EVALUATION OF THE NEED FOR WEAK BEAM-STRONG COLUMN DESIGN IN DUAL FRAME-WALL STRUCTURES

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SUMMARY

According to Capacity Design principles, in multi-storey frames it is considered preferable to enforce the formation of plastic hinges in the beams, in order to spread plasticity throughout the whole structure. However adding a wall to the frame may eventually contribute to spread plasticity even if the hinges in the frame form at the columns.

This possibility was studied by comparing the results of non-linear dynamic analysis of a set of four structures: two frame structures, and two dual frame-wall systems. In each group one structure was designed to develop hinges on the beams and the other to develop hinges on the columns.

Results of the first phase of this work, on structures designed without reserve strengths, indicate that the location of the hinges has influence on the seismic performance of the dual systems and that this conclusion may be due to the adopted design criterion.

INTRODUCTION

Modern codes of practice apply the sound principles of *capacity design* in order to design earthquake resistant structures that are not only economic to build but also to repair after an earthquake. For this purpose a hierarchic formation of plastic hinges is enforced by design, in order to create the most desirable and stable energy dissipating mechanism. In this context, it is widely recognised that the most desirable location for plastic hinges in moment resisting frames is the beams extremities.

As figure 1 shows, if the plastic hinges are located at the beams, a much larger number of plastic hinges must develop to create a collapse mechanism. If plastic hinges do form at both ends of all the columns of the same storey, the load can not be further increased and therefore plasticity can not spread any further. It is rather obvious that the first system (figure 1a) possesses better energy dissipation capacities. For this reason, the codes usually prescribe the *weak beam - strong column mechanism* for ductile frames. To enforce this mechanism it is necessary to ensure that the sum of the moment resisting capacities of the columns at a given node is superior to the sum of the moment resisting capacities of the beams that converge on the same node. However, this is a code prescription difficult to enforce in current design practice, as it is often incompatible with architectural requirements. This is so because columns generally belong to frames developing in two directions in plan and it is necessary to design columns with large dimensions in those directions to meet the above-mentioned requirements in both frames simultaneously.

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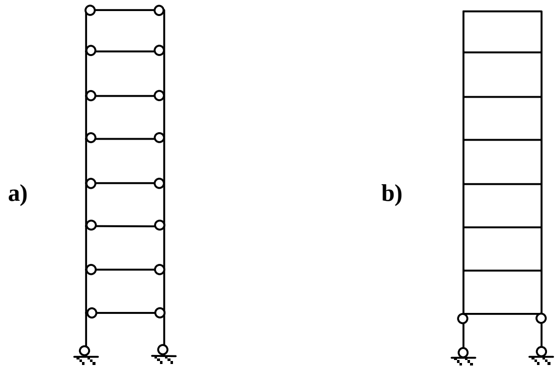


Figure 1 Different types of plastic hinges distribution:
a) plastic hinges develop in beams and base of the frame; b) plastic hinges concentrate in a single storey.

PURPOSE OF THE WORK

As discussed above, it is relevant to investigate possible design alternatives, which could eventually lead to a substantial relaxation of the *weak beam-strong column* requirement.

One of the ways to achieve a better spreading of plasticity along the height of the building, in the case of plastic hinges developing in the columns, would be the addition of a structural wall to the building horizontal resisting system. Figure 2 illustrates this concept by showing that after the columns of a given floor have yielded in both extremities and the wall has yielded at the base, the structural system does not form a mechanism since the deformed shapes of the frame and the wall are not cinematically compatible. Therefore, the horizontal load can be further increased, leading to the spreading of plasticity along the columns throughout the height of the building. It is assumed that the wall is designed according to “Capacity Design” principles, in order to yield just at the base section.

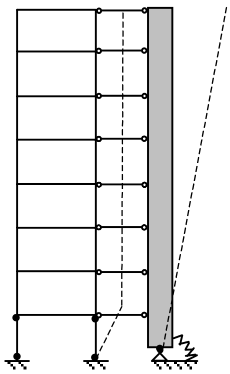


Figure 2 Incompatible deformation mechanism after yielding of an entire floor.

METHODOLOGY

In order to study the effect of adding a structural wall to a frame, four standard reinforced concrete structures were considered. The first two are single frames, considering a *strong beam - weak column* criterion (sb1) and a *weak beam - strong column* criterion (wb1). The second group has a dual frame-wall systems (sb2 and wb2), formed by the same frame and a wall with an elastic rotational spring at the base (k_θ =4675000kNm). The first and second group are modelled as represented in figure 3, imposing the same horizontal displacement to all nodes in each floor. The characteristics of the structures, equal in all cases, are as shown in figure 3.

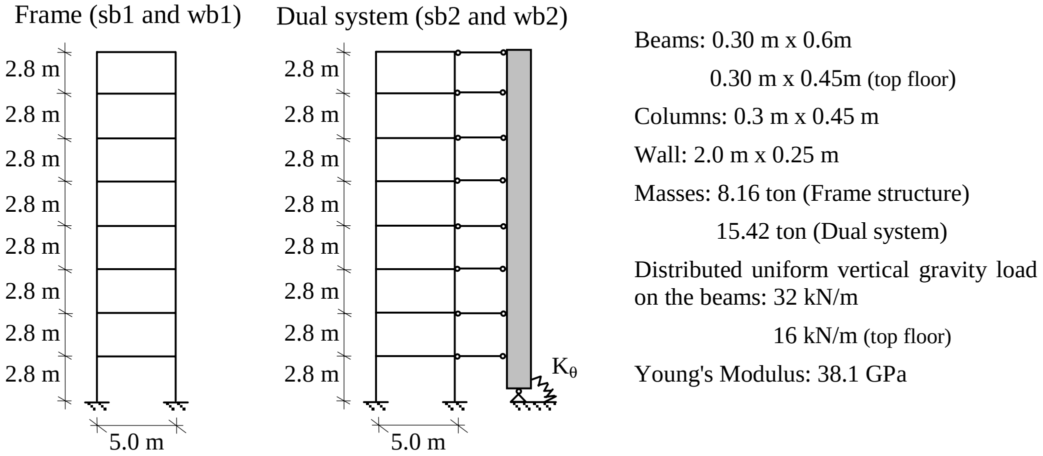


Figure 3 Definition of the analysed structures.

The masses were chosen in such a way that the fundamental frequency is the same for both groups of structures and equal to 1.36 Hz. Two actions were considered: gravity loads and half of the seismic action (corresponding to design using a behaviour factor q=2). The action-effects due to the seismic action were evaluated by elastic multi-modal dynamic analysis. The seismic action was defined by EC8 (CEN, 1994) elastic spectrum for subsoil class B, assuming a peak ground acceleration (a_g) equal to 0.3g.

In the first phase of the study, reported in this paper, the moment capacities at the end sections of beams and columns were assigned the sum of the non-factored action-effects due to the two actions defined. Only two exceptions were considered:

- A situation in which the bending moment was too small due to quasi-symmetrical action-effects caused by the dead load and seismic load. Since rupture will be defined as a function of the inverse of the ultimate curvature (proportional to the yield curvature) of the sections (eq 1), very low in these cases, it would be unrealistic to condition the collapse by these sections. Since in real situations the flexural capacity is never excessively low, due to requirements of minimum reinforcement, this situation was considered by assigning a minimum value to the flexural capacity of all sections of Mrd = 40kN.m, which corresponds to a low amount of flexural reinforcement.

- On top of the columns of the first floor the action-effects are always much smaller than in its vicinity, both below and above, leading to premature collapse of those sections. This is unrealistic, since real columns are never designed with such weak points. To overcome this situation the seismic bending moment of this section was assigned the same value of the bottom one.

The balance between flexural capacities of beams and columns in the joints was maintained by adding, in each node, flexural strength to the beam/column not strengthened according to the above criteria.

In each group of structures two cases were considered: development of plastic hinges in the columns (sb1 and sb2) and in the beams (wb1 and wb2). The two cases are intended to be extreme limits of real design practice. In each case the moment capacities of the elements that were intended not to plastify were multiplied by an overstrength factor of 1.35. In the case of the wall the highest bending moment takes place at the base section, since the spring at the foundation is relatively stiff. Since it is intended that the wall remains elastic above the base hinge, the same flexural capacity was assigned to the moment capacity of all the wall cross sections throughout the height.

The influence of high shear forces in the walls or high axial forces in the columns was not considered.

The above moment capacities in the frame do not correspond to most design situations, in which sets of elements are usually assigned the moment capacities equal to the highest design moment of the group for the sake of standardarization. In fact, the above situation can be considered one possible extreme in terms of design criteria, in which the designer tries to match actions and resistances almost to the limit. The other extreme design criteria, in which the designer gives full priority to standardarization, would lead to beams and columns equal in all floors. The latter design criterion will be studied in a later phase of this work.

Five accelerograms generated from the elastic spectrum of EC8 (CEN, 1994) were used for the nonlinear dynamic analysis of the four structures. The analyses were performed using the Drain-2D program (Kannan & Powel, 1975). To model the structural elements a non-linear stiffness-degrading element was adopted (element type 6). This element is a *one-component model*, which consists of a linear elastic element with non-linear rotational springs at each end. The elastic element is intended to model the elastic deformations, whereas all the non-linear deformations are concentrated in the two end springs. In fact, all plastic deformation effects, including the degrading stiffness effects, are introduced by means of the moment-rotation relationships for the hinge springs. The moment-rotation relationship for each hinge is an extended version of the Takeda model (Takeda *et al.*, 1970). The post yield stiffness for the force displacement relationship of each member was assigned a value of 2.5% of the respective elastic stiffness. The plastic hinge length of all structural elements was assumed equal to one tenth of each member's length. The displacement ductility was assigned a value of 15. This is higher than the ductility generally available in reinforced concrete members, in particular the walls. However, it was intentional in this phase of the study, as it was intended to emphasise only the differences of the non-linear behaviour due to the location of the hinges.

Collapse was defined at all sections by the Park and Ang damage index ($i_{P\&A}$), as follows:

$$i_{P\&A} = \frac{\chi_{max}}{\chi_u} + \beta \frac{E_h}{\chi_u M_y} \tag{1}$$

This index takes into account not only the maximum curvature, χ_{max}, normalised relatively to the ultimate deformation in a monotonic test χ_u, but also the dissipated hysteretic energy, E_h, normalised to the energy dissipated by an elasto-plastic model with a yield model, M_y. The parameter β characterises the contribution of the dissipated hysteretic energy to collapse and was assigned the value of 0.02.

The collapse of the structure was considered to take place when the damage index reached the value 1 at any section of the structure.

RESULTS

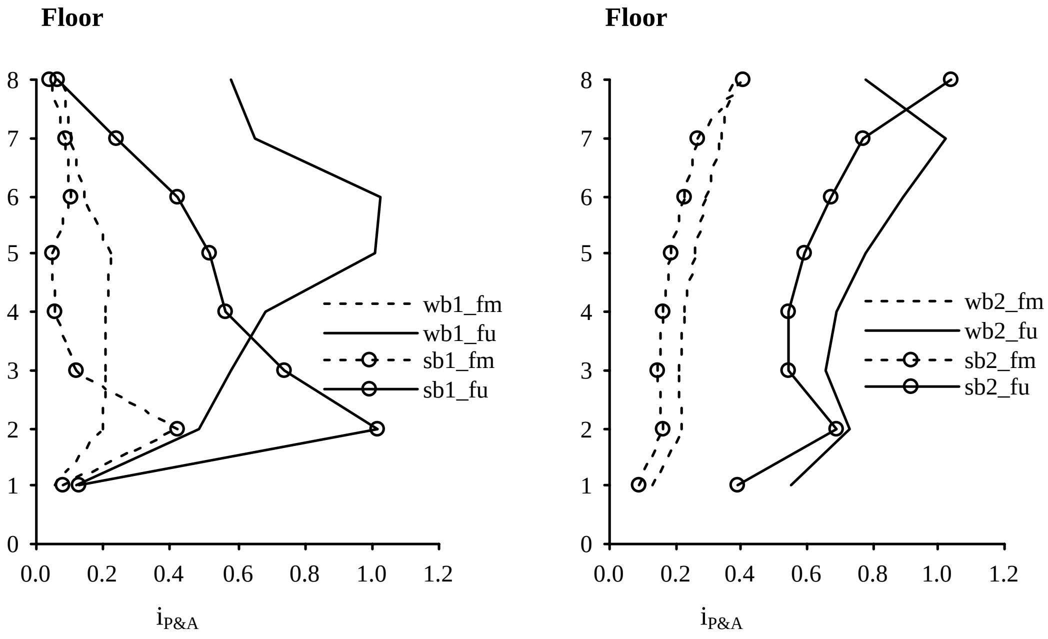
Each structure was subjected to all the accelerograms (Acc.1 to Acc.5) scaled by an intensity factor “f”. In each case the value of “f” was varied until the value corresponding to collapse (f_u) was found, as described by Table 1. For accelerograms 1 and 4 the intensity factor corresponding to first yielding (f_y) is also presented.

Table 1 Intensity factors corresponding to first yielding and collapse.

f_y/f_u	**Acc. 1**	**Acc. 2**	**Acc.3**	**Acc. 4**	**Acc. 5**	**Mean**
wb1	0.23/1.47	1.62	1.58	0.22/1.55	1.40	1.53
sb1	0.28/1.05	1.20	1.05	0.26/0.96	0.98	1.05
wb2	0.23/2.15	2.15	2.10	0.26/2.48	1.90	2.16
sb2	0.27/1.45	1.55	1.60	0.31/1.60	1.30	1.49

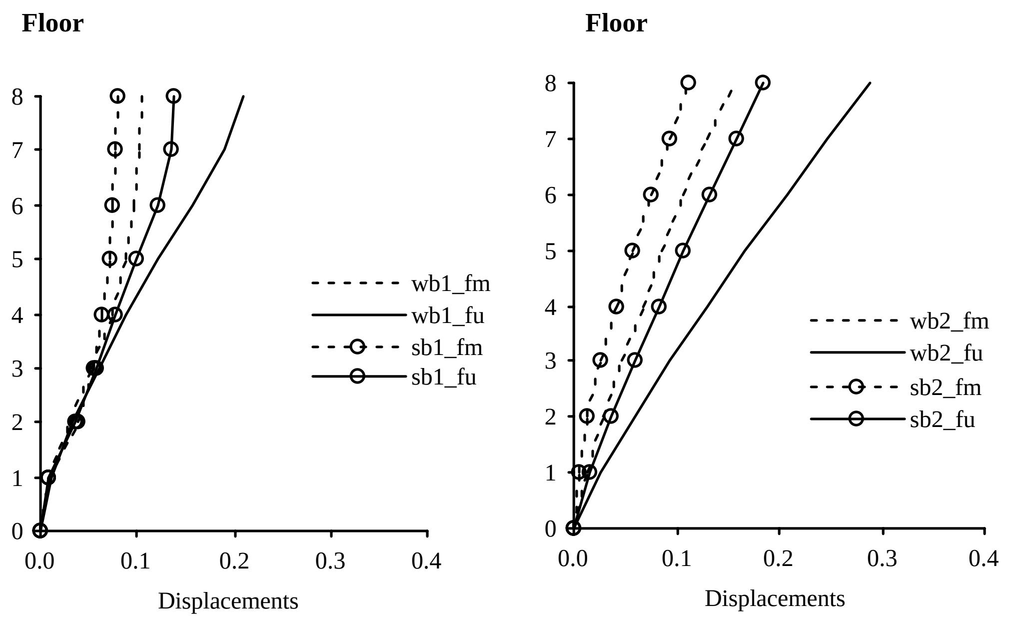
The results show that, for the studied structures, the location of the hinges has an effect on the intensity factor at collapse (f_u) of the mixed structures (wb2 and sb2) similar to the effect on the frame structures (wb1 and sb1). In both cases the location of the hinges in the beams (wb1 and wb2) leads to higher intensity factors at collapse. However a more detailed analysis of the results indicates that these conclusions can not be easily extrapolated.

Figures 4 and 5 reveal the distribution of the damage indices and of the maximum horizontal displacements relatively to the foundation along the height for the four structures at two seismic intensity levels of accelerogram 1: at collapse (f_u) and at the average of the yield and collapse intensities (f_m). No qualitative differences were found with other accelerograms.



a) Frame structure **b) Dual system**

Figure 4 Distribution of the damage indices along the height.



a) Frame structure **b) Dual system**

Figure 5 Distribution of the maximum horizontal displacements relatively to the foundation along the height.

Figure 4 shows that the presence of the wall leads to a more uniform distribution of the damage indices along the height. This can be easily understood by the analysis of the displacements along the height: it indicates that the wall behaves almost as a rigid body with a base hinge, leading to similar interstorey drifts above the second floor. This allows for a much better exploration of the energy dissipating capacity of the frames. Such was an expected result for the frames with hinges formation at the beams. The observation of similar behaviour in the structure with column hinges can be attributed to two factors: 1) the fact that the wall uniformizes interstorey drifts along the height regardless of where plastic hinges develop in the frame (beams or columns), and 2) the design criterion. Since the capacities of the sections match the design action-effects there are almost no excess strengths and all the columns tend to plastify almost simultaneously, a feature confirmed by the results. This derives from the fact that the multi-modal dynamic analysis leads to a distribution of action-effects similar to the one obtained by time history analysis in the elastic range. However, after large incursions in the inelastic range, the distribution of action-effects changes. In the elastic range, the associated distribution of inertia forces exhibits an inverted triangular distribution along the height, while in the inelastic range it tends to a more uniform distribution. This leads to a concentration of the ductility demand at certain levels, as shown in figure 4a. Since the presence of the wall inhibits the concentration of the ductility demand, this effect is not so strong in the dual systems, as shown in figure 4b. It is therefore expected that, if a more realistic design criterion had been used, with different levels of excess strength throughout the structure, the distribution of the ductility demand along the height in the single frame structures would have been different and would also depend on the seismic intensity. This effect would probably have stronger negative consequences (in what concerns spreading the plasticity) if the hinges form in the columns, because the number of hinges necessary to develop a mechanism is smaller.

Figure 4b also shows that collapse of structure sb2 was conditioned by the columns of the upper floor, which had damage indices much higher than the other columns after strong inelastic excursions. This means that if inelasticity had been delayed at this location by providing excess strength, as it is probably the case in most real design situations, an higher intensity factor at collapse could be obtained. In structure wb2 the difference between the damage indices along the height is smaller, therefore improvements due to changing the design criterion are likely to be smaller.

For structure wb2, collapse is reached in some cases by failure of the wall at the base, where its non-linear behaviour was concentrated, and in the other cases when the structure collapses the wall is also close to collapse presenting damage indices slightly below 1. For the dual system with column hinging, the wall damage indices at collapse were about 0.5.

In this study larger than real ductility was available for all structural members. The reality is that the available ductility in the walls may be lower than the one in linear members, such as beams or columns with low axial forces. Therefore, the consideration of this limitation is likely to reduce more the intensity factors in the structures in which the walls exhibited the larger damage indices at collapse, this is, the ones designed for hinge formation at the beams.

The above discussion points to the possibility that with a different design criterion (more uniform distribution of strength capacities) the difference between the intensity factors at collapse (f_u) could widen for the frame structures and reduce for the dual systems.

It should also be mentioned that if second order effects had been accounted for, the f_u values would be smaller in all cases. However, since the structures designed to form plastic hinges in the beams exhibited larger displacements, the reduction in f_u would be larger. Therefore, the differences due to the difference on the location of the hinges would be smaller, both for the frame structures and the dual systems.

CONCLUSIONS

Two frame structures and two dual frame-wall systems were designed without reserve strengths throughout the height of the building and all members were provided with the same large ductility level. In these conditions the results have shown that the formation of the hinges at beams extremities leads to a better seismic performance than if the hinges develop in the columns, both for frame structures and for dual frame-wall systems.

The above conclusion can not be safely extrapolated since the first condition leads to almost simultaneously yielding throughout the height and consequent spreading of plasticity. This would not occur in a more standard design in which the reserve strengths may vary considerably. The consideration of lower limits for the walls ductility would probably affect the conclusion, since the exploration of the ductility of the walls is larger in the case the frame hinges develop on the beams.

The distribution of seismic action-effects along the buildings height changes after large inelastic excursions. The associated distribution of inertia forces, which follows an inverted triangular distribution in the elastic range, tends to a more uniform distribution along the height in the non-linear range. For frame structures this leads to a concentration of the ductility demand at localised regions, regardless of the location of the plastic hinges (beams or columns). This effect is strongly reduced by the presence of the wall, which uniformizes the ductility demand along the height if the wall is allowed to plastify only at the base.

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