a linear behavior for both RC members and infill panels, and assuming a reduction factor α of the initial elastic stiffness of all the infill trusses (evaluated according to Equation (1)).

As shown in Figure **4**, an iterative procedure is applied, varying the coefficient α between 0 (infinitely flexible infills) and 1 (infills with elastic stiffness). For each α value, LTH analyses for all of the selected records are performed, and the median value of the maximum IDR, $\eta_{IDR|Sa(T1,5\%)}$, is estimated at the IM level equal to $S_a(T_1,5\%)_{DL}$. The iterative procedure stops when the value of $\eta_{IDR|Sa(T1,5\%)}$ is equal to IDR_{DL}, i.e. 1.4‰ in our case; the corresponding α factor, $\alpha_{kel,infill}$, is the reduction factor of the initial elastic stiffness of infills providing an effective stiffness of infill panels such that a linear dynamic analysis of the infilled numerical model leads to a displacement demand in terms of maximum IDR at DL LS which is approximately equal to the maximum IDR demand evaluated through non linear dynamic analysis on the same model.

In Figure **5** a schematic example of results of the procedure explained herein is reported.

It is worth noting that, since that it is not possible to know a-priori which storey is involved in the achievement of DL LS, the reduction factor is applied to all of the infill panels. Such a procedure allows to calibrate the factor α that provides the "real" maximum IDR demand – at a certain level of intensity measure – through a linear analysis, but an error may occur in the estimation of the corresponding top displacement. This could be considered as an unavoidable approximation due to the limitation of a linear analysis itself, which is not able to capture the effects of a concentration in post-elastic displacement demand. However, this error could be considered not of a primary importance, since DL LS check is performed in terms of another displacement demand parameter, namely maximum IDR.

Non linear behavior of RC members can be considered, too, by repeating the described procedure for different value of effective stiffness of RC members. Such an effective stiffness can be estimated as a rate (α_{RC}) of the elastic (pre-cracking) stiffness of each RC member, e.g. 50% (EC8-part 3). For each value of α_{RC}, a $\alpha_{kel,infill}$ factor can be estimated.

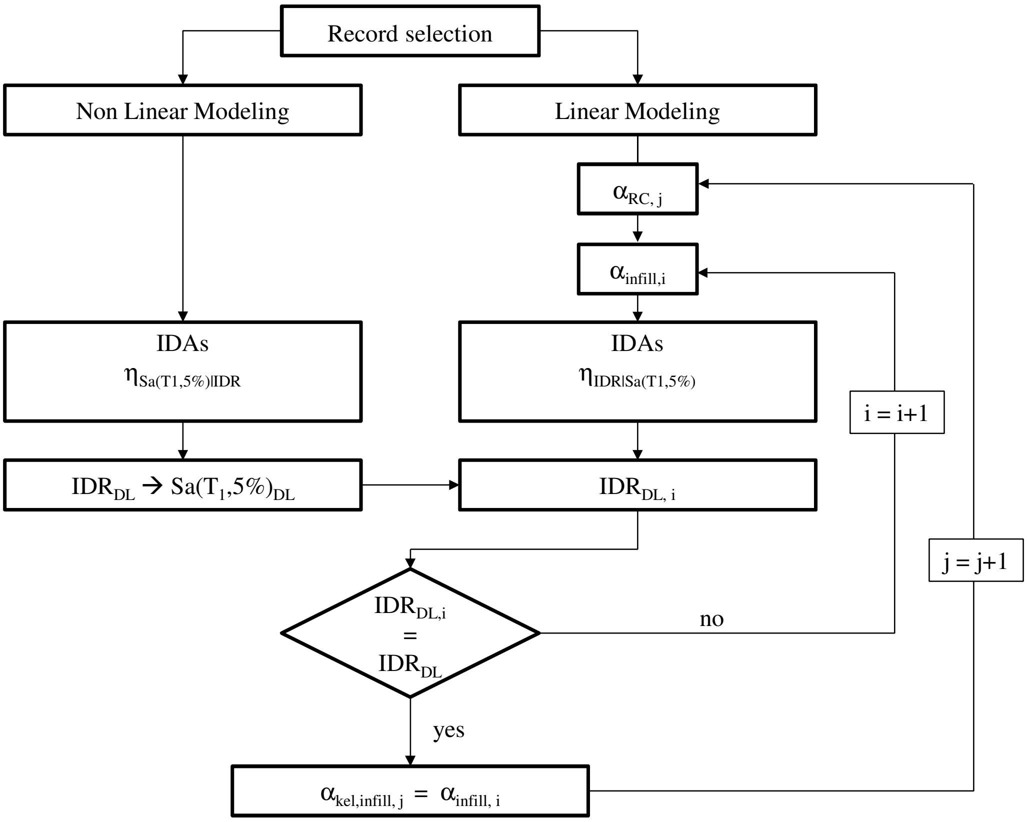

Figure 4: Steps of the procedure: research of the effective stiffness for infill panels

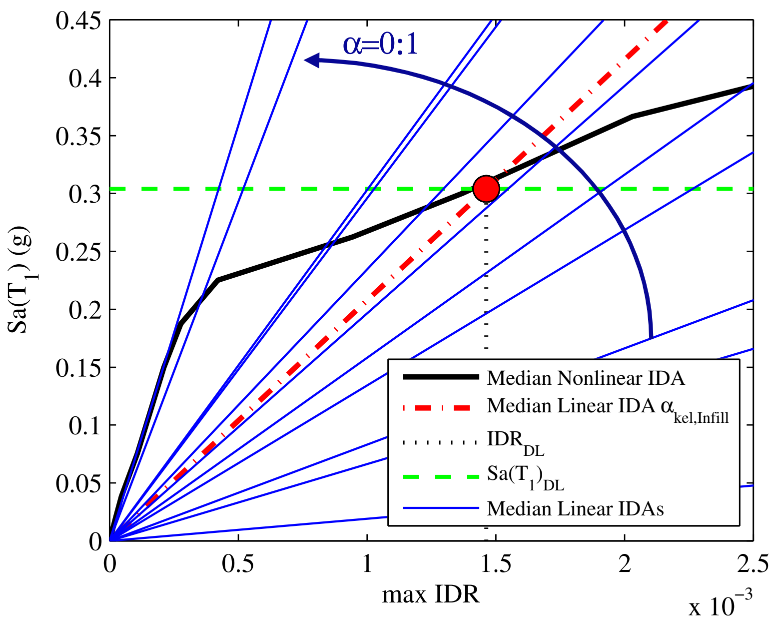

Figure 5: Schematic example of the results of the procedure

5 ANALYSIS OF RESULTS

Hence, the procedure described in the previous paragraph is applied for each of the case-study frames.

In Table 3, the first-mode periods, the PGA capacity at DL LS and $S_a(T_1,5\%)$ capacity at DL LS, respectively, are shown for each case study frame.

	GLD			SLD		
	T_1 (s)	PGA_{DL} (g)	$Sa(T_1)_{DL}$ (g)	T_1 (s)	PGA_{DL} (g)	$Sa(T_1)_{DL}$ (g)
4 storey	0.122	0.32	0.32	0.114	0.41	0.39
8 storey	0.268	0.19	0.28	0.239	0.22	0.30

Table 3: Elastic periods of infilled frames; PGA and $Sa(T_1)$ capacity at DL LS

The results of such a procedure in terms of non linear IDA curves and LTH analyses are reported in Figure 6, where $S_a(T_1,5\%)$ capacity at DL LS and the IDR corresponding to the achievement of DL LS are represented, too.

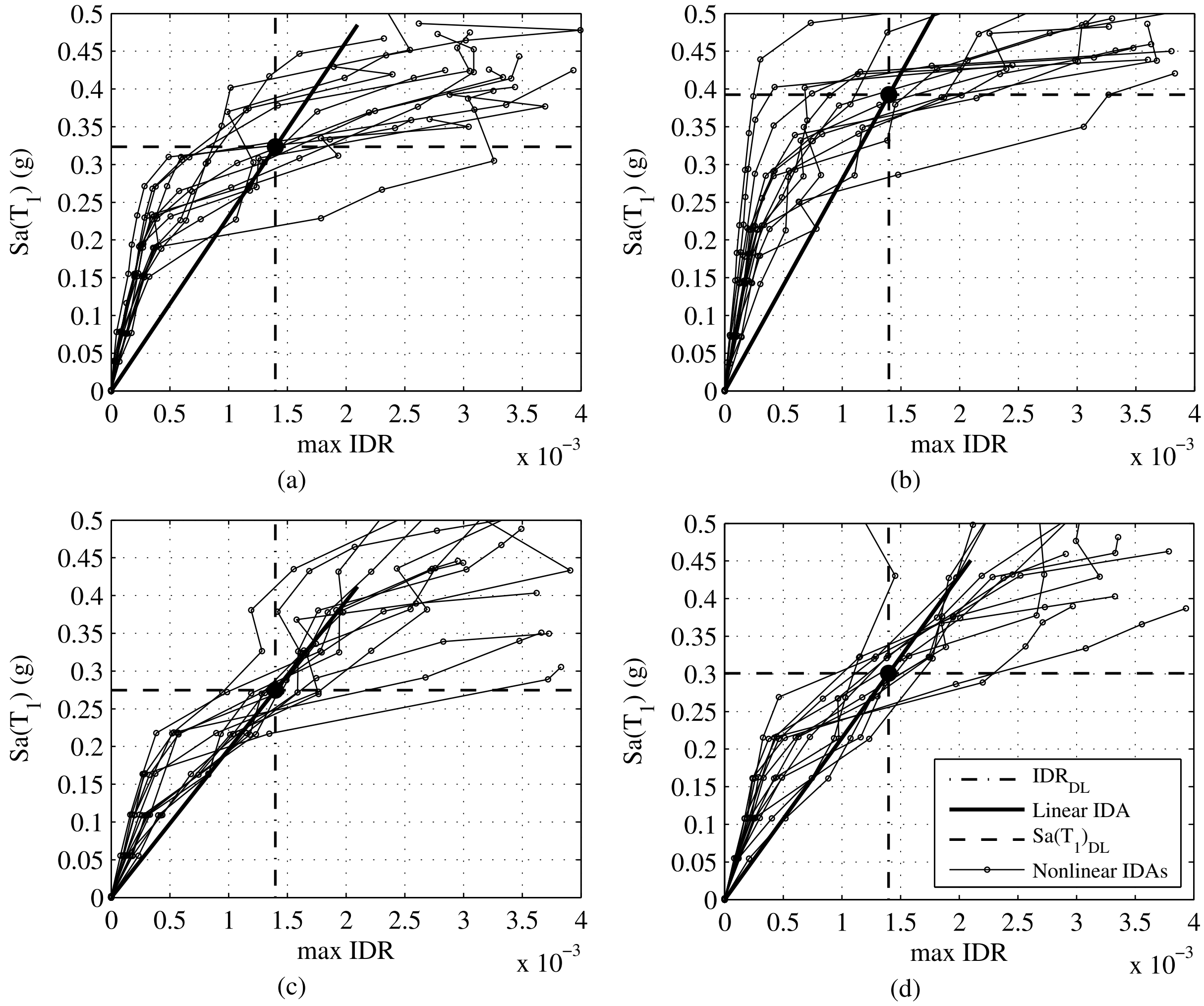

Figure 6: Non linear IDAs and Linear IDA for $\alpha_{kel,infill}$: 4-storey GLD (a) and SLD (b) frames, 8-storey GLD (c) and SLD (d) frames

The distribution of IDR demand corresponding to the achievement of DL LS can be evaluated through the non linear analyses as the median of the IDR demand at each storey for all of the records when an IM value equal to the IM capacity at DL LS is considered. A concentration of IDR demand is shown at lower storeys both for 4- and 8-storey frames, as shown in Figure 7. It is worth noting that the maximum IDR demand shown in this figure can be a bit lower than the IDR_{DL} limit because of the procedure by which it is estimated: if the IDR_{DL} is achieved at different storeys when different records are analyzed, the median value of the maximum IDR demand at each storey for all of the records will result lower than IDR_{DL}. Finally, the values obtained for $\alpha_{kel,infill}$ and the corresponding elastic periods of the case-study frames with elastic stiffness of infills reduced by $\alpha_{kel,infill}$ are reported in Table 4.

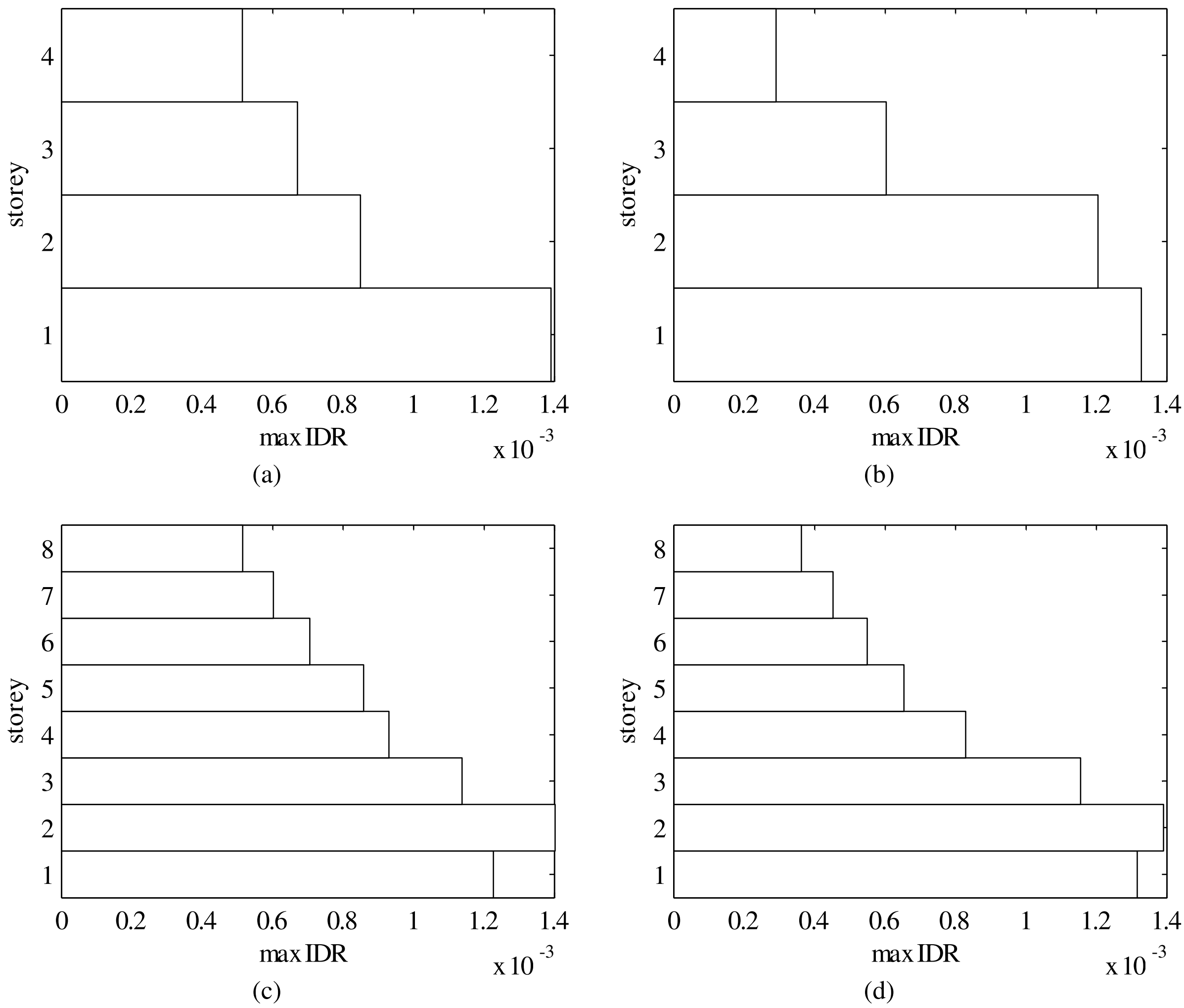

Figure 7: IDR demand at each storey: 4-storey GLD (a) and SLD (b) frames, 8-storey GLD (c) and SLD (d) frames

	GLD		SLD	
	$\alpha_{kel,infill}$	$T(\alpha_{kel,infill})$ (s)	$\alpha_{kel,infill}$	$T(\alpha_{kel,infill})$ (s)
4 storey	0.31	0.202	0.34	0.196
8 storey	0.36	0.351	0.41	0.331

Table 4: Obtained values of $\alpha_{kel,infill}$ (with α_{RC}=1) and corresponding elastic periods

When also non linear behavior of RC members is considered, by repeating the procedure for different value of effective stiffness of RC members, i.e. for different values of α_{RC}, a vector of $\alpha_{kel,infill}$ can be estimated.

In Figure 8 and in Table 5 the variation of $\alpha_{kel,infill}$ depending on the investigated values of α_{RC} is represented for each case-study frame. Figure 8 clearly shows that if α_{RC} decreases, a higher effective stiffness has to be assumed for infill panels in order to obtain the same displacement demand, as expected. However, the curves representing the variability of $\alpha_{kel,infill}$ depending on α_{RC} have a very low slope: the value of $\alpha_{kel,infill}$ is almost constant when the effective stiffness of RC members ranges between zero-stiffness and the gross elastic stiffness. This trend is more evident for the 4-storey GLD frame with respect to the 4-storey SLD frame, because of the increase in the influence of infill panels when RC members have a lower stiffness.

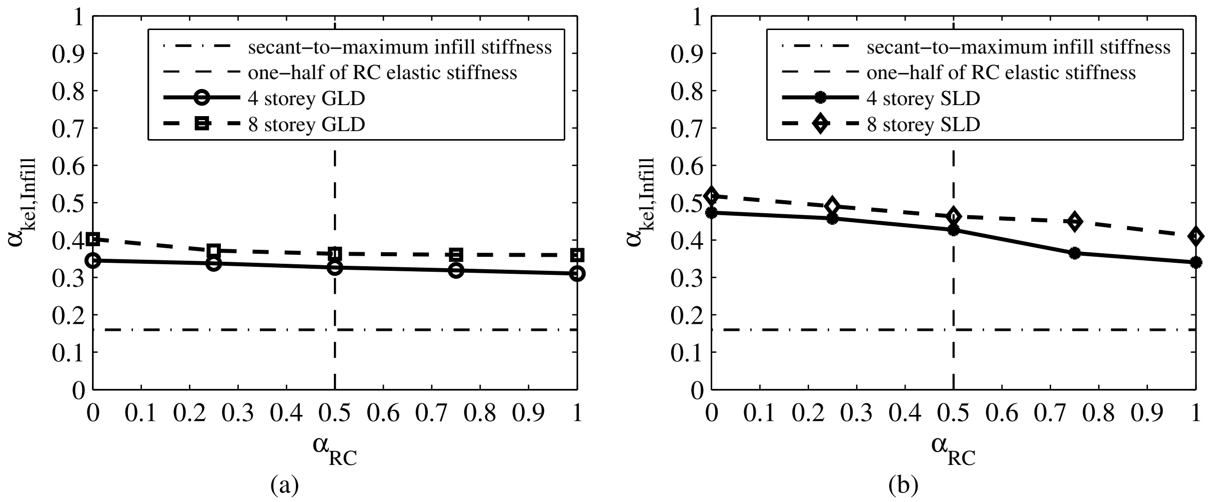

Figure 8: Variation of $\alpha_{kel,infill}$ depending on α_{RC} for GLD (a) and SLD (b) frames

	$\alpha_{kel,infill}$			
α_{RC}	4storey GLD	4storey SLD	8storey GLD	8storey SLD
0	0.35	0.47	0.40	0.52
0.25	0.34	0.46	0.37	0.49
0.50	0.33	0.43	0.36	0.46
0.75	0.32	0.36	0.36	0.45
1	0.31	0.34	0.36	0.41

Table 5: Variation of $\alpha_{kel,infill}$ depending on α_{RC}

If the same design typology is considered, $\alpha_{kel,infill}$ is higher in the case of the 8-storey frames with respect to the corresponding 4-storey frames, whichever α_{RC} is considered. From a qualitative point of view, it could be expected that the reduction in the initial elastic stiffness of infills is lower when it involves a higher number of infill panels, such as in the case of 8-storey frames, since the achievement of DL LS involves always one storey only.

If frames with the same number of storeys are considered, a stronger dependence of $\alpha_{kel,infill}$ on α_{RC} is observed for SLD frames with respect to GLD frames. For example, comparing the 4-storey SLD frame with the 4-storey GLD frame, a higher contribution to lateral strength and stiffness of RC members is present in SLD case: the higher this contribution, the more sensitive the variation of the effective stiffness of infills with respect to the effective stiffness of RC members.

When a α_{RC} equal to 0.50 is adopted for RC members (as suggested in EC8), $\alpha_{kel,infill}$ ranges between 0.33 and 0.46 (Table 5) for the analyzed frames; since DL LS check is performed in terms of displacement, a conservative value of $\alpha_{kel,infill}$ to estimate the effective stiffness of infills is the lower value of this range, i.e. 0.33, thanks also to the quite small variation of $\alpha_{kel,infill}$ depending on α_{RC} and on the characteristics of the frame. Moreover, Figure 8 shows that effective stiffness of infills estimated through $\alpha_{kel,infill}$ is always higher than the secant-to maximum stiffness (Mainstone's stiffness), i.e. $\alpha_{kel,infill}$ equal to 0.16 in the analyzed case-studies. Hence, if an effective stiffness equal to the secant-to maximum stiffness is adopted for infills, DL LS check could be too conservative. Considering the geometrical and mechanical properties of the infill panels modeled in the case-study frames (Table 1), a $\alpha_{kel,infill}$ value equal to 0.33 implies a width-to-diagonal length ratio equal to 0.18, quite close to the value proposed in Fardis (2009) [30], where a width-to-diagonal length ratio equal to 0.20 is suggested at DL LS. The variation of that width-to-diagonal length ratio depending on α_{RC} is shown in Figure 9.

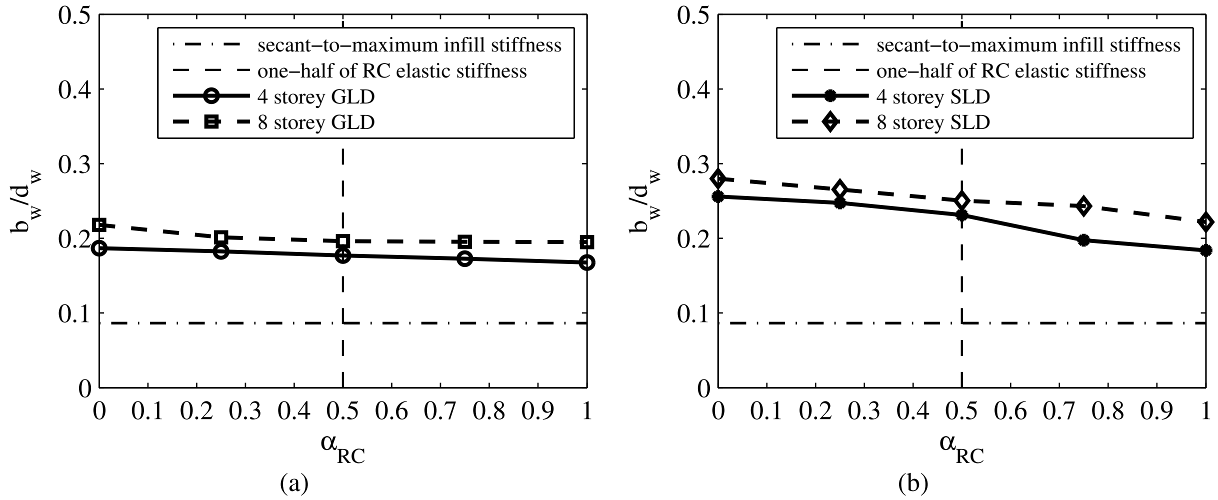

Figure 9: Variation of width-to-diagonal length ratio depending on α_{RC} for GLD (a) and SLD (b) frames

6 CONCLUSIONS AND FUTURE DEVELOPMENTS

Based on the results of several non linear dynamic analyses, a reduction factor of the initial elastic stiffness of infills has been estimated, in order to obtain an effective stiffness of infill panels such that a linear dynamic analysis of the infilled numerical model leads to a displacement demand at DL LS – in terms of maximum IDR – which is approximately equal to the demand evaluated through a non linear dynamic analysis on the same model, under the seismic demand leading to the attainment of such LS.

To this end, IDAs were performed on four- and eight-storey infilled frames, designed for seismic loads according to the current Italian technical code and for gravity loads only according to an obsolete technical code. The influence of number of storeys and design typology on the obtained effective stiffness was analyzed, depending also on the effective stiffness adopted for RC members. In particular, when an effective stiffness equal to one-half of the elastic one is adopted for RC members [20], a value of effective stiffness equal to one-third of the elastic one for infills appears to be a conservative value to perform the DL LS check in terms of displacement.

Work is ongoing to carry out a comparison between LTH results and a Response Spectrum Analysis (RSA) in order to simplify further that kind of analysis. The estimation of the effective stiffness for infills at DL LS could come from an equivalence between non linear time history and RSA analysis, since the latter is the most widespread method of analysis, especially for seismic design of new structures.

Further studies should investigate the effects of a possible variation in masonry infill characteristics, namely displacement capacity and hysteresis rules, on the estimation of the effective stiffness. The influence of the presence of openings in infill panels could be investigated, too. Moreover, RC structural types different from moment resisting frames (i.e., shear wall frames) could be considered.

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