

Inelastic seismic response of building structures with flexible diaphragm

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ABSTRACT: The inelastic seismic response of one-story buildings with flexible diaphragm is analyzed. The structural model consists of a floor system supported by seven elements with degrading stiffness properties. The elastic design analysis is carried out by modeling the floor as a rigid or, alternatively, as a flexible beam. The actual non linear response of 330 structures is studied via numerical simulations. Different stiffness and strength distributions in the lateral resisting systems and in the diaphragm are considered. The results show that the deformability of the diaphragms plays an important role only if the lateral-force resisting system has a markedly nonuniform stiffness distribution. Even in such case, however, adopting rigid diaphragm hypothesis leads to a more conservative design.

1 INTRODUCTION

Floors of buildings play an important role in redistributing the horizontal seismic forces among vertical substructures.

Many seismic codes (see I.A.E.E. (1988)) rely on the ability of floors to behave as diaphragms with infinite stiffness in their own plane. This justifies the Rigid Floor modeling (RF), that simplifies the analysis and reduces the computational efforts. Indeed, in case of R/C floors, with or without tiles, the in-plane stiffness (K_{floor}) is usually very high if compared with the translational stiffness of vertical structures (K_{vert}). However, some seismic codes say that the RF hypothesis can not be always retained. For some geometrical and structural configurations the actual force distribution among vertical resistant elements can differ considerably from that obtained with the RF hypothesis. The most important factors in this respect are:

- the ratio between the plan dimensions related to the position of the vertical elements;
- the presence and the position of holes and/or reentrances in the floors;
- the structural system.

Some seismic codes, such as the Italian tentative code CNR-GNDT (1985) and the Eurocode n. 8 (C.E.C. (1988)), as well as the New Zealand Standards (see I.A.E.E. (1988)), contain also provisions for strength verifications of floors.

All the above standpoints are summarized in CNR-

GNDT (1985), which prescribes that "for the analysis of buildings an elastic structural model, made of structural elements connected by in-plane rigid diaphragms, can be generally adopted. The adoption of such model is however conditioned by a rational evaluation of the actual stiffness of floors (taking into account eventual openings and interruptions) and of their ability of redistributing the seismic actions among the various elements, while remaining in an essentially elastic state".

In spite of the consideration given to this problem by seismic codes, very few researches have been made until now. Studies on the effects of the floor flexibility have been carried out mostly in the linear range (see De Matteo et al. (1988), Pagano (1990), Button et al. (1984), Roper and Iding (1984)). Some general results are given in the first two works, where two parametric investigations are described. The main conclusion of De Matteo et al. (1988) was that the validity of the RF hypothesis depends essentially on the ratio $K_{\text{floor}}/K_{\text{vert}}$, between the in-plane floor stiffness K_{floor} and the vertical element stiffness K_{vert} , and on the distribution of such stiffness. Pagano (1990) examined the stress state of the floor slab. He highlighted that in some cases special reinforcements are required.

All the above considerations emphasize the need of studying the effects of floor flexibility on the non linear behavior of structures subjected to strong seismic actions. At this aim in the present work the inelastic dynamic response of a large number of

simple symmetric structures is investigated in detail.

2 MODELING

The reference structural system is a single-story three-dimensional frame, with two by six bays of 5 m span.

The floor is idealized as a unique beam, modeled by elastic or elasto-plastic beam elements. Each vertical structure is idealized as a stiffness degrading beam element, fixed at its base and connected by hinges to the floor beam, so that no moment is transmitted to the floor (see fig. 1). The vertical elements can therefore be thought of as any vertical structural system (frame or wall), simply characterized by a given overall stiffness and strength. The unitary mass is equal to 1 t/m^2 and has been concentrated at the nodes of the floor. A stiffness degrading model has been adopted, to describe effectively the behavior of R/C elements subjected to cyclic loads.

The initial stiffness of the vertical elements in the standard case, is evaluated as the translational stiffness of frames with 0.40 by 0.40 squared section columns. The moment of inertia of the vertical elements is therefore equal to $I' = 0.0256 \text{ m}^4$.

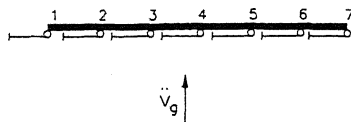


Fig. 1 Structural idealization

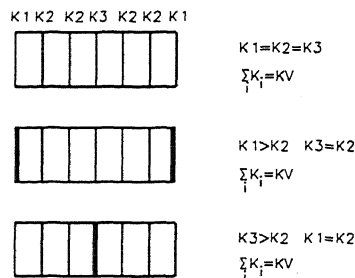


Fig. 2 Stiffness distribution of the vertical elements

The stiffness distribution of vertical structures has been varied, while keeping the total stiffness constant. Besides the uniform distribution, two other distributions have been considered (see fig. 2): one with two very stiff elements at the opposite edges and the other with one very stiff element at the middle. The first situation simulates the presence of end walls, the second situation refers to the presence of a central core.

To evaluate the flexural stiffness of the floor, refer-

ence is made to a tile lintel floor, with a 0.05 m thick R/C upper slab and 0.50 m by 0.30 m border beams. By considering a maximum floor width of 10 m, a moment of inertia $J1 = 11.6689 \text{ m}^4$ is obtained, which is about 450 times greater than the moment of inertia of the standard vertical elements.

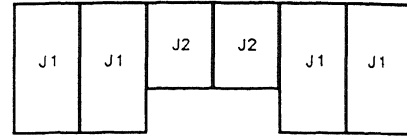


Fig. 3 Floor shape

The stiffness of the floor is varied by inserting a reentrance in the two middle bays, whose moment of inertia will be called $J2$ (see fig. 3).

3 ANALYSIS

Each structure is completely defined in terms of geometry, stiffness and mass. The strengths of the vertical resistant elements of each structure are decided on the base of a response spectrum analysis. The seismic action is represented by the elastic response spectrum for site A of EC8 (C.E.C. 1988) shown in fig. 4. The structural coefficient is taken equal to 3. Both the FF (Flexible Floor) and the RF (Rigid Floor) hypotheses are adopted alternatively in the elastic analyses.

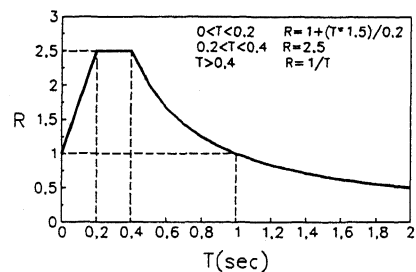


Fig. 4 Elastic 5% damping response spectrum

Once the element strengths are defined, a series of non linear analyses are carried out in order to evaluate the structural response. Each structure is subjected to ten artificial accelerograms consistent with the response spectrum assumed for the elastic design analysis. In order to interpret the non linear behavior of the structures, the maximum displacement ductilities of vertical elements are assumed as response indices. They are calculated as averages of the maximum ductility demands in the 10 step-by-step analyses.

The damping ratio is taken equal to 5% in the elastic analyses and to 2% in the inelastic analyses, where a stiffness proportional damping matrix is assumed. The integration time step is taken equal to 0.005 secs.

4 PARAMETRIC INVESTIGATION

The following parameters have been considered in this study:

- 1) Distribution of stiffness among vertical elements. If K_1 is the stiffness of the two external elements, K_3 is the stiffness of the middle element and K_2 is the stiffness of the remaining elements (see fig. 2), the following cases have been examined:

| | |
|-----------------|-----------------|
| $K_1 = K_2$ | $K_3 = K_2$ |
| $K_1 = 20 K_2$ | $K_3 = K_2$ |
| $K_1 = 100 K_2$ | $K_3 = K_2$ |
| $K_1 = K_2$ | $K_3 = 20 K_2$ |
| $K_1 = K_2$ | $K_3 = 100 K_2$ |
- 2) Distribution of the in-plane stiffness of the floor. If J_2 is the moment of inertia of the two middle bays and J_1 is the moment of inertia of the remaining bays (see fig. 3), the following three cases have been considered:

$$J_2 = J_1, \quad J_2 = J_1/20, \quad J_2 = J_1/100$$
- 3) Total translational stiffness of the vertical structures, KV . The following cases have been considered:

$$KV = K, \quad KV = K/4, \quad KV = K/9$$
- 4) Post-yielding stiffness i , expressed as a percentage of the initial elastic stiffness. The following values have been considered:

$$i = 1\%, \quad i = 3\%, \quad i = 10\%$$
- 5) Strength of the floor, Sf . The following three values have been adopted:

$$Sf = \text{infinite}, \quad Sf = R, \quad Sf = R/2$$

R has been evaluated with reference to a 50 cm² reinforcement in the border beams, and differs from bay to bay, according to the floor width. The steel reinforcement has been fixed according to the strength for bending usually required by vertical loads. Seismic floor stresses have not been considered in designing floor strength since it is not requested by many codes.
- 6) Floor modeling in design analysis. The two following assumptions have been made:
 - RF (Rigid Floor hypothesis)
 - FF (Flexible Floor hypothesis).

In the second case the actual bending and shear stiffness of the floor is assumed.

While all possible combinations of the parameters specified in points 1, 2 and 6 have been considered, the same does not hold for the parameters specified in

3, 4, 5. In tab. 1 there are listed the cases considered for these three parameters.

| CASE | POST-YIEL. STIFFNESS i | TOTAL STIFFNESS KV | FLOOR STRENGTH Sf |
|------|--------------------------------|----------------------------|---------------------------|
| 1 | 1% | K | infinite |
| 2 | 3% | K | infinite |
| 3 | 10% | K | infinite |
| 4 | 3% | $K/4$ | infinite |
| 5 | 3% | $K/9$ | infinite |
| 6 | 3% | K | R |
| 7 | 3% | K | $R/2$ |
| 8 | 3% | $K/4$ | R |
| 9 | 3% | $K/4$ | $R/2$ |
| 10 | 3% | $K/9$ | R |
| 11 | 3% | $K/9$ | $R/2$ |

Tab. 1

For each of the cases listed in tab. 1, 30 structures, are obtained by varying the stiffness distribution of the vertical elements and of the floor and by considering the RF and FF hypothesis in the design analysis. A total amount of $11 \times 30 = 330$ structures have been examined, each one subjected to 10 different accelerograms, for a total number of 3300 non linear analyses.

5 RESULTS

In the diagrams of figs. 5, 6, 7, there are shown the strengths of the vertical elements, assumed equal to the elastic stresses divided by three, for the two hypotheses on the floor stiffness. The results relevant to the RF (Rigid Floor) hypothesis coincides with those relevant to the $J_1 = J_2$ case. The 11 cases listed in tab. 1 are reduced to only the 3 cases relevant to the KV values, since the post-elastic stiffness i and the floor strength Sf do not affect the elastic analyses. In the cases of uniform stiffness distribution in the vertical elements ($K_1 = K_2 = K_3$), the floor behaves like a rigid diaphragm, even for strong reduction of the floor width in the middle ($J_2 = J_1/100$). The seismic forces are therefore distributed uniformly among the vertical elements.

In the other cases the RF hypothesis determines a strength distribution in the vertical elements similar to their stiffness distribution. The strength distribution varies according to the variation of the floor flexibility, when the FF hypothesis is adopted. When increasing the floor flexibility, a strength reduction in the stiffer vertical elements and an increase in the more flexible ones are obtained. A reduction of the

overall strength of the vertical elements can be noted, when passing from $KV=K$ to $KV=K/9$. This is due to the elongation of the fundamental period, which is equal to 0.234 secs for $KV=K$ and RF hypothesis, combined with the reduction of the spectral ordinates for $T > 0.4$ secs..

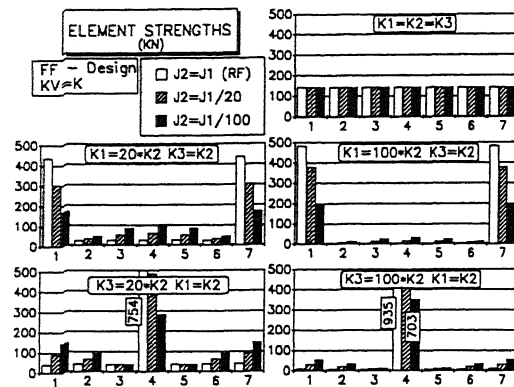


Fig. 5 Strength distribution for $KV=K$.

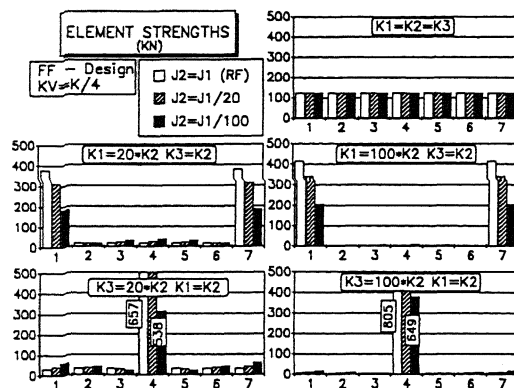


Fig. 6 Strength distribution for $KV=K/4$.

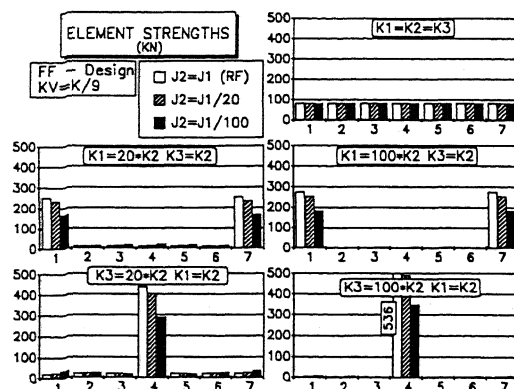


Fig. 7 Strength distribution for $KV=K/9$.

The diagrams of figs. 8, 9, are relevant to the results of some inelastic analyses. The response index adopted is the ductility demand in the vertical structures m . The most interesting consideration is relevant to the markedly different responses that the two design hypotheses on the floor stiffness produce in some cases. In particular the RF hypothesis leads to a low ductility demand to the "strong" elements, while a high ductility demand occurs in the "weak" elements. The contrary happens for the FF hypothesis, which leads to an overdesign of the "weak" elements and an underdesign of the "strong" elements.

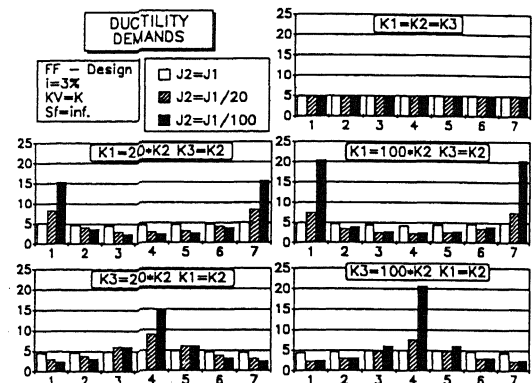


Fig. 8 Ductility demand for FF design hypothesis.

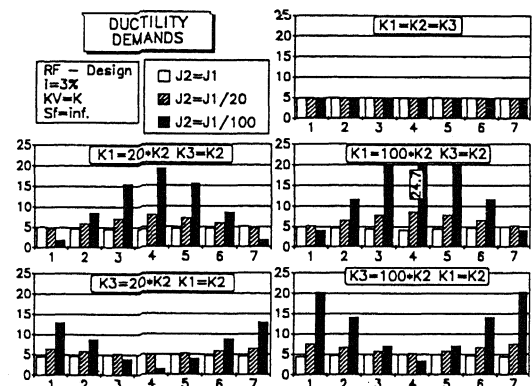


Fig. 9 Ductility demand for RF design hypothesis.

This result can be explained by considering the variation of the stiffness ratio $K_{\text{floor}}/K_{\text{vert}}$ during the earthquake. When the vertical elements are stressed beyond the yield limit, a sudden reduction of the stiffness of the vertical elements occurs, so that the actual stiffness ratio $K_{\text{floor}}/K_{\text{vert}}$ increases. The behavior of the floor becomes more similar to that of a rigid floor, and the relative displacements among the different vertical elements decrease. Consequently a displacement distribution different from that obtained by the

elastic analysis occurs. This results in a high ductility demand in the stiffer elements.

Since the stiffer elements are also the main elements with regard to the overall structural behavior, their ductility demands can be considered the most significant response indices. For this reason in the following diagrams only their values will be examined.

In figs. 10,11 the influence of the total translational stiffness of the vertical structures (KV) can be seen. Its reduction leads to a general decrease of the ductility demand, as well as it was observed for the elastic stresses.

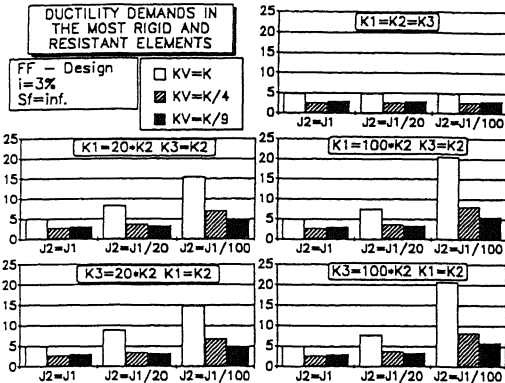


Fig. 10 Ductility demand for various KV - FF design.

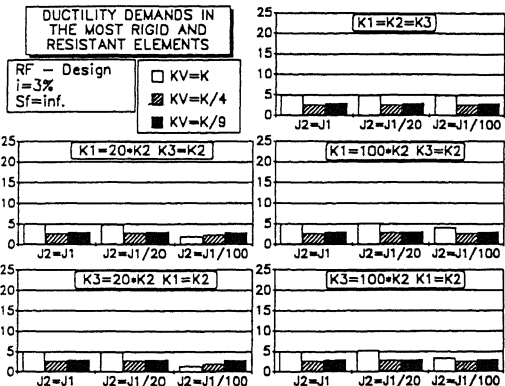


Fig. 11 Ductility demand for various KV - RF design.

In the inelastic response this effect can be ascribed to the elongation of the "effective" period. The ductility demand decrease is more marked when the stiffness varies from K to K/4 rather than from K/4 to K/9. In fact the strength design for K and K/4 does not differ substantially, as can be seen in figs. 5 and 6. On the contrary the stiffness reduction that characterizes the non linear response, produces the elongation of the equivalent period. For KV=K the equivalent period will still lie in the constant ordinate branch

of the spectrum, while for KV=K/4 the effective period will be shifted in the decaying branch, and a strong reduction of the seismic effects ensues. When passing from K/4 to K/9, the variations of the equivalent period in the non linear response occur in the decaying branch of the spectrum, so that less important variations of the ductility demand are obtained.

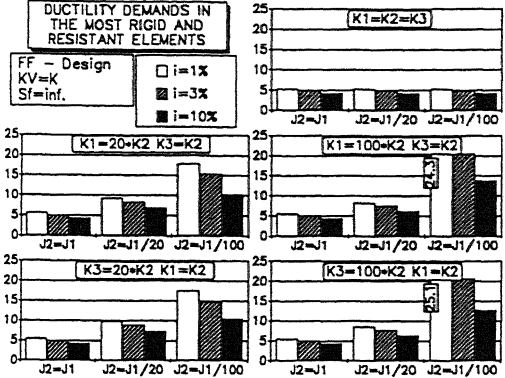


Fig. 12 Ductility demand for various i - FF design.

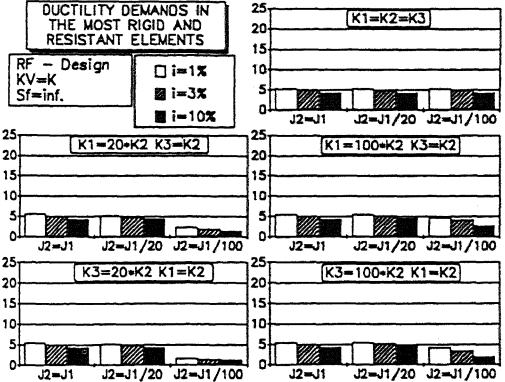


Fig. 13 Ductility demand for various i - RF design.

In figs. 12, 13 the influence of the post-yielding stiffness i can be observed. The results show that the variations of the ductility demand depend only on the value of the maximum displacement d_{max} , which usually decreases when increasing i. Therefore the increase of the post-yielding stiffness produces a general decrease of the ductility demand in the vertical elements.

In figs. 14,15 there is shown the influence of the floor strength (Sf). If finite values are assumed, yielding occurs sometimes also in the floor. This happens in the weak zone of the floor for a non uniform distribution of the stiffness of the vertical elements. The increase of floor deformability due to yielding produces a double effect: an increase of the effective

period and a variation of the displacement distribution, with a relative decrease of the displacements of the stiffer elements. When the floor yields, a reduction of m in the strong elements and different response in the weak elements can be observed, according to the prevailing of one effect or the other.

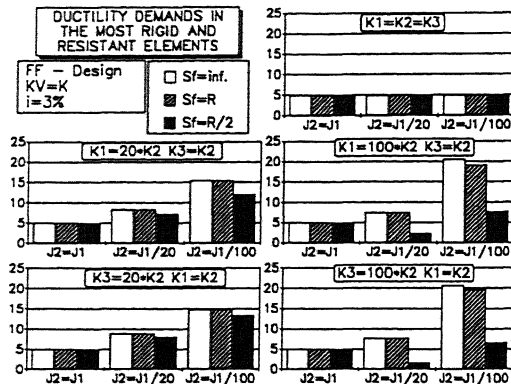


Fig. 14 Ductility demand for various S_f - FF design.

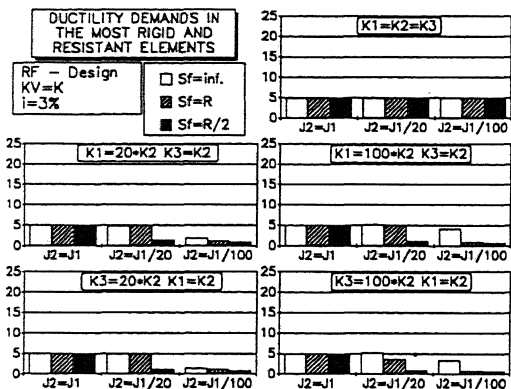


Fig. 15 Ductility demand for various S_f - RF design.

A last observation is relevant to the stresses and the ductility demands in the floor. No diagram is provided in this paper, but the results of the analyses have shown that higher stresses occur when a RF hypothesis is adopted in the design analysis. If the floor slab and the floor beams are not adequately designed, excessive ductility can be required to the floor structure.

6 CONCLUSION

The results of the present study show that the deformability of the floor does not produce important effects on the linear and non linear structural response when the distribution of stiffness of vertical elements

is uniform. On the contrary important effects on the linear and non linear response occur in case of a considerable deformability of the floor, e.g. due to reentrances, and of strong differences of stiffness among the resistant elements. However a strength design based on elastic analyses that takes into account the actual flexibility of the floor leads to a worse non linear response of the structure, since a very high ductility demand in the more rigid and resistant vertical elements occurs.

Such a conclusion is contrasting with the specifications of some modern seismic codes. They require that the actual flexibility of the floor should be considered in the design analysis, when the floor can not be considered infinitely stiff. If this provision is adopted, the more resistant and stiff element, which are obviously the most important in the structure, would be underdesigned and subjected, under strong earthquakes, to unacceptable ductility demands. For this reason it seems that the rigid floor (RF) hypothesis is more conservative in terms of global safety, provided that the less rigid and resistant elements are given an adequate ductility capacity and the strength of the floor structure is designed according to the seismic analysis.

7 REFERENCES

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