ON THE RELATIONSHIP BETWEEN OVERALL AND LOCAL DUCTILITY DEMAND FOR PLANE FRAMES SUBJECTED TO EARTHQUAKES

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SUMMARY

The use of inelastic response spectra in seismic design is recognized to be a good practical approach, but its adequacy to limit the ductility demand within acceptable values is not well assessed. This paper examines the response of frames designed according to Italian and CEB Seismic Codes; attention is focused on the variability of the overall and local ductility demand and of their ratio due to different assumptions and rules more or less closely satisfied in proportioning the resistant members. The effectiveness of some design modifications to obtain a more reliable distribution of ductility demand among girders and columns and to limit peak inelastic deformations is studied.

INTRODUCTION

In the seismic design of structures subjected to strong ground motions inelastic deformations are commonly accepted; notwithstanding this, the response-spectra-based modal analysis is generally considered to be the most practical approach to perform a seismic analysis of a structure. The capability of structures to survive high intensity earthquakes is taken into account making reference to inelastic design response spectra obtained through modification of a smoothed linear response spectrum using factors which depend on the accepted ductility level. This procedure shows a number of severe defects for multidegree-of-freedom structures, for which the behaviour and the distribution of inelastic deformations is markedly sensitive to actual excitation input as well as to dynamical characteristics of structure and local resistance of members (Refs. 1, 2, 4).

Actual aseismic Codes, although based on similar design philosophies, exhibit some differences as regards the shape of design response spectrum and the technique for calculating reduced seismic elastic forces. But in any case many studies have evidenced that the Code procedures predict with small accuracy the average ductility demand and are not able to predict the maximum local demand (Refs. 3, 4). Thus, other design rules which influence the proportioning of the structural members become quite important in order to select a structure showing as uniform distribution as possible of ductility demand with limited peak values.

In this work the inelastic response of two-bay five-story and ten-story plane frames designed according to CEB Seismic Code (CSC) proposed in Ref. 5

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and to Italian Seismic Code (ISC) - now under revision - is studied to enlight some aspects of the design of r.c. structures, mainly those - as Code prescriptions and practical proportioning decisions - which affect the relationship between overall and local ductility demand, the latter parameter being quite important to evaluate the effectiveness of the Code.

DESIGN OF FRAMES

The ISC refers to an allowable stress design and prescribes a unique design response spectrum to be used for calculating seismic forces; though no explicit reference is made to a design ductility level, the acceleration values are such that the spectrum belongs to the class of inelastic design response spectra. Instead the CSC assignes spectra with different shapes depending on several factors and, most important, their values are related to three specified ductility levels; besides an ultimate state design procedure is prescribed. Therefore, for the purpose of obtaining comparable results between the nonlinear responses of frames designed according to the two Codes, the first item is concerned with the proper selection of the ductility level in the CSC. Taking into account that a safety factor of about $2 \div 2.5$ with respect to ultimate load supported by each member is obtained when using ISC, the ductility level II was selected, which corresponds to a behaviour factor of 3.5 to be applied for reducing the elastic forces (Ref. 5). Referring to standard values of the coefficients accounting for the importance of building, for the site, etcetera and to a peak ground acceleration $A = 0.35\,g$ in CSC, the two spectra are plotted in fig. 1, from which with the above mentioned safety factor comparable forces result in the low periods range, while for $T > 0.5$ s those prescribed by ISC are somewhat higher.

However, this is not the unique difference between the two Codes. Indeed, while the Italian one does not impose to the structure any specific requirement to improve the seismic resistance, the CSC asks for the satisfaction of some strength and stiffness requirements, for the compliance with well-extablished performance criteria and prescribes a number of local and global verifications as well as specific design and detailing. In the following, attention will be mainly posed to the stiffness and strength requirements which read:

a) the story 'stiffness ratio', defined as the ratio of the inverse of the story drift to the average of the inverse of the story drifts, shall not less than 0,70 at any story;
b) the ratio between actual and design story shear capacity (T_u/T_d) at each story shall not vary more than 20 percent along the building height;
c) at any beam-column joint, the sum of the flexural strengths of the columns shall not be less than the sum of those of the beams.

Two frames with 5 and 10 stories respectively were designed. The global dimensions are shown in fig. 2; square sections, all equal at a given story, were assumed for the columns. For the five-story frame two solutions were considered as regards the distribution of column sections along the height, while the distribution of girder sections does not vary in the two cases. Besides, local modifications were introduced into the original design of both frames concerning the steel reinforcement ratio (ρ) of columns, and thus their yield moments, in order to analyse the importance of some prescriptions of CSC more or less closely satisfied or to simulate different design solutions allowable with the ISC.

The set of frames considered is described in Table 1 in which section dimensions and periods are given. Each frame (F) is labelled using four symbols, of which the second (5 or 10) indicates the number of stories, the third (1 or 2) different dimensions of column sections, the fourth (I or C) the Design Code referred to; local modifications introduced in frames .1 or .2 give rise to .1B or .2B frames. The yield moments M_y of all members are reported in Table 2.

The masses considered in the design correspond to full dead plus 0.3 live load. A static analysis was performed to compute member forces due to gravity loads while the earthquake forces were determined by modal analysis using spectra in fig. 1. The section dimensions adopted for the columns are such that the maximum compressive stress under static loads is less than 0.2 cylindrical ultimate resistance. Plastic moment capacities of columns were determined through the interaction diagram in correspondence to gravity axial loads only. When using CSC the minimum requirement for ρ of both girders and columns is satisfied. In all cases the same yield moment was assumed at the two member ends, equal to the maximum required value.

As a general observation on the two original frames designed according to ISC and CSC it can be noticed that the level and the distribution of yield moments are quite different, although the dimensions of the sections are the same. Due to different procedure referred to - allowable stress or ultimate state design - and to the greater level of earthquake forces prescribed by ISC, the M_y values of the columns in this case are notably higher; they are higher as well for the beams at the intermediate and upper floors, notwithstanding the most unfavourable condition for these latter is the gravity one when using CSC. Besides, since it was choosen to change the dimension of the columns every two stories for easier construction, the required ultimate resistance of the columns with ISC shows a pattern along the height with several sudden changes, in absence of any requirement of minimum reinforcement ratio (ρ_{min}) ; better is the case when the variation of column dimension is performed at every story (see F5.2I). Instead, when referring to CSC the distribution is much more regular, due to necessity of satisfying $\rho_{min} = 1\%$ in several columns, which produces overstrengthened members, mostly in the ten-story frame. All frames designed according to CSC satisfy the stiffness, stability and 2nd order effects prescribed verifications as well as previously mentioned requirement a), while requirements b) and c) are locally violated in some cases.

METHOD OF ANALYSIS AND GROUND MOTIONS

Many models were introduced to study the inelastic seismic response of a framed structure; the results obtained often evidenced the reliability of simple models as well in predicting the yielding pattern. Since in this work the effectiveness of two Codes prescriptions is evaluated by comparing the response of different frames, it was considered acceptable to model each member as beam elastic element between two elasto-perfectly plastic hinges at the ends; successively, the results will be checked by a more refined model.

In measuring the inelastic response, the rotational ductility μ_θ was used, which is the ratio between the maximum plastic rotation θ_p at the member end and the rotation θ_y at yielding moment for an antisymmetric deformed shape.

The inelastic response of frames was calculated by dynamic time-history analysis using time integration scheme utilized in Ref. 5 already. Within each time step of the procedure, the elasto-plastic flexural solution $\underline{m} = \{m_i m_j\}^T$ is obtained making use of the Haar-Karman principle. The values of m_i, m_j furnish the plastic rotations at the two ends of the beam. When gravity loads are considered θ_y is not exactly the yield rotation any more and thus μ_θ is not the rotational ductility ratio in the strict sense but a well approximate value.

In performing time-history analysis the following assumptions were made: masses are lumped at the floor levels, the behaviour of the member is described as single component model with elasto-perfectly plastic hinges the initial stiffness of which is assumed like that of the uncracked beams and columns, the yield moment of columns does not vary with the axial load, initial moments at the member ends due to gravity loads are considered, P-δ effects are neglected, no damping is introduced.

The ground motion characteristics - intensity, duration and frequency content - markedly affect the dynamic structural response; thus it may be advisable to work with an ensemble of several representative accelerograms. Herein two sets of few input motions were used in order to limit the cost of analysis. The first set is constituted by three artificially generated accelerograms which match the CEB spectrum. The generation procedure used is reported in Ref. 6; the target CEB spectrum and the three spectra of generated accelerograms are compared in fig. 3a.

The second set is constituted by three Californian earthquakes, selected from a set of 13 accelerograms used in a previous work (Ref. 2), which have been recognized suitable for a numerical investigation. As regards their intensity, the acceleration peak was scaled in such a way to have a spectrum of the same level of the CEB spectrum in the period range 0 - 0.5 seconds (fig. 3b).

ANALYSIS OF RESULTS AND CONCLUDING REMARKS

Inelastic structural response is analysed in terms of maximum rotational ductility. Plots of mean of the maximum values obtained in the various floors with the three artificial accelerograms for columns and girders are given in figures 4 and 5. In Table 3 the following quantities are reported: mean for all member ends (μ_m), mean for all floors of the floor maxima (μ_f), absolute maximum in the frame (μ_{max}), amplification coefficient $a_\mu = \mu_{max}/\mu_m$.

The results obtained for the 5-story frame are first analysed, by comparing the responses of F5.1I and F5.1C. The former shows story concentration of ductility demand at the 2nd story columns, due to too low ρ value allowed by ISC, which produces a localized lack of resistance; as a consequence μ_{m_c} is much greater than μ_{m_g}. Instead F5.1C exhibits better yielding pattern over the structure with acceptable distribution of local and mean ductility between girders and columns, just due to imposition of ρ_{min}, which produces smoothed variation of the ratio T_u/T_d. The global μ_m has a value very closed to the predicted one and the a_μ coefficient is satisfactorily low for both columns and girders.

The role played by the ratio T_u/T_d is evidenced by the results of F5.1BI, where the discontinuity of 40% present in F5.1I at the 2nd story is reduced to 25% by increasing ρ up to 1%. The inelastic deformation request changes in such a way that μ_{m_c} becomes lower than μ_{m_g} and a notable decrease of global μ_m follows with respect to F5.1C as well. This latter result, which can be recognized also in the comparison between F10.1C and F10.1I, descends from the higher level of the ISC design spectrum at periods greater than 0.5s, provided a good proportioning of the structure be assured. Nevertheless, the a_μ value of both columns and global is still greater in the Italian frames, since stronger concentrations occur.

Consider now the frames F5.2 which exhibit more gradual variation of column section dimensions. Crude application of ISC produces a suitable distribution of yield moments thus obtaining a good inelastic response with $\mu_{m_c} < \mu_{m_g}$. On the contrary CSC gives a frame (F5.2C) showing greater ductility demand in the columns and global μ_m value a little higher than the predicted one. In this frame the strength condition c) at the beam-column nodes is not exactly satisfied at the internal nodes of 2nd floor, where concentration of inelastic deformation occurs. When this condition is satisfied correctly (F5.2BC), local and overall improuvement is achieved in columns. In this context, the better behaviour of F5.2I already mentioned (strong columns-weak girders), also with respect to F5.2BC, is just bound up with the condition c) resulting largely satisfied with the exception of the upper floors. In any case however the local concentrations of ductility demand in the columns of CEB frames are lower, due to explicit imposition of strength requirements b) and c).

One solution only is presented for 10-story frame, whose proportioning takes advantage of the information gained with 5-story frame and can be considered similar to F5.2 frame. Both F10.1I and F10.1C behave as strong columns-weak girders frames, the former for the same reasons as F5.2I, the latter for being largely overstrengthened (+40 ÷ 90%) in the lower and intermediate columns in order ρ_{min} be satisfied. Analysis of ductility distribution over height shows more uniform pattern for F10.1C with lower a_μ value, while F10.1I has an irregular pattern with peaks corresponding to discontinuity of T_u/T_d ratio and to overstrength of girders at the upper floors due to gravity loads.

All the points made refer to mean response of frames to artificial motions. By way of example, the ductility distribution obtained with single realizations for F5.1C are plotted in fig. 6 which shows limited variations with respect to mean values; more notable differences occur locally in those frames which do not show regular yielding pattern along height. The general features of the response evidenced for various frames are confirmed with recorded motions (see Table 2), the effects of which are however of minor entity due to steepest descent of the relevant spectra for $T > 0.5$ s.

From the results presented some concluding remarks can be drawn. The necessity of paying attention to some additional prescriptions with respect to strength required by inelastic spectrum modal analysis in the proportioning of members frame clearly comes out. In particular a proper value of the column-beam flexural strength ratio (major than unity) is essential to surely enforce ductility demand into girders; to the same purpose a technique of moment redistribution to reduce negative moments at the beam ends can be suitably utilized. Within this context the design procedures based on the limit state are recognized more realistic and more reliable with respect to allowable stress design procedures. Besides the variation along the height of the T_u/T_d ratio (or similar parameter) must be carefully controlled to avoid undesirable concentrations of inelastic deformations.

Under these conditions a satisfactory relation between overall and local ductility demand is obtained, connected with a quite uniform distribution of inelastic deformations among resistant members, relation which results also less sensitive to input motions. Thus a global ductility parameter can be considered representative of the inelastic response of the whole structure and can be reliably referred to in deriving Code design response spectra. With a certain number of design contraints in the proportioning of the structure (see Ref. 1), it seems possible to limit mean ductility demand within prescribed values and to control local inelastic deformations also by elastic analysis.

ACKNOWLEDGEMENTS

This study is partially supported by C.N.R. (Italian Research Council) under Grant No. CT 80.02294.07.

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Table 1

Floor	F10.1I-F10.1C col.	gird.
10	35	30 x 50
9	40	30 x 60
8	45	40 x 60
7	50	40 x 60
6	55	40 x 60
5	55	40 x 70
4	60	40 x 70
3	60	40 x 70
2	65	40 x 70
1	70	40 x 70
T	1.171	

Floor	F5.1I F5.1C col.	F5.2I F5.2C col.	both frames gird.
5	30	35	30 x 50
4	40	35	30 x 50
3	40	40	30 x 50
2	50	45	30 x 60
1	50	50	30 x 60
T	0.904	0.947	

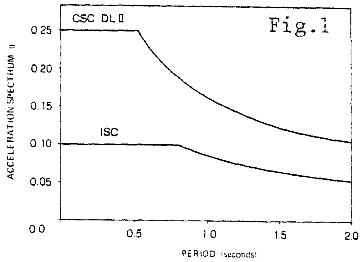

Fig.1

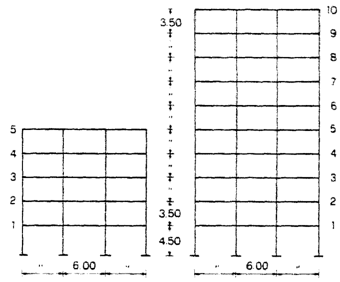

Fig.2

Yielding moments girder(g) columns(c) ext(e) int(i)

Table 2

a)

Floor		F5.1I F5.1BI e	F5.1I F5.1BI i	F5.1C e	F5.1C i	F5.2I e	F5.2I i	F5.2C F5.2BC e	F5.2C F5.2BC i
5	g	21.2	20.6	20.1	19.2	21.0	20.1	19.5	18.6
	c	16.2	11.9	10.4	9.5	15.0	8.6	11.5	10.1
4	g	27.1	25.6	23.6	21.9	26.4	25.1	22.9	21.2
	c	19.2	19.7	15.0	18.2	21.8	23.1	13.3	16.5
3	g	32.4	31.0	29.7	27.2	31.6	29.9	29.5	26.8
	c	26.2	31.0	16.9	22.2	26.4	30.8	17.1	22.4
2	g	39.3	36.5	38.2	34.3	38.3	35.6	37.7	33.1
	c	18.8*	29.8+	27.1	33.4	26.6	36.8	22.1	27.0°
1	g	40.3	38.2	40.5	36.3	40.1	37.2	40.1	35.3
	c	45.9	51.6	31.7	39.0	41.8	51.6	31.3	38.6

*27.1 for F5.1BI, °31.0 for F5.2BC, +31.3 for F5.1BI

b)

Floor		F10.1I e	F10.1I i	F10.1C e	F10.1C i
10	g	21.6	19.7	13.5	13.5
	c	12.8	10.0	10.7	16.0
9	g	29.0	31.0	23.1	23.5
	c	18.6	19.0	18.2	20.8
8	g	38.4	39.0	34.0	32.6
	c	25.1	31.0	27.7	30.5
7	g	44.4	45.0	41.3	39.6
	c	28.1	40.2	30.3	34.6
6	g	49.0	49.5	45.0	45.0
	c	31.5	44.5	43.1	46.9
5	g	58.0	56.8	56.6	52.7
	c	41.4	65.1	46.6	49.8
4	g	60.7	59.5	58.7	56.0
	c	44.7	70.2	60.6	65.1
3	g	63.8	62.1	62.7	59.3
	c	56.8	89.1	64.0	67.6
2	g	65.7	63.6	65.3	61.5
	c	61.2	98.2	81.2	85.7
1	g	63.6	60.4	63.4	58.7
	c	109.0	129.6	101.3	106.9

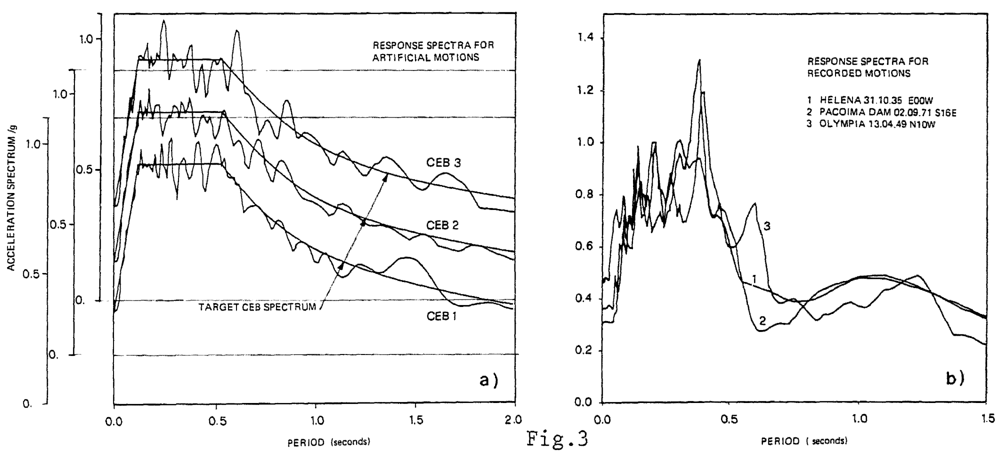

Fig.3

Table 3

Mean and maximum ductility values

	Frame	girders μ_m	girders μ_f	girders μ_{max}	girders a_μ	columns μ_m	columns μ_f	columns μ_{max}	columns a_μ	global μ_m	global a_μ
artificial	F5.1I	2.62	3.41	4.66	1.77	5.77	7.06	23.7	4.07	4.42	5.30
	F5.1C	3.25	3.75	4.98	1.53	3.60	4.13	6.13	1.70	3.45	1.78
	F5.1BI	3.14	4.04	4.94	1.58	3.00	3.68	5.57	1.87	3.06	1.89
	F5.2I	4.05	5.27	6.53	1.58	2.65	3.70	9.35	3.44	3.25	2.80
	F5.2C	2.83	3.73	5.62	1.98	4.46	5.10	6.50	1.46	3.76	1.73
	F5.2BC	3.22	3.87	5.66	1.76	3.99	4.56	6.71	1.67	3.66	1.83
recorded	F5.1C	1.95	2.33	3.69	1.89	2.57	2.94	4.03	1.56	2.30	1.88
	F5.2I	2.26	2.72	3.82	1.69	1.85	2.33	4.10	2.22	2.03	2.05
	F10.1I (a)	3.98	4.57	6.49	1.63	2.88	3.51	7.92	2.67	3.35	2.36
	F10.1C (a)	4.98	5.43	7.36	1.48	2.26	2.60	4.97	2.08	3.42	2.14
	F10.1C (r)	3.76	4.14	6.50	1.72	1.58	1.82	3.20	1.88	2.51	2.57

Fig. 4

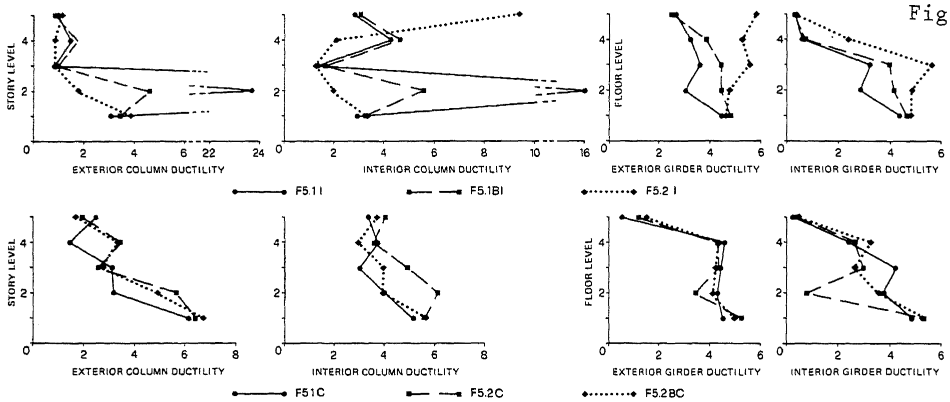

Fig. 5

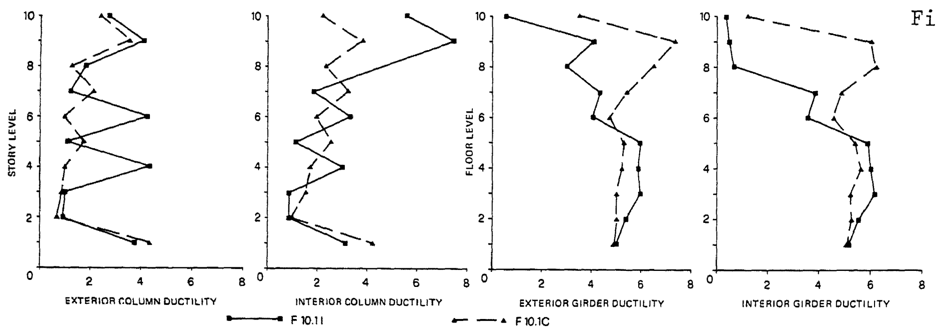

Fig. 6