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Seismic actions on the different floors of R/C buildings

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ABSTRACT: In this paper the problem of defining the seismic actions at the various floors of reinforced concrete buildings, in order to state the design criteria for the verification of "non structural" elements and "appendages", is dealt. Data recorded on buildings during seismic events are analyzed and elaborated; in particular, values of the maximum floor acceleration and of the base motion amplification have been analyzed. A significant over reistence has been noticed and discussed. The obtained data allow to determine realistic values of the induced actions and to formulate criteria for the prediction of the response.

1 INTRODUCTION

What in a building is non strictly resistant structure but only connecting element like exterior walls or every kind of covering, or simply supported element such as functional partitions, pieces of furniture, piping of different equipment, what is not considered in modeling the elastic skeleton of the building is usually not engineered. Nevertheless such stuff often determines the global quality of a building and its vulnerability is the most important factor in the losses caused by the earthquake.
In addition they are often conditioning the structural behaviour or strongly conditioned by it.
Unfortunately it is really hard to introduce into the structural model the mechanical characteristics of the elements connected to the structure, first of all because of the uncertainties related to their evaluation; the dynamic global response is so not correctly computed, and the acceleration transmitted by the structure to the supported elements may be not realistically predicted.
The consequence of this difficulty is made evident by the damages that the earthquake, even if not so strong, produces to buildings and their equipment.
If the analytical procedures are not completely able to describe the dynamic behaviour of the whole: structure and its involved non-structural elements, some indications can be got from the data recorded during past earthquakes by instruments placed inside the buildings.
Many records obtained at different floors of buildings during the earthquakes show much stronger accelerations than those derived by the prescriptions of the code. It is evident that in such case the structure has not attained the foreseen yielding level: its behaviour remained perhaps in the elastic phase and the response reached higher values.
The global ductility factor, usually adopted to define the design seismic action, has not been completely effective in calibrate the resistance: the overall construction is stronger, and that is generally a safe result, but the supported elements suffer a stronger action, and that is not safe regarding damage.
Often the over-resistance is due just to the "non-structural" exterior walls or partition walls, connected to the structure and structurally behaving. And because of their low resistance they usually suffer the most damages in even light earthquakes.
These remarks intend to emphasize that safety and damage protection are more effectively attained through a reliable design of the non-structural elements than by means of a sofisticated dynamic analysis of the bare structure. In order to provide the designer of a realistic evaluation of the acceleration that can strike the supported members, perhaps the better way, (so it seems at present), is to derive it from experience made by true buildings in true earthquake.

2. *EXAMINED RECORDS*

In order to collect data of actual behaviour

of buildings 401 records obtained in 12 different seismic events have been examined. The maximum acceleration recorded at the base ranges between 0.01g and 0.40g. Over 60 R.C. buildings from 3 to 24 stories height have accounted for; they represent a large set of buildings different for structural characteristics and constructive modalities, but selected among structurally regular buildings. All the peculiar situations, not statistically coherent to the most of the data, have been eliminated, and in such way all the examined set of buildings can be considered representative of the typologies provided for a code.
The only atypical case included in the study is the Imperial County Service Building (6 floors), strongly damaged by the earthquake on 1979.
In most of the buildings instruments had been placed on the basement and on the top, in other cases other intermediate floors had been instrumented (Decanini 1987, 1991).
249 values of acceleration records have been collected and compared.
For each case, the recorded accelerograms were analyzed and the amplification of the horizontal acceleration were determined in relation to the base acceleration. The E.P.A.values were accounted for. The available data were treated statistically, in order to conveniently organize them into groups(Merz 1977).
Data derived by four California earthquakes are prevalent in the data base, since many instrumented buildings were in that area.
The principal characteristics of the examined records are contained in table I.

3. *OBSERVED OVER RESISTANCE*

In various cases of buildings subjected to severe earthquake which have resulted only minor damaged, values of shear force at the base several times greater than those required by the code have been found (Bertero 1986; Meli 1991).
The maximum global shear force acting on the building during the earthquake can be. with good approximation derived from the recorded accelerograms. Such value, compared with the correspondent design force gives a clear informatiOn on the actual over-resistance of the building.In the cases reported in Fig.1 the seismic design coefficient is known, so it was possible to evaluate the ratio O.F.:

$$O.F. = \frac{\text{Seismic recorded coefficient}}{\text{Seismic design coefficient}}$$

It must be noticed that the buildings had suffered only some damage, bat values O.F. major than 1 have been always found, and often the actual strength was more than two or tree time the design.
The phenomenon was accentuated in

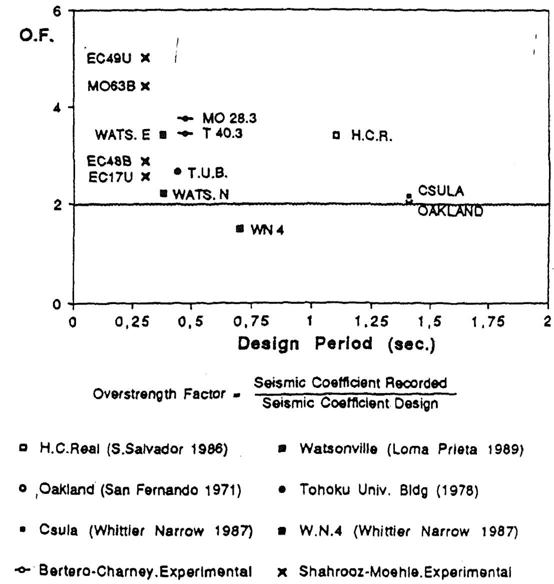

Fig. 1 - Observed Overstrength Building Factor

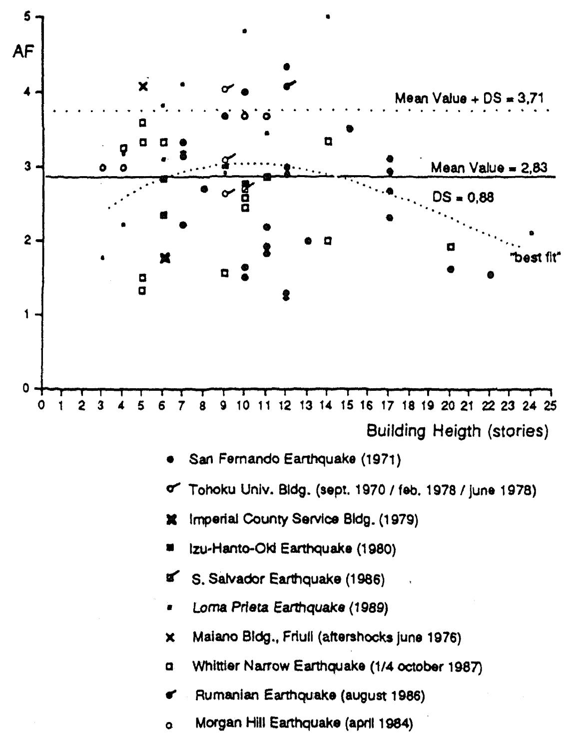

Fig. 2 - Horizontal Upper Level Building Amplification Factor.

experimental tests (Bertero, 1984; Shahrooz Moehle,1990).
As it has been said, such gap between the actual behaviour and the intent of the code

TABLE I - Characterics of the examined records

Earthquake	Magnitude	R (Km)	Number of Bldgs	Bldgs. Height (N. of Floor)	Total Records	Records Upper Levels	Peak Acc. Base (g)
San Fernando 9 Feb. 1971	M = 6.5	20 / 80	30	7 / 22	166	106	0.03 / 0.27
Japan 14 Sept 1970	M = 6.2	134	1 *	9 *	4	2	0.047
Japan 20 Feb 1978	M = 6.7	126	1*	9*	4	2	0.173
Miyagi K. O. 12 Jun 1978	M_L= 7.4	112	1*	9*	4	2	0.263
Imp. Valley 15 Oct 1979	M = 6.6	30	1	6	14	9	0.331
Izu Hanto Oki 29 Jun 1980	M = 6.7		4	6 / 11	18	10	0.007/0.035
San Salvador 10 Oct 1986	M_S= 5.4	4.5	1	10	5	3	0.34/0.47
Loma Prieta 10 Oct 1979	M_S= 7.1	18/171	12	3/24	89	62	0.04:0.39
Morgan Hill 24 Ap. 1984	M_L= 6.2	19/74	4	3/11	21	13	0.01/0.11
W. Narrow 1/4 Oct. 1987	M_L= 5.9 M_L= 5.3	9/109	13	4/20	56	30	0.03/0.39
Friuli, Aft. 8/17 June 76			1	5	16	8	0.01/0.02
Rumania 30/31 Aug 86			1	12	4	2	0.06/0.09
TOTAL			68		401	249	

* : Tohoku Univ. Bldg. - R : Epicentral Distance

reveals some inadequacy in the design procedure and reduces the effectiveness of the calibrations assumed in the prescriptions of the code.

4. MAXIMUM ACCELERATION AT DIFFERENT FLOORS; AMPLIFICATION FACTOR FOR HORIZONTAL ACCELERATIONS

In order to present the most important results derived from the records, the maximum floor acceleration was divided for the peak base acceleration. In this way the data appear as amplification factor A.F. (Merz, 1977).
In fig.2 the A.F. relative to the top floor has been plotted versus the number of floors (66 cases). The mean value of the A.F. at the roof is 2.83 with SD=0.88. It is evident the big scattering at every height, except, perhaps, over 17 floors. In addition no trend is possible to find out in A.F. for less then 17 floors buildings.
The amplification factor seems to be independent on the natural period of vibration up to T=1.7 seconds, as corresponding to a flat response spectrum.
The same information derives from the acceleration recorded at intermediate level (42 cases examined). At half high the mean value of A.F. is 1.87 with SD=0.48.
Accounting for the value 1 at the base of the building it is evident then the maximum acceleration linearly increases with the distance from the base.
Such variation is reproduced in fig.3, where

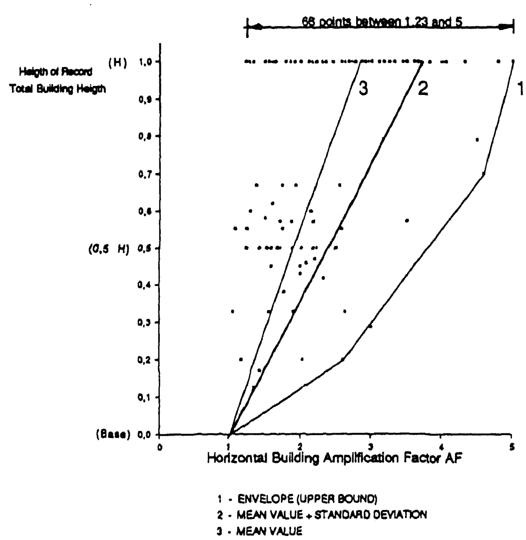

Fig. 3 - Distribution of Horizontal Building Amplification Factor over Building Heigth

the MV, the MV+SD, the envelope, has been represented. The equation corresponding to the MV+SD is:

$$A.F. = 3.7(h/H) + 1$$

where H is the total high and h is the distance of the considered level from the base.

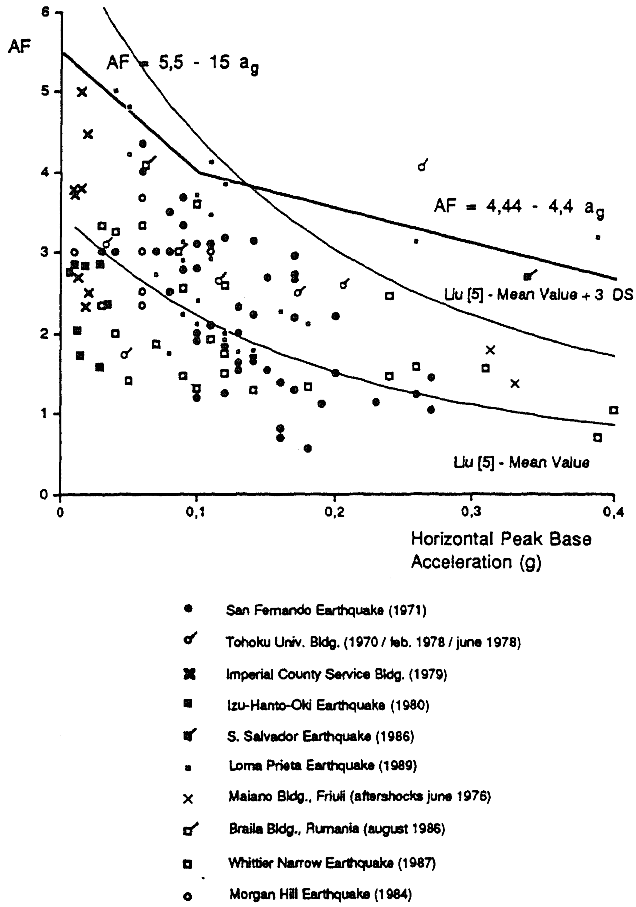

Fig. 4 - Horizontal Building Amplification versus Peak Base Acceleration.

5 AMPLIFICATION FACTOR VERSUS PEAK BASE ACCELERATION

Another interesting information derived from the collected records regards the relationship between the amplification factor and the peak base acceleration (Fig.4).
The data shows a clear trend: the amplification factor decreases with the increasing of the base acceleration, and, at last, is in agreement with usual expectation. With the increasing of the base acceleration the yielding of the structural elements goes on and they becomes less able to transmit the acceleration to the upper stories.
This result is strongly scattered too: a bilinear relationship can be derived as upperbound, a bit different by the one proposed by Liu on 1977 on the base of less number of data.
It is suggested the following expression of A.F. as function of the ground acceleration a_g:

A.F. = $5.5 - 15\ a_g$ $(0 \leq E.P.A. \leq 0.1g)$
A.F. = $4.4 - 4.44\ a_g$ $(0.1g \leq E.P.A. \leq 0.5g)$

6 SUGGESTED VALUES OF THE FLOOR ACCELERATION

On the base of the previous statistical elaborations is possible to propose the following values for the amplification factor at the different levels of a building, as function of the effective peak acceleration.

Severe Earthquake

EPA	0.25H	0.50H	0.75	1.00H
0.35	0.52	0.68	0.84	1.00
0.25	0.40	0.55	0.70	0.85
0.15	0.26	0.36	0.47	0.57

Frequent Earthquake

EPA	0.25H	0.50H	0.75	1.00H
0.18	0.24	0.41	0.53	0.65
0.13	0.21	0.30	0.39	0.48
0.08	0.14	0.19	0.25	0.30

7 CONCLUSIONS

- Stronger values of acceleration then expected have been recorded on actual buildings.
- No correlation has been found between the amplification factor and the total high of the building.
- The amplification factor is strongly scattered, as a demonstration that some relevant element, in the construction, is not controlled by the designer.
It is believable that non structural element have such significant influence.
- If from one side the recorded data allow to propose values of A.F. at every level of multy stories buildings, related to the E.P.A., from an other side it seems important to suggest a more conscious design of the "non-structural" elements, in such a way then they could be modeled into the structure or kept clearly aside.
- If this proposal is not put into effect, and the code do not account for it the effectiveness of the most of the design parameter is strongly cut down.

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