Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.V)

7-7-3

INFLUENCE OF VERTICAL IRREGULARITIES ON SEISMIC RESPONSE OF BUILDINGS

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SUMMARY

The objective of this paper is to study the seismic behaviour of reinforced concrete buildings exhibiting vertical irregularities. 16 storey-high buildings are thoroughly studied for three different horizontal layouts and for five vertical configurations. Simplified analysis will be extended to similar buildings 12 and 20 storey-high.

The building were idealized as a set of plane moment resisting frames connected to shear walls by rigid diaphrams. Non-linear behaviour for both the frames and shear walls were considered for different behaviour coefficients. A step-by-step integration of the equations of motion for 10 different acceleration time series was performed and the results were interpreted in statistical terms. Preliminary conclusions are referred for a few cases.

INTRODUCTION

In past earthquakes damage to buildings have proved to depend quite considerably on structural descontinuities namely on vertical irregularities. In order to quantity the effect of these irregularities in the seismic response and be able to recommend design procedures, a set of buildings exhibiting this type of irregularities was studied. A regular vertical configuration served as comparison.

The basic structural layout of buildings under analysis was obtained from the current practice in modern building construction in Lisbon: 12, 16 and 20 storey-high; plan configurations with moment resisting frames plus shear walls or cores; vertical configurations with setbacks and sudden interruption of shear walls.

The plan configurations selected were already the object of a linear - analysis study (Ref. 1), made in order to compare the new Portuguese Seismic Code provisions (Refs. 2,3) for "medium" and "good" ductility.

This paper develops a non-linear model for plane moment resisting frames connected to shear walls by rigid diaphrams and applies it to a 16 storey-building with one central core and 5 different vertical configurations.

DESCRIPTION OF BUILDINGS

The buildings under analysis are essencially made of cellules of 4x5m, displayed as shown in Fig 1. The geometric characteristics of resisting members are presented in Table 1. Slabs are 15cm thick rigid diaphrams. The initial modulus of elasticity is 30GPa, Poisson ratio 0.20, and the vertical load 8980N/m^2

Table 1 - Geometric characteristics of members

STOREY	COLUNMS (hxb)										SHEAR WALL OR CORE					BEAMS
HIGH	0-2	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	0-4	4-8	8-12	12-16	16-20	(hxb)
12	60x40	60x40	60x30	50x30	40x30	30x30	-	-	-	-	25	20	20	-	-	60x20
16	80x40	80x40	60x40	60x40	60x30	50x30	40x30	30x30	-	-	30	25	20	20	-	60x20
20	100x40	100x40	80x40	80x40	60x40	60x40	60x30	50x30	40x30	30x30	35	30	25	20	20	60x20

The three plan configurations are referred as NC - for association with central core, 2NL - for association with 2 lateral core, and PAL -for association with 2 lateral shear walls. The following number stands for building storey high. The five vertical configurations are referred as P - for interruption of shear walls or core without setback, p - for interruption of moment resisting frames with setback. The number that follows each letter stands for storey number at which that particular member. As example, NC16P12p8 is a 16 storey high building with one central core interrupted at level 12, and with a setback at level 8, Fig.1(e).

Table 2 shows the first two lower frequencies of vibration for each one of the buildings selected for analysis.

Table 2 - Frequencies of vibration

TYPE FREQ.	NC16 P16p16	NC16 P12p16	NC16 P16p12	NC16 P16p8	NC16 P12p8	2NL16 P16p16	2NL16 P12p16	2NL16 P16p12	2NL16 P16p8	2N 16 P12p8	PAL16 P16p16	PAL16 P12p16	PAL16 P16p12	PAL16 P16p8	PAL16 P12p8
f_1	0.96	0.93	1.06	1.05	1.20	1.08	0.99	1.19	1.21	1.14	0.78	0.77	0.87	0.87	1.02
f_2	3.83	2.31	3.91	4.09	2.03	4.87	2.18	5.20	5.30	2.00	2.58	1.69	2.70	2.81	1.82

A set of 10 acceleration time series was used to compute the response. These time series correspond to a stochastic process representing the seismic action of type I, soil type 2, zone A of the Portuguese Seismic Code (Ref.2) with an average of 0.175g peak ground acceleration, 15sec duration and a main frequency content in the 2-5Hz bandwidth, Fig.2.

STRUCTURAL NON-LINEAR BEHAVIOUR

The building is modeled as an association of two members (a shear-beam and a shear wall) connected by rigid links at each storey level, Fig.3. The shear beam represents the moment-resisting frames and the shear wall is modelled by a beam element with two degree of freedom, $\underline{\theta}$ and $\underline{\delta}$, the rotations and horizontal translations at each storey level, respectively. The global stiffness matrix $\underline{K}$ referred to horizontal displacements is obtained by

$$\underline{K} = \underline{K}_f + \underline{K}_{sw}$$

where K_f is the stiffness matrix of shear beam (frame) and $\underline{K}_{sw}$ is the stiffness matrix of shear wall condensed to horizontal degrees of freedom. A diagonal mass matrix and a damping ratio of 2% for each individual mode is assumed. The equation of motion was expressed in terms of horizontal degrees of freedom and solved by a step-by-step procedure. The Newmark constant acceleration method was used to integrate the equations of motion, with $\underline{\beta} = 0.3086$ and $\underline{\delta} = 0.6111$ to introduce numerical damping in the higher modes. At each time the horizontal displacements are transformed into the member degrees of freedom. The non-linear behaviour is analysed member by member: the moment resisting frames considering a shear force - distorsion relation and the shear wall a moment-curvature relation. The non-linear effect at each member is computed from the ficticious forces which are transported to the nodes of the structural members and transformed into horizontal equivalent forces at storey levels. In order to assure compatibility and equilibrium, iterations are performed at each step before proceeding to a new step, until sufficient convergence is obtained.

The non-linear behaviour is modeled at each structural member by a modified Takeda tri-linear model with degrading stiffness and resistence, and pinching, as referred in Fig.4 (Ref. 4). This model is controlled by a set of the following parameters: a) craking; b)yielding; c) initial stiffness; d) stiffness after craking; e) stiffness after yielding; f) degrading stiffness; g) pintching effect; h) degrading of resistence.

Parameters a) through f) were fixed for each structure and were kept constant during the entire response. Parameter f), g) and h) depend on number of cycles and on the ductility of members (Ref. 5). In the present study the same values for craking and yielding were adopted leading to a bilinear model. The yielding value, one of the controlling parameters, was dependent on the behaviour coefficient; stiffness after yielding was 5% of the elastic stiffness. The buildings were designed according to the methodology presented in Ref. 6.

Figure 5 presents an output of the response for linear and non-linear analysis of a typical building in terms of top-storey displacements and overall moments at the base. The influence of separating non-linear behaviour in frame and shear wall is clearly observed. (Note that 2NL16P16p16 is the regular building).

The principal results are summarized in Table 3 which presents the values of the required ductility for different combinations of the values of the behaviour coefficient for the frame and the shear wall. This required ductility value, which is separately presented for the frame and the shear wall, is the maximum value of the ductility observed at the different storeys.

Figures 6 and 7 show the variation of maximum ductility demand with the frame behaviour coefficient, in the shear wall and in the frame, respectivelly. Fig.8 presents the distribution of ductility demand in height, for two cases.

FINAL COMMENTS

Even though the subject matter deserves further study, the following comments can be made this far (the complete results will be published in Ref. 7):

- The ductility demand distributions are irregular in shear walls but fairly regular in the frames, except for storeys imediatelly above a discontinuity were there is a significative increase in the frame ductility demand.
- If there are descontinuities in the frame structure, the required ductility in the shear walls markedly increase.
- The values of the required ductility in the shear walls show some decrease with increasing values for the behaviour coefficient in the frame; it should be noted, however, that for high value of those behaviour constant, the required ductility in the shear walls is almost constant.
- The values of the required ductility in the frames is pratically independent of the value of the behaviour coefficient for the shear wall.

- It was found out that ductility demands in the frame and shear wall are almost the same for regular buildings. For irregular buildings, the ductility demand can be nearly twice the ductility demand for regular building. In general, if the irregularity occurs in the frame structure, the increase is observed in the shear wall; if the irregularity occurs in the shear wall, the increase in ductility demand is observed in the frame.

This paper will be extended in the near future to the other plan configurations of Fig.1 and to 12 and 20 storey-high. The same methodology will be used to derive building vulnerability.

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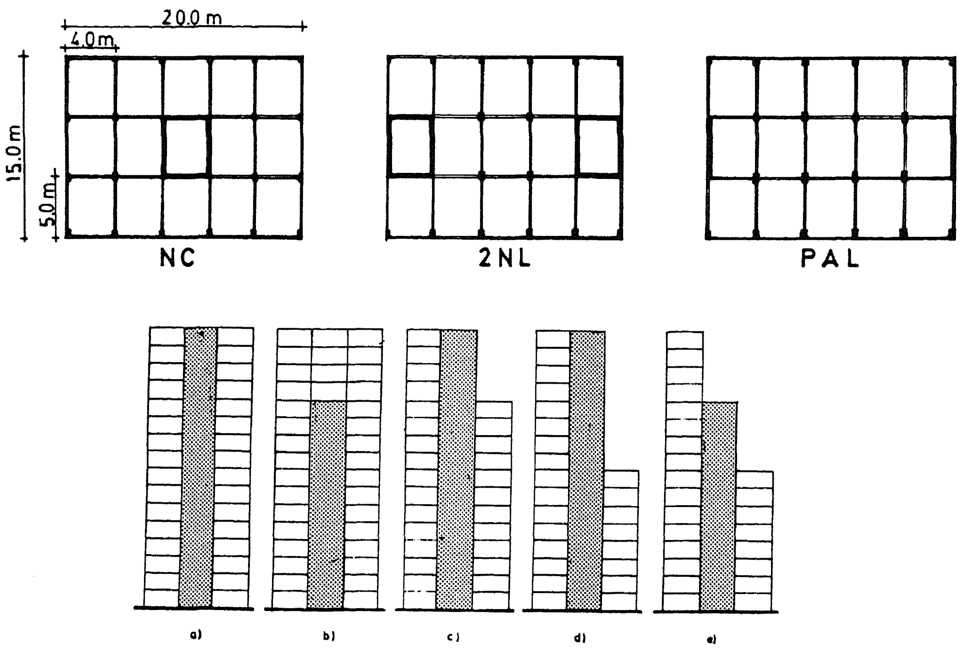

Fig. 1 - Geometrical configuration of buildings under study

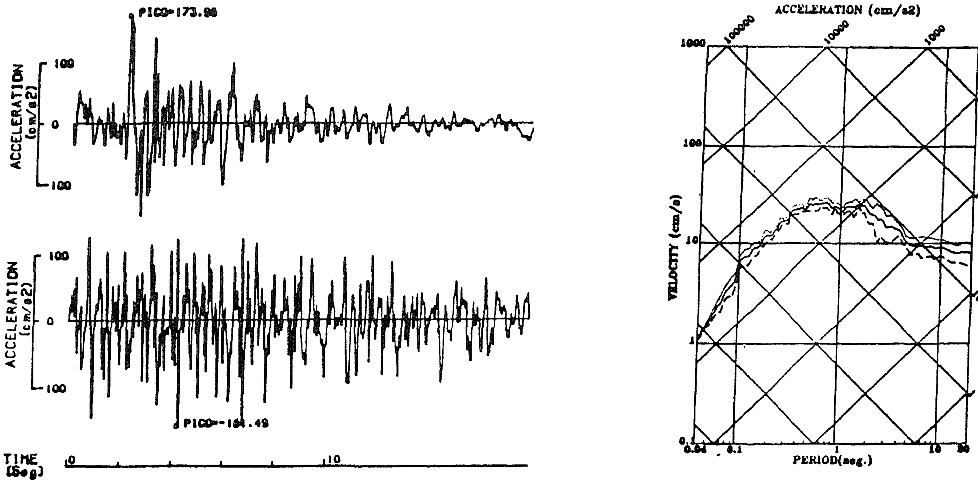

Fig. 2 - Input ground motion and mean response spectra

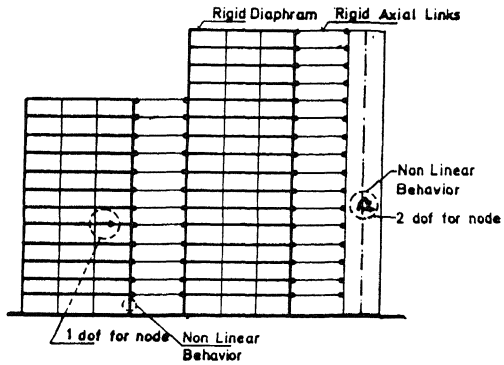

Fig. 3- Skematic representacion of model

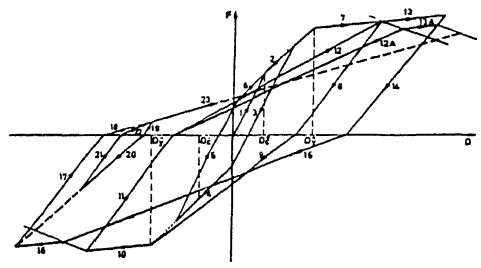

Fig. 4 - Description of histeretical model

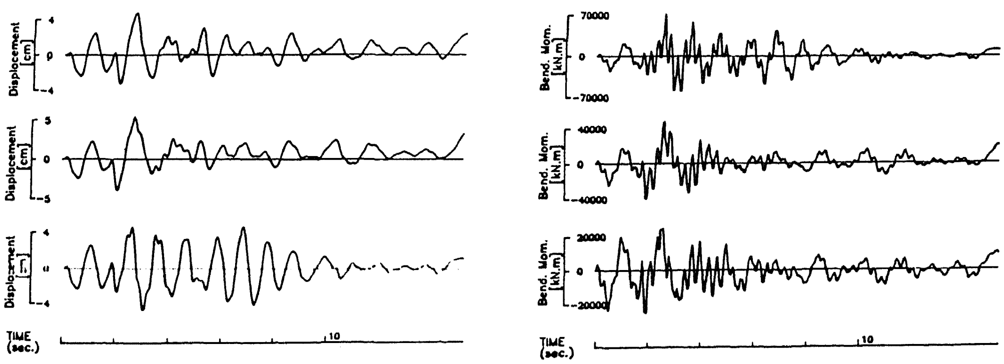

Fig. 5 - Typical time-histories of displacement at the top (left) and bending moment at the base (right). Upper plot-linear response. Nonlinear response central plot: behaviour coeff. for frame = 3 and for shear-wall = 1,5; lower plot: behaviour coeff. for frame = 1.5 and for shear-wall = 2

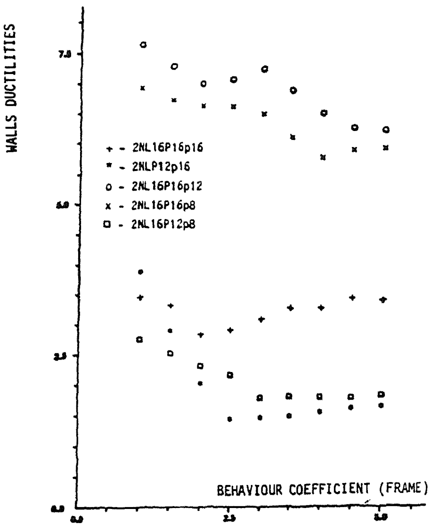

Fig. 6 - Variation of the maximum ductility demand in the shear wall with the frame behaviour coefficient.

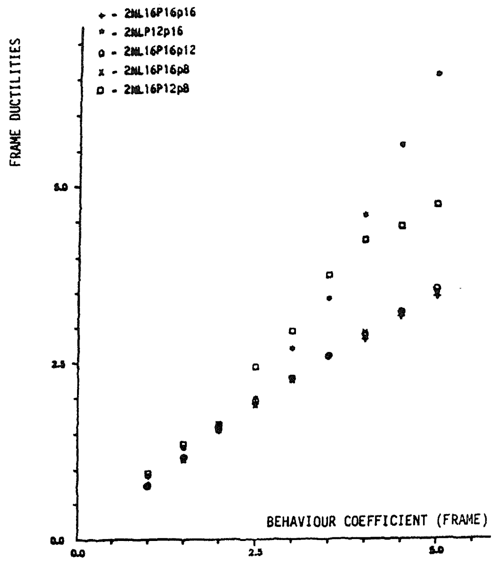

Fig. 7 - Variation of the maximum ductility demand in the frame with the frame behaviour coefficient.

Table 3 Maximum ductility values in the frame and shear wall for the dif. combinations.

	FRAME			SHEAR WALL			
μ_F \ μ_{SW}	1.0	1.5	2.0	1.0	1.5	2.0	
1.0	0.9	0.9	0.8	1.0	2.0	3.5	2NL16P16p16
1.5	1.3	1.2	1.2	0.9	1.9	3.3	
2.0	1.6	1.6	1.5	0.9	1.8	2.8	
2.5	1.9	1.9	1.9	0.9	2.0	2.9	
3.0	2.2	2.2	2.3	0.9	2.0	3.1	
3.5	2.6	2.6	2.6	0.9	2.4	3.3	
4.0	3.1	3.0	2.8	0.9	1.9	3.3	
4.5	3.6	3.4	3.3	0.9	1.8	3.4	
5.0	4.1	3.9	3.4	0.8	1.8	3.4	
1.0	1.0	0.9	0.9	0.9	2.4	3.9	2NL16P12p16
1.5	1.4	1.3	1.3	0.8	1.3	2.9	
2.0	1.8	1.7	1.6	0.8	1.1	2.0	
2.5	2.1	2.1	2.0	0.7	1.1	1.4	
3.0	2.7	2.7	2.7	0.7	1.0	1.5	
3.5	3.7	3.7	3.4	0.7	1.0	1.5	
4.0	5.0	5.0	4.6	0.7	1.0	1.5	
4.5	6.1	6.0	5.6	0.7	1.1	1.6	
5.0	7.3	7.2	6.6	0.7	1.1	1.6	
1.0	0.9	0.8	0.8	1.0	3.1	7.6	2NL16P16p12
1.5	1.2	1.2	1.2	0.9	2.9	7.3	
2.0	1.5	1.5	1.6	0.9	3.0	7.0	
2.5	1.9	1.8	2.0	0.9	3.1	7.1	
3.0	2.3	2.2	2.3	1.0	3.1	7.2	
3.5	2.7	2.5	2.6	1.0	3.0	6.9	
4.0	3.1	2.9	2.9	1.0	3.1	6.5	
4.5	3.4	3.3	3.2	1.0	2.7	6.2	
5.0	3.7	3.6	3.6	1.0	2.7	6.2	
1.0	0.9	0.8	0.8	1.0	2.7	6.9	2NL16P16p8
1.5	1.3	1.2	1.1	0.9	2.8	6.7	
2.0	1.6	1.5	1.5	0.9	2.9	6.6	
2.5	2.0	1.9	1.9	0.9	2.9	6.6	
3.0	2.4	2.3	2.2	0.9	2.7	6.5	
3.5	2.7	2.7	2.6	0.9	2.5	6.1	
4.0	3.1	3.0	2.9	0.9	2.6	5.8	
4.5	3.5	3.3	3.2	0.9	2.4	5.9	
5.0	3.9	3.8	3.5	0.9	2.4	5.9	
1.0	1.0	1.0	0.9	1.0	1.6	2.8	2NL16P12p8
1.5	1.4	1.3	1.4	0.8	1.4	2.5	
2.0	1.9	1.6	1.6	0.8	1.3	2.3	
2.5	2.6	2.6	2.4	0.8	1.3	2.2	
3.0	3.1	3.2	2.9	0.8	1.3	1.9	
3.5	4.1	4.1	3.8	0.8	1.2	1.8	
4.0	4.3	4.4	4.2	0.7	1.2	1.8	
4.5	4.7	4.6	4.4	0.7	1.2	1.8	
5.0	5.2	5.0	4.7	0.8	1.3	1.8	

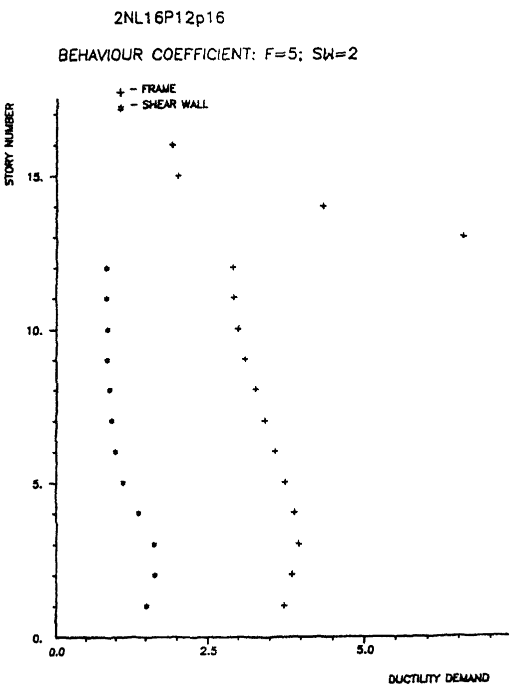

Fig. 8 - Distribution of ductility demand in height for frame and shear wall.