ISORELIABLE SEISMIC DESIGN OF R/C COLUMNS

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SUMMARY

An investigation on the safety level obtained by the application of seismic design rules is the object of this work.

A Level I procedure is set up for the design of r/c seismic structures, accounting for their ductility by the use of convenient design spectra.

Probabilistic safety checks are then performed, according to an improved Level II analysis.

The results of an extended investigation show the influence of the major design parameters on the safety level. The uniformity of the reliability obtained for different structural cases proves the effectiveness of the design procedure.

INTRODUCTION

The safety levels ensured by seismic codes are up to now little investigated.

In this work, a single type of structure, as the isolated r/c column, has been selected for a systematic investigation toward that end.

This choice derives from the need of a design criterion for industrial plant buildings, whose structure often consists only of columns and roof. But the results can also be meaningful for multistory buildings with soft first story and rigid upper part.

The safety analysis raises up a wide range of problems, as:

a) selection of basic random variables, and probabilistic definition of them;
b) definition of a structural model and of "limit" deformations;
c) evaluation of the response to the seismic action in terms of deformation;
d) computation of the probability of failure.

The operative choices concerning the above items will be described.

The major need is that the formulation of all of them must be mutually compatible. A suitable limit state function must be realistic enough to properly describe the structural behavior, and must explicitly incorporate the variables affected by the uncertainties.

The limit state function expresses the condition that the deformation demanded by the seismic action is equal to the deformation available in the structure.

The models employed, structural and seismic, are formulated in order to explicitly depend on the basic random variables.

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THEORETICAL MODELING

Model of the structure.

The relationship between lateral displacements and a cyclic lateral force for a r/c column can be reproduced by a multilinear diagram defined by few characteristic points, corresponding to the decompression (D), the yielding (Y), and the failure (U) of the critical section, respectively (fig.1).

The analysis of the column, accounting for a realistic distribution of the curvatures as shown in fig.2, leads to the following expressions of the coordinates of the points Y and U:

$$Y \equiv \{\Delta_Y = \frac{\theta_D}{c} + (\theta_Y - \frac{\theta_D}{c}) \frac{3 - 2c - c^2}{4} \frac{L^2}{3} \quad , \quad F_Y = \frac{M_Y - N\Delta_Y}{L} \}$$

$$U \equiv \{\Delta_U = \Delta_Y + (\theta_U - \theta_Y) \ell_p (L - \frac{\ell_p}{2}) \quad , \quad F_U = \frac{M_U - N\Delta_U}{L} \}$$

where θ_D, θ_Y, θ_U are the curvatures of the cross section in the typical conditions.

In view of the dynamic analys the scheme in fig.1 may be replaced by a bilinear skeleton Takeda model, defined by the points Y and U only, and having as parameters:

$$T = 2\pi \sqrt{(N\Delta_Y/F_Y g)} \quad ; \quad \alpha = (F_U - F_Y)\Delta_Y/(\Delta_U - \Delta_Y)/F_Y \quad ; \quad \mu = \Delta_U/\Delta_Y \quad ;$$

where: T is the natural period in the elastic range, derived from the slope of O-Y; α is the relative slope of the branch Y-U, that may result strain-hardening or softening, depending on the importance of the 2nd order effects of the axial force; μ is the ultimate ductility factor.

Random variables. The above parameters are functions of the basic variables controlling the behavior of the materials and of the structure, which are:

f'_c (concrete strength)
f_Y (yield strength of steel)
ε'_{cu} (ultimate concrete strain)
ℓ_p (equivalent length of plastic hinges)

All these variables have been assigned log-normal distributions.

The first two variables mainly influence the yielding point Y of the force-displacement relationship, while the others affect the ultimate deformation, hence, the available ductility.

It may be observed that point Y affects both the yield strength of the structure F_Y and the elastic stiffness $K = F_Y/\Delta_Y$, which determines the period T. Then, the global characteristics F_Y and T are correlated, and a probabilistic analysis dealing with them as with independent variables would be uncorrect.

On the contrary, the strengths of the materials are independent, thus being suitable as basic variables.

It is not exactly the same for the variables ℓ_p and ε'_{cu}. These are likely to be related, as the transverse reinforcement, as well as shear and normal forces, affect both of them. However, they control a single global characteristic: the ultimate ductility. Assuming them as independent, due to the difficulty of expressing the correlation, leads to an overestimation of the randomness of the ductility.

Model of the seismic action.

The seismic action depends on the intensity and on the frequency content of the ground motion.

The maximum value of the peak ground acceleration (PGA) for the structure's lifetime is taken as the paramter of the intensity.

Following ref.[2], the latter random variable has been assigned an extreme type I distribution. The mean value $\bar{A} = 0.1$ g and the coefficient of variation $V_A = 0.5$ are representative of the seismicity of the Friuli region (Italy).

On the other hand, the influence on the frequency content was analyzed (see refs.[3,6]) by means of a numerical simulation procedure, using accelerograms generated by the computer program PSQGN (ref.[7]), which fit with the USNRC Reg. Guide 1.60 elastic spectrum, and with the Friuli recorded spectra (1976), too.

The statistics of the response of the Takeda model have been worked out as functions of the model's parameters T, α, and of the strength ratio $\eta = F_Y/AM$ between structure's strength and non amplified response.

For design purpose, a diagram has been drawn, plotting on the plane(T, η) a series of curves for given values of the ductility μ, representing a 0.84 fractile of the ductility demand.

The curves are called "design" spectra (fig. 3). In fact, for a given value A of the PGA, the diagram yields the value of the design capacity $F_Y = \eta AM$ needed for a 0.84 probability of the corresponding available design ductility not being engaged.

The spectra are obviously valid only for the given structural model and for sites where the USNRC response spectrum may be accepted.

The uncertainty related to the shape of the accelerograms, within the above limitation, is thus attached to the value of the ductility demand, which is expressed in the form

$$\mu_D\ (T,\alpha,\eta) = \bar{\mu}_D\ (T,\alpha,\eta)\ \rho$$

where $\bar{\mu}_D$ results from the statistical investigation, and ρ is a random variable with an extreme type I distribution: $\rho\ (\ \bar{\rho} = 1\ ,\ \ V_\rho = 0.35)$.

DESIGN PROCEDURE AND SAFETY CHECKS

The limit state function.

According to the definition, the limit state function is expressed by the equality of demanded and available ductilities:

$$\mu_D = \mu_A$$

Given the column's dimensions and a value of the axial force N, for every combination of the random variables, contained both in the structural model $\underset{\sim}{R} \equiv \{\ f_c,\ f_y,\ \varepsilon'_{cu}\ ,\ \ell_p\ \}$ and in the seismic model $\underset{\sim}{S} \equiv \{\ A,\ \rho\ \}$, the corresponding available and demanded ductilities are determined.

In fact, the strucutural model allows for the calculation of the parameters $T(\underset{\sim}{R} \mid N)$, $F_Y(\underset{\sim}{R} \mid N)$, $\alpha(\underset{\sim}{R} \mid N)$, as well as the available ductility $\mu_A(\underset{\sim}{R} \mid N)$. The design level, referred to the actual value of the PGA, is represented by $\eta = F_Y / AM$ (where $M = N/g$).

The ductility demand is deduced from the results of the statistical analysis, as function of the parameters T, α, η just obtained and of the random variable ρ.

Design by the Level I Procedure.

The design is performed replacing all the random variables X by the design values X_d obtained by means of suitable "characteristic values" X_K and of suitable partial safety factors γ : $X_d = \gamma X_K$.

The structure is designed in order to fulfill the equation

$$\bar{\mu}_D (T_d, \alpha_d, \eta_d)\, \rho_d = \mu_A (\underset{\sim}{R}_d \mid N)$$

where T_d, α_d, η_d are calculated using the design values of the variables.

In the numerical applications of the present work, the following values have been adopted.

X	$\bar{X}$	V_x	X_k	γ	X_d
f_c	40 MPa	0.15	30 MPa	0.55	16.5 MPa
f_y	480 MPa	0.05	440 MPa	1.0	440 MPa
$\bar{\varepsilon}'_{cu}$	0.007	0.15	0.007	1.0	0.007
ℓ_p	$\bar{\ell}_p(H,L)$	0.15	$0.78\, \bar{\ell}_p$	1.0	$0.78\, \bar{\ell}_p$
ρ	1.0	0.35	1.35	1.0	1.35
A/g	0.1	0.5	0.194	1.0 1.22 1.44 1.66 1.88 2.10	0.194 .237 .280 .323 .365 .408

For sake of simplicity, in the numerical applications the graphs with $\alpha = 0$ (fig. 3) were used for all the cases.

120 structures were examined. Different heights of columns (L = 3.,4.,4.5, 5.,6. meters) were considered; for each height, four different cross sections, corresponding to different values of the average compressive stress (fig. 4). All the cross sections are rectangular with symmetric reinforcement: six different reinforcements, for satisfying the design equations corresponding to the six values of γ applied to the acceleration, have been selected.

The values of the average stress, i.e. the dimensions of the section, noticeably influence the column's available ductility. The highest is $\bar{\sigma}$, the less is the ductility: higher capacities are necessary for satisfying the limit state equation. For example, in fig. 4 the necessary capacities F_y versus $\bar{\sigma}$ are shown for a value $A_d/g = 0.28$ (i.e. $\gamma = 1.44$). The shape of the curves is conditioned not only by the available ductility, but also by the dynamic response.

Reliability analysis.

Putting the limit state equation in the form:

$$Z = \mu_A (\underset{\sim}{R} \mid N) - \bar{\mu}_R (T,\eta,\alpha)\, \rho$$

the probability of failure is measured by the probability of Z being negative:

$P_F = P_r\ (Z \leqslant 0)$

The computation of P_F has been performed according to the Level II method (ref. |1|),by means of the "safety distance" β. The probabilistic effectiveness of the index β has been improved (with reference to the original meaning) accounting for the actual shape of the safety domain around the point of minimum distance (see ref. |4|). After this improvement, the β value measures the correct probability of failure with enough approximation.

Results.

All the structures designed have been subjected to the Level II safety check.

In fig. 5 the values of β obtained are reported as function of the column's height L. It may be observed tha the β values are rather uniform. This derives from the design criterion being sophisticated, i.e. accounting for all the major parameters affecting the collapse, and from the highest importance of the uncertainty of the seismic intensity with respect to the others.

A systematic trend toward the reduction of safety is observed, with the increaseing of the column's height L, depending on the design having been performed with $\alpha = 0$ (diagram of fig. 3), while the second order effects of the axial loads make the slope α decrease with the increasing of L.

On the other hand no trend appears referred to σ . This means that, given the height of the column L and the axial force N, the same safety would be obtained with any cross section, provided that the reinforcement is designed accounting for the available ductility,according to the criterion defined before.

In fig. 6 the values of β obtained for every case are plotted as function of the design acceleration . Obviously, the safety increases with the increasing of the design acceleration.

E.g. for obtaining an average $P_F = 10^{-5}$ ($\beta = 4.27$), the design collapse acceleration that should be assigned results: $A_d \simeq 0{,}32$ g, i.e., $\gamma = 1.66$.

In that figure, the degree of non uniformity of the safety obtained by the Level I design is more clear. The deviation of the β value from the mean appears about 0,25, on the ensemble of the cases.

CONCLUSIONS

The analyses illustrated in the preceding paragraphs aimed to demonstrate that, when the structure is designed accounting for the ductility and the nonlinear dynamic response, the design value of the collapse acceleration can define its reliability, within a reasonable margin of error.

The model employed in the structural analysis had to contain explicitly and realistically the basic parameters , affected by the main sources of uncertainty.

But not always that objective can be easily attained. In fact, the way of computing the available ductility in the employed model may call some criticism, and the computational effort for relating the ductility to ε'_{cu} and l_p may be not completely justified.

The design would be simplified if some tables were available, showing nominal values of the ultimate ductility, as function of the fundamental parameters.

Perhaps, under certain conditions about the detailing of reinforcements, it would be possible to furnish values of the ultimate ductility that could be realistically expected as a minimum, given the height of the column, the dimensions of the section, and the average concrete stress.

The outcome of the investigations suggests some general remarks, synthetized as follows:

1) The aumont of nonlinear deformations available in the structure is an essential design element.
2) The randomness of the seismic action, referred to both the intensity and the frequency content, is definitely prevailing on the incertainties of the structural behavior, which have little influence on the overall reliability.
3) The Level I design accounting for ductility, with the usual values of the safety coefficients of the materials, yields a reliability tight related to the value of the design acceleration.

The extension of the results illustrated here to structures more complex than single columns requires an appropriate analysis of their dissipative characteristics, and forms a matter now under research.

ACKNOWLEDGMENT

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REFERENCES

|1| "Common Rules for Different Types of Constructions and Materials" C.E.B.-F.I.P. International Recommendations, Vol. I, 3rd Editions 1978.

|2| C.E.B.-C.E.C.M-C.I.B.-F.I.P.-I.A.B.S.E.-R.I.L.E.M. Joint Committee on Structural Safety: "Basic Notes on Actions" Lisbon 1976.

|3| Giannini, R., Giuffrè, A.,Pinto,P.E.: "Reliability and Optimization in Seismic Design" - Proceedings of the Int.nl Conf. on Applications of Statistics and Probability in Soil and Structural Design. Sydney - Jan.1979.

|4| Giuffrè,A.,Giannini,R.: "Contributions to the Implementation and Applications of the Level II Method". _ C.E.B. Meeting of T.C.C. Group - London, November 1978.

|5| Giannini, R., Giuffrè,A., Menegotto,M.: "A Reliability Analysis for Seismic Limit Design of R.C. Columns",- Ist. Scienza e Tecnica delle Costruzioni - Univ. Roma, N.123, 1979.

|6| Murakami, M., Penzien,J.:"Non Linear Response Spectra for Probabilistic Seismic Design of Reinforced Concrete Structures". Sixth WCEE, New Dehli, 1977.

|7| Ruiz,P., Penzien,J.: "Probabilistic Study of the Behavior of Structures during Earthquakes". Earth. Eng. Res. Center, N.EERC 69-3, University of California, Berkeley, 1969.

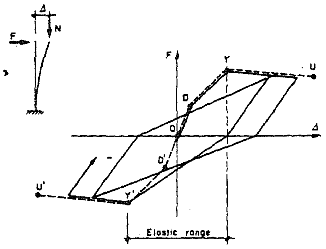

Fig.1

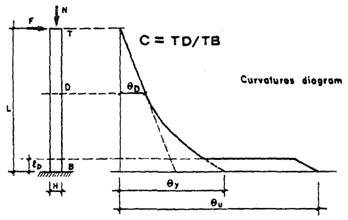

Fig. 2

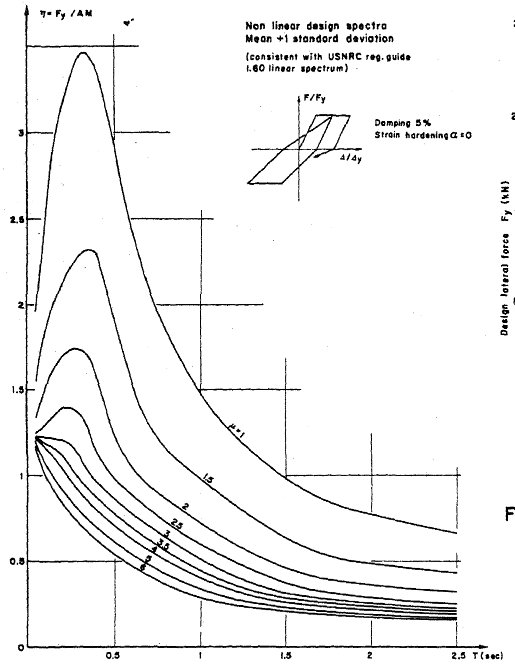

Fig. 3

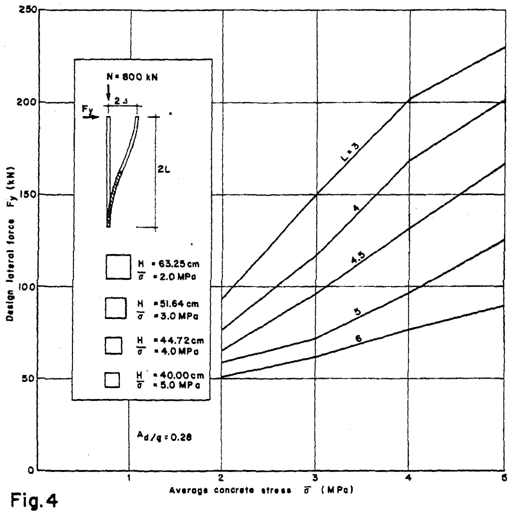

Fig.4

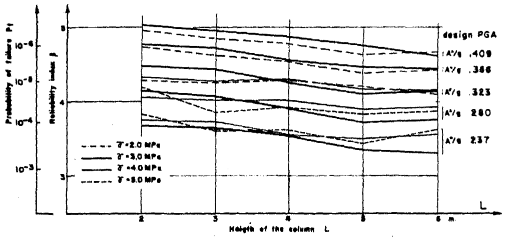

Fig.5

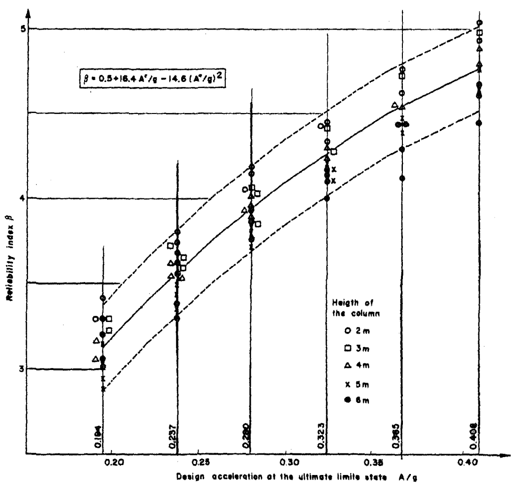

Fig. 6