ASSESSMENT OF CURRENT PREDICTION CAPACITIES OF THE RESPONSE OF EXISTING REINFORCED CONCRETE BUILDINGS

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SUMMARY

The main purpose of the paper will be to assess the capacity of more or less refined non linear analysis tools to predict the response of existing, not properly designed reinforced concrete buildings.

The deficiency in expected seismic performance of existing reinforced concrete buildings designed before seismic code provisions more or less equivalent to those currently in practice were implemented has been widely recognised, as a consequence of two major failings in the original design process: poor detailing of reinforcement, and lack of a capacity design philosophy. Typical deficiencies in detailing relate to amount, distribution, and anchorage of transverse reinforcement, and often result in local damage and collapse modes extremely difficult to be numerically simulated. As a consequence of the lack of capacity design considerations in the design process, the global response may be governed by complex post-elastic mechanism, difficult to be assessed.

Very few non linear analysis tools are available all over the world to simulate the response of this class of buildings, and recent comparisons seem to show that significantly different predictions may result. Recently some simplified approaches have also been proposed, based on displacement and energy dissipation capacity assessment. Again, there is little evidence on the reliability of any simplified approach.

To verify and compare the quality of the results obtainable from different modeling options, a simple frame will be considered, discussing the different results that may be obtained and their relevance on the global structural assessment.

INTRODUCTION

It is evident that more concern for human life protection and economical loss should arise from the potential for damage and collapse of existing buildings rather than of structures still to be constructed. Nevertheless, until recently, the scientific community has devoted much more attention to codes for design than to codes for assessment and strengthening.

More precisely the border line between buildings which are likely to respond conceptually in a different way, coincides with the enforcing of seismic codes based on a capacity design (CD) philosophy. Only then actually, the damage and collapse modes have started to be controlled, more or less explicitly, and consequently the results obtained from simplified design approach can be expected to be similar to the actual structural response.

For recent buildings, or better for CD designed buildings, the response appears to be dominated by the flexural response of beams and column bases and in general the available numerical models can be accepted as capable and reliable. Fiber models have been implemented into many codes, and with some exceptions that will be pointed out later, they can reproduce faithfully the response of buildings dominated by flexural problems in well confined regions.

On the opposite, when older, or better when buildings designed without considering any capacity protection of undesired failure modes (from now on called strength-designed (SD) structures), are considered, a number of

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events potentially difficult to be modeled have to be taken into account. These may include loss of bond and bar buckling, shear strength and stiffness deterioration, shear-flexure interaction, joint deformation, deterioration and collapse, contribution of infills with openings.
To assess and discuss the relevance of different modeling options a frame will be considered. The frame has been designed to be a representative example of strength designed structures, has been constructed at full scale and is being tested at the Joint Research Centre of the European Union in Ispra at the time of writing. As shown in Figure 1 the frame consists of 4 storeys and 3 bays. Two specimen have been constructed to be tested: the first one as a bare frame and the second one inserting low strength clay block infills.
The reinforcement amount and distribution is typical of a SD designed structure or of a gravity load designed structure, constructed in the 50's or 60's, as shown for some case in Figure 2. Smooth bars have been used.

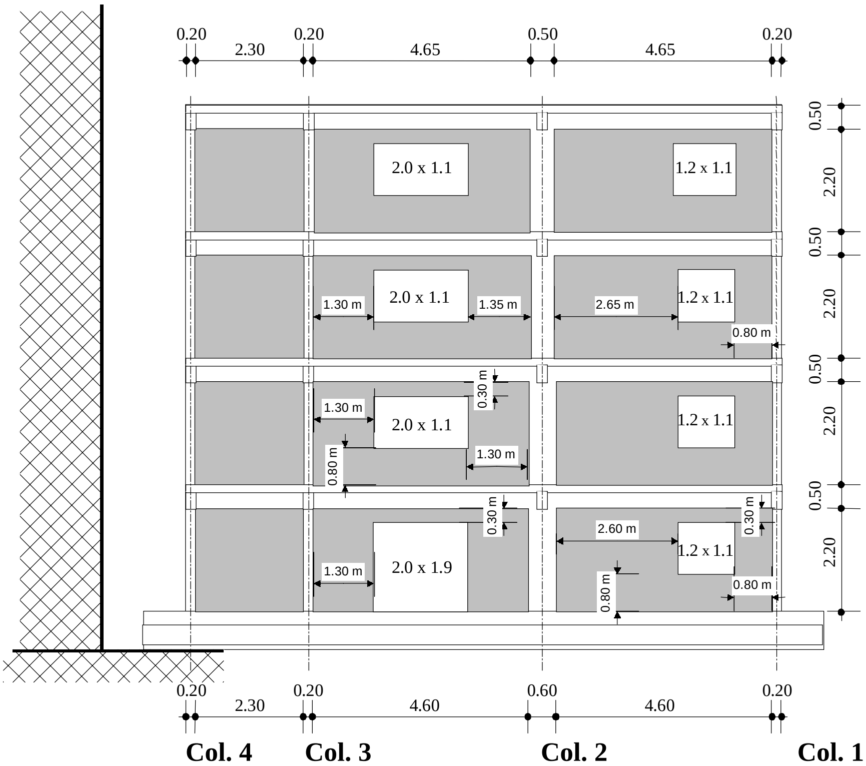

Figure 1: Geometry of the frame including infills

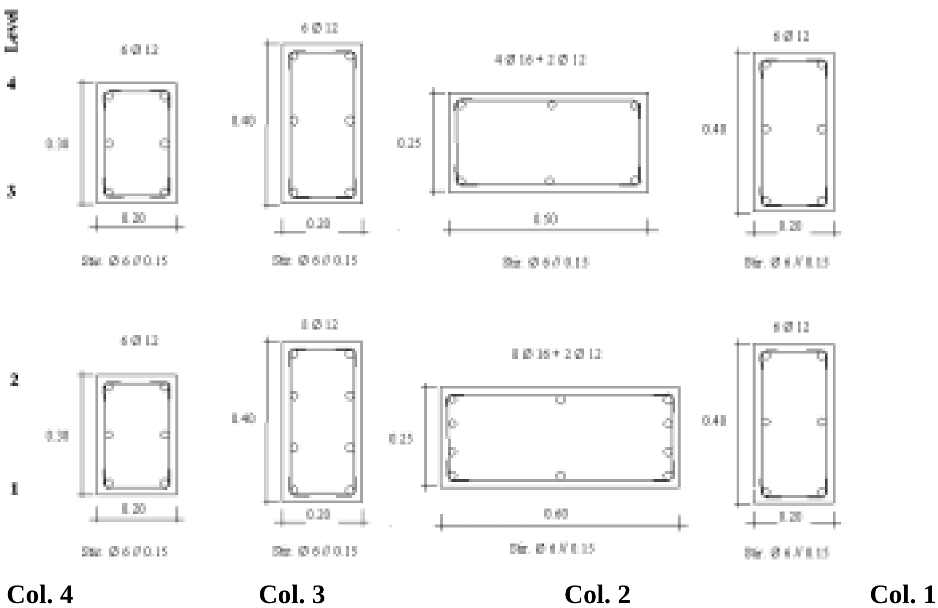

Figure 2: Reinforcement details of columns

ANALYSIS CHOICES

To contain the length of the paper it has been necessary to concentrate on modeling, without exploring a number of possible analysis options. Only a pushover analysis is therefore considered, because it has been gaining popularity and it is more and more commonly used, because it allows an easier discussion of different modeling choices and finally because its more recent popularity may make it more controversial.

In Figure 4 global force-drift curves obtained applying different force or displacement profiles to the bare frame are shown. The computer program ADAPTIC [1] has been used, firstly assuming that only flexural deterioration is taking place.

Curve *A* in Figure 4 corresponds to what is expected to be the most realistic response, having been obtained applying a triangular force distribution to the bare frame. Maybe surprisingly, curves *C* and *D* do not differ too much from curve *A*. Curve *C* corresponds to the expected response with a rectangular force distribution, curve *D* has been obtained applying a triangular displacement distribution. As discussed in the next section, in case *C* a soft storey at the base is assessed (instead of the third soft storey indicated in case *A*), while, obviously, case *D* is unable to predict any local crisis, since the displacement shape is imposed.

Finally curve *B* corresponds to the case of the infilled frame. Again, a relatively minor increase in strength and displacement capacity is assessed.

Influence of possible shear or joint collapses will be discussed later. It can be now temporarily concluded that very different (and in one case clearly wrong) analysis options do not produce very different results in term of global displacement and force capacity. However, this results from completely different displaced shape and may correspond to significantly different energy dissipation capacities and, possibly, to significantly different assessment of maximum peak ground acceleration.

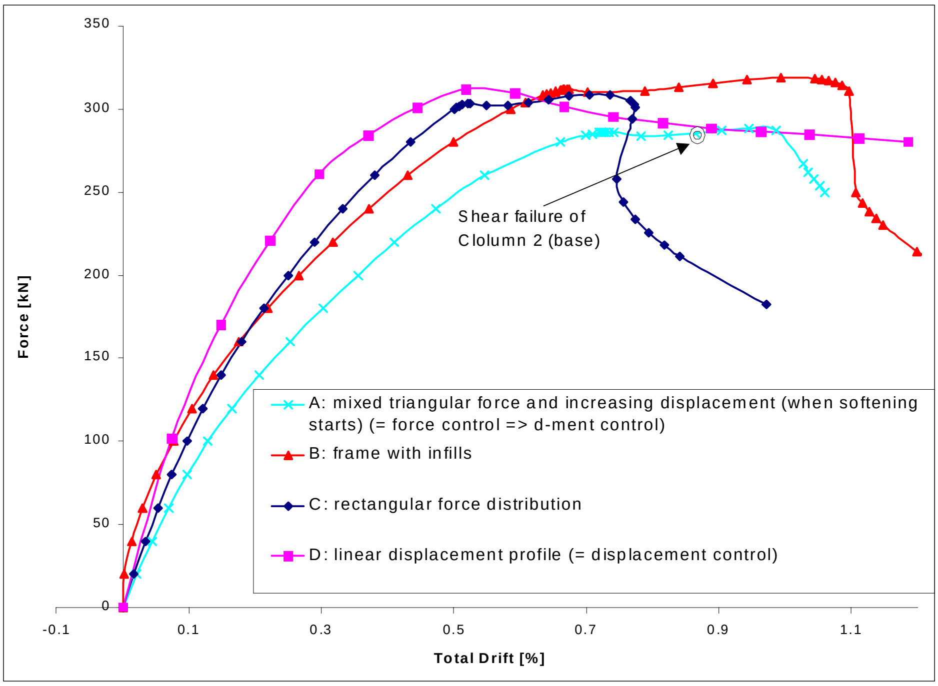

Figure 3: Comparison of force-drift curves obtained imposing increasing forces and displacements

COLLAPSE MODE

As briefly mentioned the collapse mode resulting from the most reliable analysis implies the formation of a soft-storey at the third storey. This is evident in Figures 5 and 6 where the moment curvature relations for all critical sections of column 2 and the displacement profiles obtained at increasing global drift level are depicted. The maximum curvature ductility demand is around 15 and the corresponding drift at the third storey is around 2%. A detailed check of shear demand and capacity at critical sections allows to exclude any potential for shear failure, with the only exception of the base of column 2. Applying the shear formulas proposed in [3] the column should fail in shear at a point corresponding to a global drift equal to 0.85 %, as indicated in Figure 3. It has to be noted that all local collapse modes take place in columns rather than in beams, as shown in Figure 6, where the sequence of plastic hinge formation is depicted, together with the approximate values of curvature ductility

obtained at an average global drift equal to 1.0 %, i.e. close to failure. It may be noted that the final crisis is assessed to be due to column 2, regardless the relatively late plastic hinge formation.
A check of the joint response, applying the equations recommended in [4] and [5], indicate premature joint damage due to excessive tensile principal stress in several joints. Essentially, joint damage should start at a global average drift between 0.35 and 0.70 %, i. e. approximately when also a global mechanism formation is indicated (see Figure 3). Although these events do not correspond to collapse, the progressive joint deterioration certainly contributes to an increased global deformability. Note that for column 1 and 2 the analysis indicate a large contribution of joint rotation to the global deformation of the frame, while column 3 and 4 seem to deform more as a series of double bending column elements.

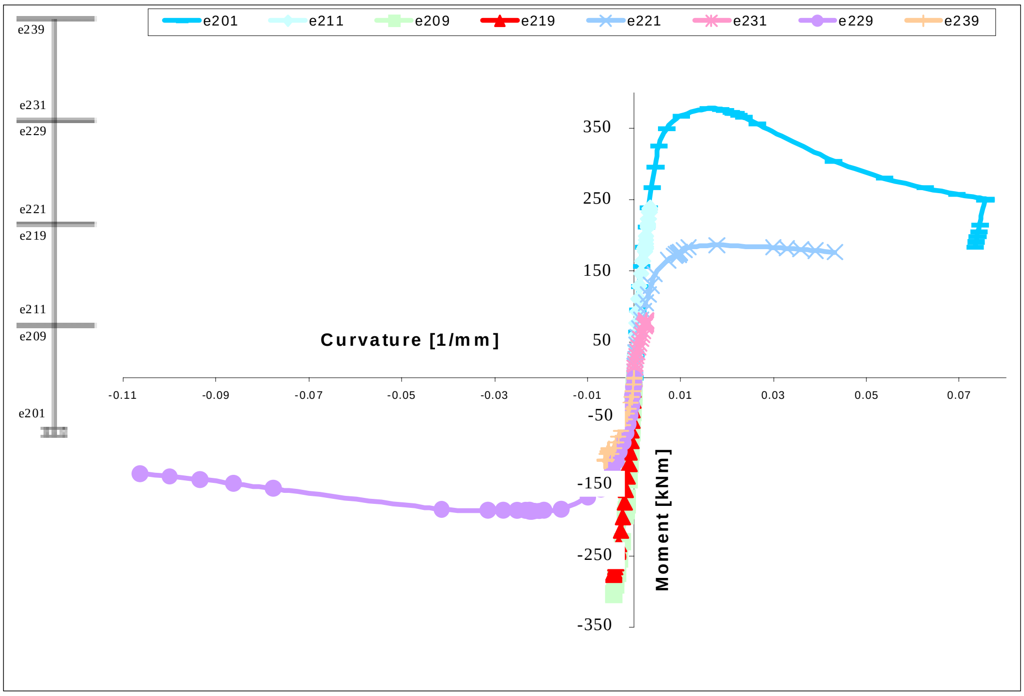

Figure 4: Moment - curvature relations at all critical sections of column 2. The large curvature demands at the third floor (e221, e229) are evident

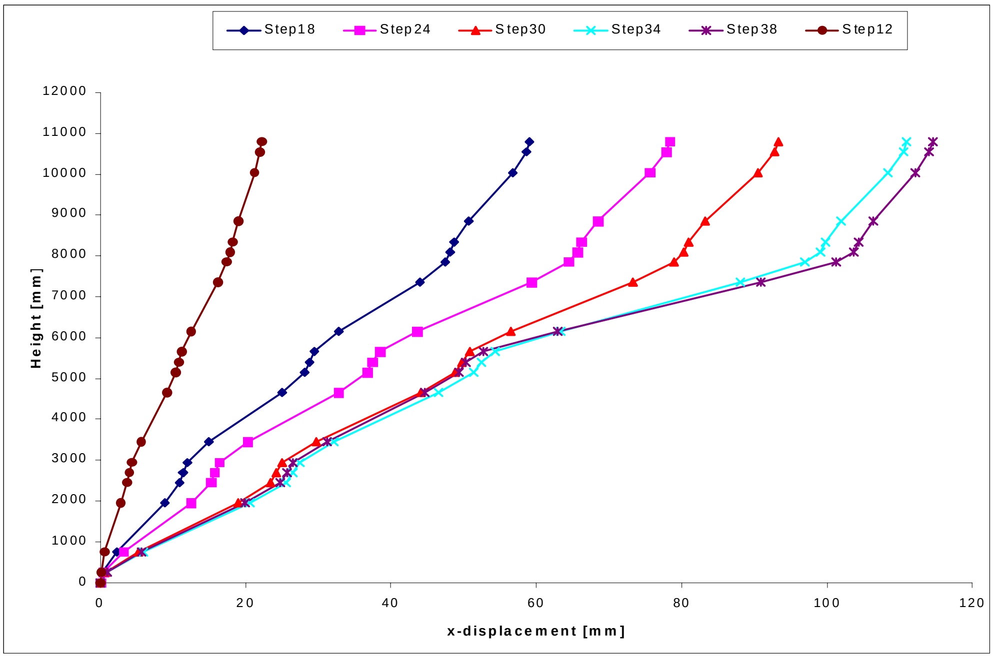

Figure 5: Displacement profiles for different numbers of global drift

Col. 1 **Col. 2** **Col. 3** **Col. 4**

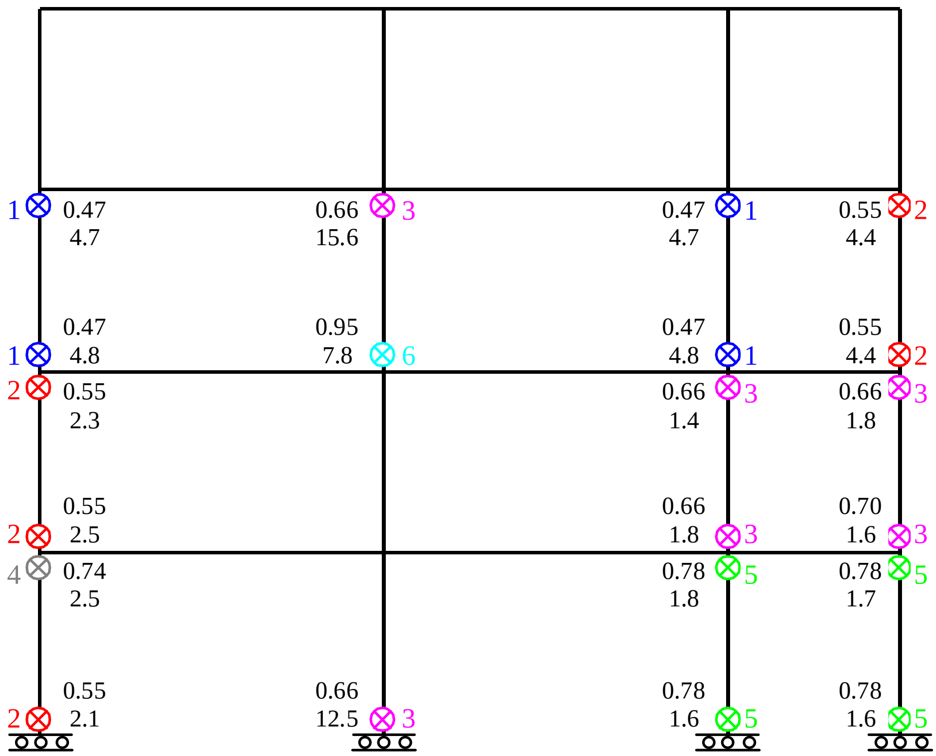

Figure 6: Sequence of plastic hinge formation (1 - 6); The figures indicate the global drift corresponding to the plastic hinge formation (upper) and the curvature ductility demand (lower) at 1% global drift

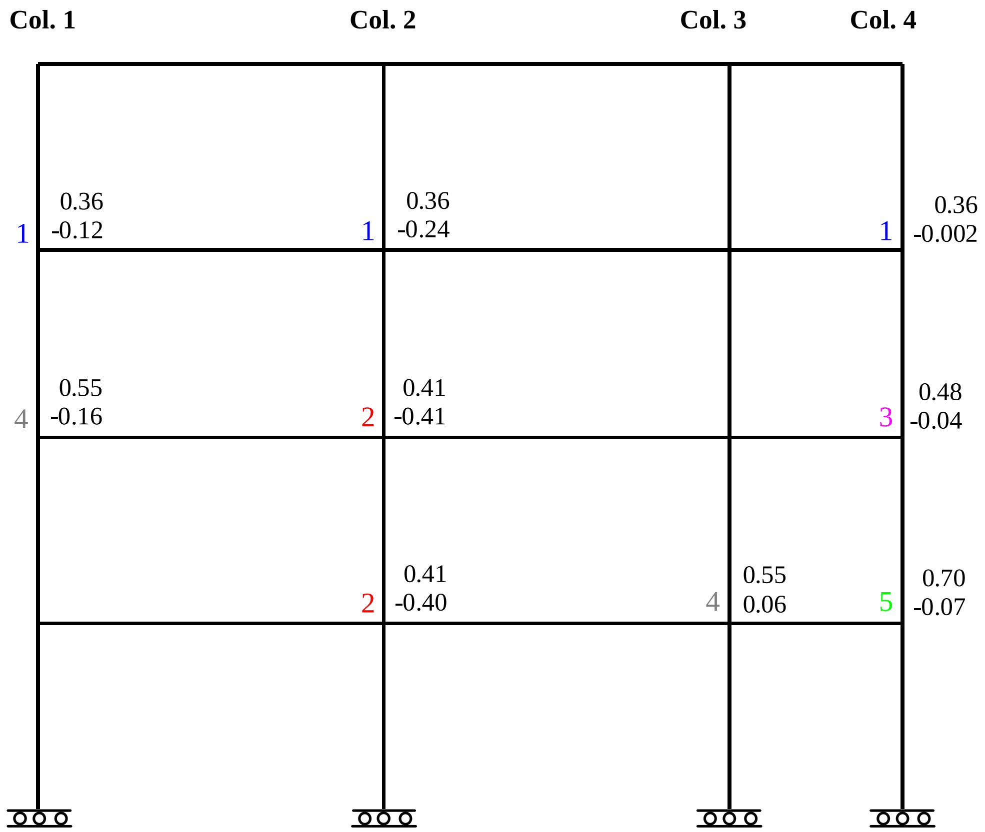

Figure 7: Sequence of joint collapses - the figures indicate the joint rotation [%] corresponding to the detected collapse condition (lower) and the corresponding global drift demand [%](upper)

DISPLACEMENT DEMAND AND CAPACITY

The prediction of actual displacement demand and capacity is probably where numerical analyses and real behaviour differ more significantly. This applies to CD designed buildings, though to a minor extent, as well as to SD designed ones. The reason has to be searched in the many sources of deformation neglected by all kinds of analytical tools, such as penetration of bar slippage into foundations and joints, joint deformability, shear deterioration etc. The difference between displacement predictions derived from semi-empirical formulations established on the base of experimental data and refined numerical programs is surprising. For example, if the approximate relations proposed in [6] and [9] are applied, the equivalent global yielding drift is assessed between 0.9 and 1 %. This value is approximately two times the value indicated by the analysis. Such a ratio is consistent with differences shown by several comparisons between experimental and analytical studies (see for example [10]). Since the sources of additional deformation (such as yield penetration and shear deformation) are strongly influenced by the force level, and the force does not vary significantly after equivalent yielding, it has been often suggested that the post-elastic part of the global force-displacement curve predicted by analytical tools should be essentially correct. Unfortunately this does not apply to the additional deformation due to progressive joint degradation, as previously noted. The increased global average drift is impossible to be assessed with simple empirical equations, due to lack of experimental data, but it is reasonable to assume that also the post-elastic part of the analytical curve should be somehow elongated.

EFFECTS OF INFILLS

The effects of infills on the global response of a frame is not easy to be quantitatively defined, and depends significantly on the nature of the infills. In the frame considered in this exercise, weak infills have been considered, with a compression strength of approximately 1.2 MPa and a Young modulus of approximately 1500 Mpa. The presence of infills has been simulated adding diagonal struts, with appropriate strength and stiffness evaluated according to [7]. The envelope of a force-displacement diagram of a diagonal strut equivalent to the infills is shown in Figure 8.
It has been shown [1] that the global effects of the presence of the infills is generally very important at low level of excitation, while it tends to become less significant for strong earthquake levels.

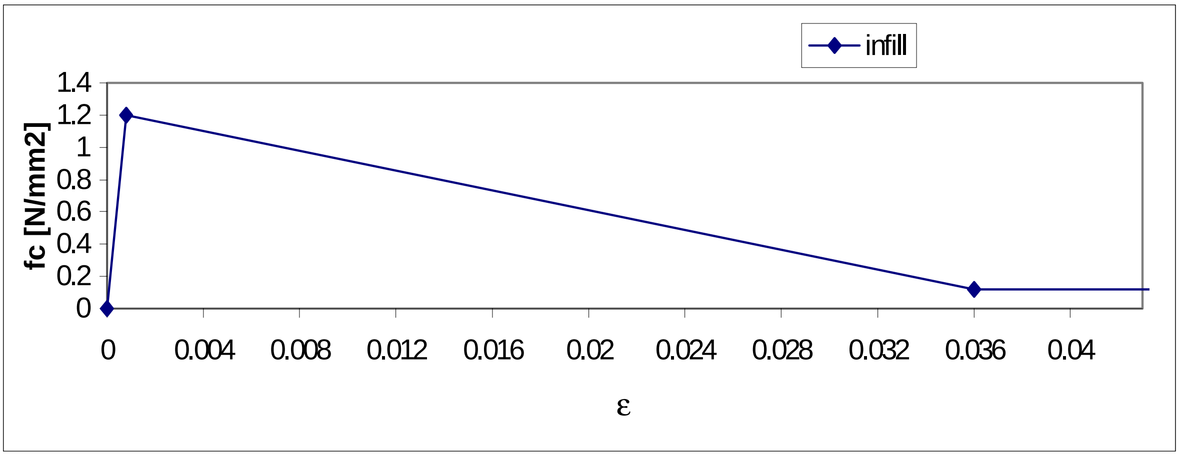

Figure 8: Stress-strain curve for infill material properties

In the present case the use of equivalent struts is made more difficult because of the presence of openings. To have a feeling of the effects of openings on strength and stiffness of the panel, a series of parametric linear analyses have been run using plane stress finite elements. The use of linear constitutive relations is justified by the response essentially brittle of the panels. As shown in Figure 9, both local stress and global displacement seem to vary linearly increasing the area of the openings. Until more complete studies and experimental evidence will be available, it seems appropriate to reduce the area of the equivalent strut according to the following relation:

$$A_{eq\ strut,\ op} = A_{eq\ strut}\,(1 - 5A_{op}/3A_{gross})$$

As already pointed out and shown in Figure 3 (curve B), the presence of infills modify the response indicating a minor increase in strength and deformation capacity. Most of the comments made for the case of the bare frame for what concerns shear deformation and failure, joint damage and global deformation still apply.

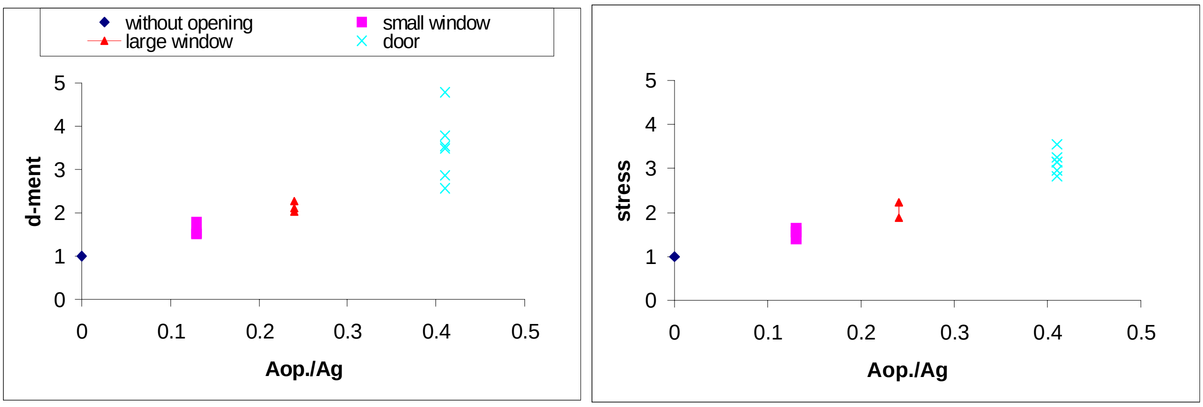

Figure 9: Increase of displacement and representative stress values with raise of the opening area in a single infill

2. VULNERABILITY EVALUATION

The final objective of a response prediction is obviously the assessment of the earthquake intensity corresponding to specific limit states of interest [4]. It is therefore of some interest to evaluate and compare the ground acceleration predicted to induce yielding and collapse, for bare and infilled frames, considering a push-over analysis or applying approximate relations.

This is simple to be obtained when the spectral content of the ground motion is known, considering either acceleration or displacement spectra and applying the correction normally recommended as a function of ductility (force reduction factor) or of energy dissipation (displacement reduction factor) [8, 9].

In the present case, it can be assumed that the frequency content of the earthquake record corresponds to that recommended by EC8 for medium-density soil, since a generated accelerogram of this type will be used for the pseudo-dynamic test.

Considering the results of the push-over analysis, and applying standard relations to compute stiffness, period of vibration and displacement reduction factor [8] the average drifts and shear strengths shown in Table 1 may be assumed for the different cases studied, obtaining the peak ground acceleration levels (a_g) also shown in the table for yielding and ultimate conditions. It may be noted that while the different models predict similar ground acceleration levels to induce equivalent yielding (around 0.11 g), the prediction of the ground acceleration levels required to induce collapse differs significantly, with values between 0.21 and 0.35.

It has been discussed how the deformability, and consequently the displacement capacity, predicted by the analysis are likely to underestimate the real values, probably of a factor of the order of 2. It is therefore of some interest to assess the ground acceleration required to induce yielding and collapse if all curves are shifted increasing all displacements of two times. Note that both strength and ductility are not modified, and in an ordinary strength analysis the initial gross stiffness is also not modified, therefore obtaining the same vulnerability prediction.

The results obtained are shown in Table 2. Again, the acceleration required to induce yielding is consistently predicted around 0.16g, while the collapse acceleration varies between 0.29 and 0.49g. It is the opinion of the authors of this paper that 0.16 and 0.33g should be considered the most appropriate prediction for the acceleration levels to be applied to the bare frame to induce yielding and collapse and similarly 0.17 and 0.38g should be predicted for the case of the infilled frame.

	A: bare frame triang. force distr**.**	**B[1]: infilled frame** triang. force distr**.**	**C[1]: bare frame** rectang. force distr**.**	**D[1]: bare frame** displ. control
V_y [kN]	280	320	300	300
δ_y [%]	0.50	0.50	0.40	0.35
δ_u [%]	1.00	1.10	0.75	1.5
K_y	7778	8889	10417	11905
K_u	3889	4040	5556	2778
T_y	0.84	0.79	0.73	0.68
T_u	1.19	1.17	1.00	1.41
μ_Δ	2.00	2.20	1.90	4.30
η_u	0.70	0.68	0.71	0.59
α_y [g]	0.11	0.12	0.11	0.10
α_u [g]	0.23	0.27	0.20	0.35

	A	**B**	**C**	**D**
V_y [kN]	280	320	300	300
δ_y [%]	1.00	1.00	0.80	0.70
δ_u [%]	2.00	2.20	1.50	3.00
K_y	3889	4444	5208	5952
K_u	1944	2020	2778	1389
T_y	1.19	1.11	1.03	0.96
T_u	1.69	1.65	1.41	2.00
μ_Δ	2.00	2.20	1.90	4.30
η_u	0.70	0.68	0.71	0.59
α_y [g]	0.16	0.17	0.15	0.14
α_u [g]	0.33	0.38	0.29	0.49

[1] Compare with curves in Figure 3

Table 1+2: Prediction of the ground acceleration required to induces yielding (α_y) and collapse (α_u) applying a displacement-based approach
Table 1: using results of analytical push-over analysis
Table 2: increasing the displacements (by a factor of 2) to account for effects not considered in the analysis

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