Studies for comprehensive iso-reliable seismic design

STUDIES FOR COMPREHENSIVE ISO-RELIABLE SEISMIC DESIGN

by

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INTRODUCTION

It is generally recognized that seismic resistant design must be based on a probabilistic treatment of the variables involved. In its simplest formulation, only the randomness of the input is considered, and the seismic action is specified by means of a single parameter (i.e. peak acceleration, velocity, etc.): the design is based on a selected fractile of this parameter. The next step involves modeling the ground motion by means of some kind of random process, thus introducing an additional source of variability on the response.

For a comprehensive approach, however, the uncertainties related to structure's behavior must also be accounted for. The latter may derive from a number of basic causes, including the scatter of material and element characteristics, as well as the uncertainty related to the analytical model.

In this study an attempt is made toward a comprehensive treatment of seismic reliability accounting for all the above mentioned uncertainties.

The level II method of reliability analysis is applied to assess the reliability of any design situation with respect to a predefined limit state involving some degree of structural damage. The results are presented in the form of charts, analogous to the familiar response spectra currently used for seismic design, and which give the design factor needed to garantee (with any chosen probability value) a prescribed level of non-linear response (max ductility ratio).

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IDENTIFICATION OF THE RANDOM VARIABLES

A) SEISMIC INPUT

Several criteria have been proposed for evaluating the seismic risk at a site, given the historical and geological information relative to the surrounding region: see for ex. ref. |1|.

In the present approach, the seismic input is modeled by means of a suitable random process, scaled by a (random) parameter representative of the seismic intensity. In the numerical applications to follow, the peak ground acceleration has been choosen as the intensity parameter, and an extreme type I distribution has been adopted for the maxima of this parameter during a given reference period. Any other intensity parameter (ex.: peak ground velocity) or form of distribution could be introduced without difficulty.

The random process selected to simulate ground motion is a (unit intensity) non stationary gaussian process with a constant power spectral density: samples of such a process have been produced by means of the computer program: (PSQGN), |4|.

The generated samples have the following characteristics: duration 15 secs, central frequency ω=15,6 rad/sec, shaping function t_1=4 sec (duration of initial build-up), t_2=11 sec (end of stationary portion).

B) STRUCTURAL MODEL

For the purpose of the present parametric study, the bi-linear histeretic stiffness-degrading model proposed by Takeda |2| has been adopted. The normalized model is shown in fig. 1, where it is characterized by F_y, X_y, and K_2/K_1, which represent the yield force, the yield displacement, and the strain-hardening ratio, respectively.

The equilibrium equation in fig. 1 is expressed in terms of the ductility ratio $\xi=X/X_y$. The independent structural parameters which appear in the equation are: the undamped frequency $\omega=\frac{2\pi}{T}$, the damping ratio υ , the strain-hardening ratio K_2/K_1 (included in the restoring force $f(\xi)$), and the factor $\eta=F_y/A\cdot M$, where A is the peak ground acceleration and M is the mass of the model. (The latter is not an independent variable, being $M = =K_1/\omega^2$). The factor η will be referred to in the following as <u>design factor</u>, since it expresses the ratio between the (design) yield force F_y of the model, and the nominal peak inertia force: A·M.

The function a(t) at the right-hand side is the normalized (unit peak acceleration) random process.

Constant values υ=0,05 and K_2/K_1=0,05 have been used throughout the present numerical calculations, so that the structure's random parameters are represented by the natural period T and the yield force F_y.

ANALYSIS OF THE RESPONSE

The statistics of the response quantity ξ in eq. 1,given the values of the parameters: T and η , depend on the characteristics of the input process a(t).

A numerical simulation procedure, using 10 artificially generated accelerograms, has been carried out to determine the statistics of the response for a convenient range of the parameters T and η.

Mean value curves of the peak response (ductility ratio): μ, as a function of the period T and for various values of the design factor η are presented in fig. 2. As expected, the ductility demand decreases for the larger values of η, and for η>2,5 the (mean) structure's behavior never exceeds the elastic range.

In consideration of the small number of samples used in the simulation procedure, no attempt has been made to fit any particular form of distribution to peak displacement response. Based on previous investigations and on theoretical arguments |3|, |4| an extreme type 1 distribution has been assumed for this variable.

The number of the samples considered allows an estimate to be made of the coefficient of variation of μ: V_μ presented in fig. 3 as function of the period T and for various η.

For any given value of η, the regression line of V_μ as a function of T is nearly horizontal, showing no dependence in the mean of V_μ on T . By calculating the average of V_μ over all the periods T: $V_\mu$av(T) for the various values of η the diagram in fig. 4 is obtained. fig. 4 shows that $V_\mu$av(T) is nearly constant for all the η's, except for very small values (<0,3) for which the average observed dispersion is higher.

The C.O.V.'s of V_μ (not shown in fig. 4, calculated when averaging over T) are also approximately constant along the η's. From the forgoing analysis it can be concluded that V_μ can be taken as constant as far as the dependence on η is concerned, while the variability on T can be accounted for in the same way for all the η's.

The coefficient of variation finally adopted, also shown in fig. 4, corresponds approximately to the average plus one standard deviation of V_μ obtained averaging over the periods.

RELIABILITY MEASURE

A) LEVEL II PROCEDURE

The level II methods, as it is well known, yield safety indexes which, while avoiding complex convolution integrals, represent approximate measures of the probability of attainment of any explicity or implicitly formulated failure condition.

The method requires the definition of the failure boundary in the space of the basic variables, followed by a transformation of the variables from their original to a gaussian distribution.

The safety index β is defined as the minimum distance of the failure boundary from the origin, in the space of the transformed and normalized (zero mean, unit variance) variables.

In the present context, by 'failure' is meant the attainment of a selected level of ductility response: μ. The index β gives then the probability that the response will not exceed such level.

For illustration, a simplified case involving only two random variables is presented in fig. 5.

A particular design situation is considered, with T=0,6 sec and η= =1,5 , so that the r.v. present are the peak ground acceleration A and the structure's response μ. The boundary is defined by the condition: μ=3, and the minimum distance is found to be: β=1,86 (to which corresponds a probability of 3,14 10^{-2}). The coordinates of the 'checking point' measure the probability of the two r.v. considered when the limit state is attained in its most probable point.

The level II safety analyses have been performed by means of the computer program described in |2|. The program can deal with any explicitly or implicitly defined failure boundary (g-function).

At each step of the searching procedure for the minimum distance the original g-function (and its derivatives) is modified, as a consequence of the transformation of the original basic r.v. into normal ones.

B) RESULTS

The direct results of the safety check analyses are of the form illustrated in figs. 6 and 7. The curves in these figures give the index β as a function of the mean natural period $\bar{T}$, for various values of the mean design factor $\bar{\eta}$; all cases refer to a failure condition defined by $\mu=3$.

In fig. 6 both the seismic input (the peak acceleration A plus the random process) and the structural parameters (F_y and T) are considered as random, with the indicated types of distribution and values of the coefficients of variation.

Fig. 7 compares some of the curves in fig. 6 with the corresponding ones obtained for the case of deterministic structural behavior ($V_{F_y}=V_T=0$). It is seen that consideration of the randomness of the structure's behavior reduces the overall reliability increasingly with increasing $\bar{\eta}$ and natural period $\bar{T}$.

With the particular values of the statistical parameters adopted ($V_{F_y}=0,15$, $V_T=0,20$), which are not unrealistic, the effect can be significant even in the normal range of designs.

For instance, for $\bar{T}=1,0$ sec and $\bar{\eta}=1,5$, the value of β is reduced almost by a half point, which corresponds roughly to a reduction of one order of magnitude on P_S.

All the curves in figs. 6 and 7 steadily increase with increasing $\bar{T}$, thus indicating greater reliability for longer period structures. This is partly due to the assumed frequency content of the artificial accelerograms, which were choosen to represent hard soil conditions, and thus did not include significant power in the low frequency range.

The final presentation of the above results is obtained rearranging the diagrams in fig. 6 by drawing horizontal sections through them. Each line collects the $\bar{\eta}$ values versus period $\bar{T}$ to which corresponds a constant level of reliability. In fig. 8 the new curves are reported: they furnish the mean design factor, i.e. the ratio between the mean yield force to the mean peak ground acceleration, required in order that structure's response does'nt exceed (with a selected degree of reliability) a specified level of ductility, as a function of the mean natural period of the structure.

CONCLUSIONS

A procedure has been illustrated for carrying out level II safety checks of non-linear random s.d.o.f. structures under random dynamic seismic excitation. The procedure is general with respect to the structural model actually used and to the probabilistic definition of the variables. Some results are presented, with comments on the influence of the randomness of the structure's behavior on the overall reliability.

Finally, the procedure has been employed to construct iso-reliable design response spectra. Each spectrum corresponds to a specified level of reliability against the exceedance of a limit-state defined in terms of maximum ductility response. With different sets of spectra available for different sets of statistical parameters and allowable ductilities, their use for design will be straightforward and general. The required input consists on the calculated mean natural period of the structure, and on the mean maximum peak ground acceleration during a reference period (ex. fifty years) relative to the site of interest. The output is the average strength the structure must be designed with.

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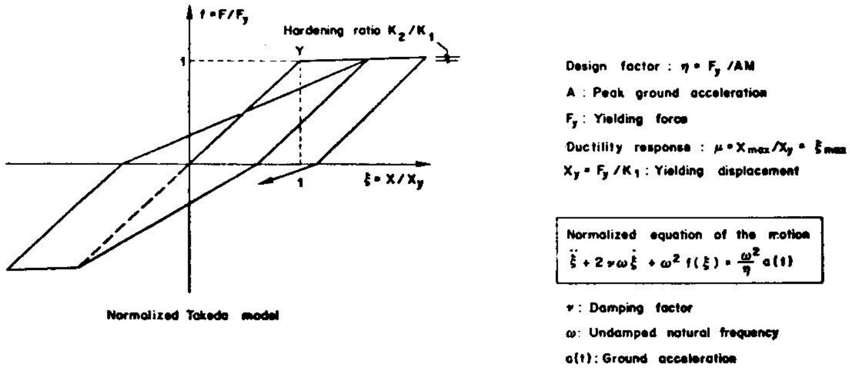
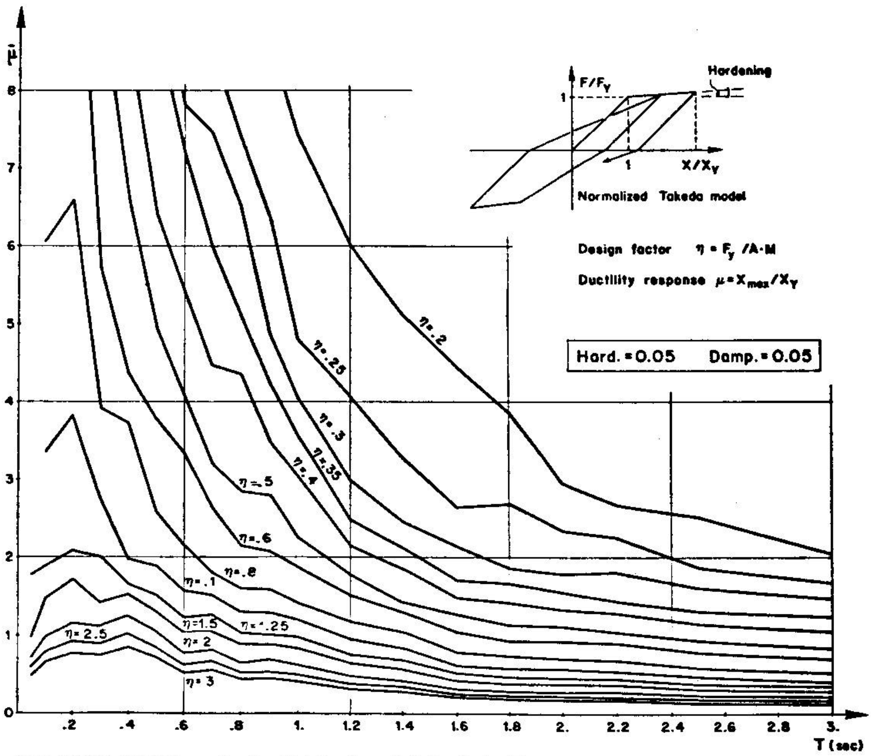
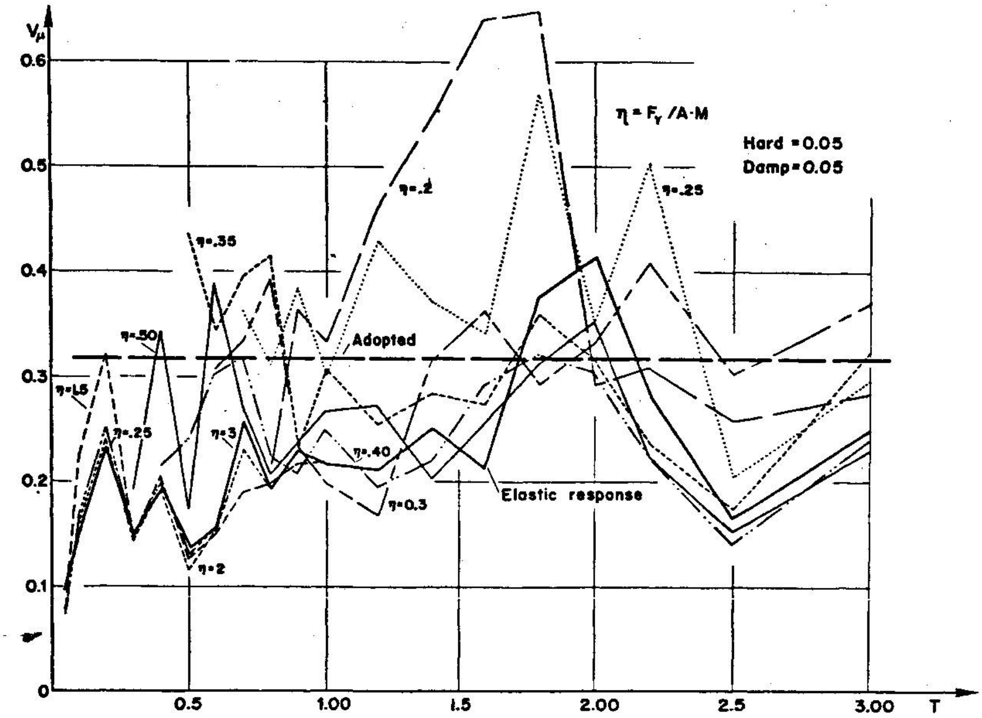


Fig. 1



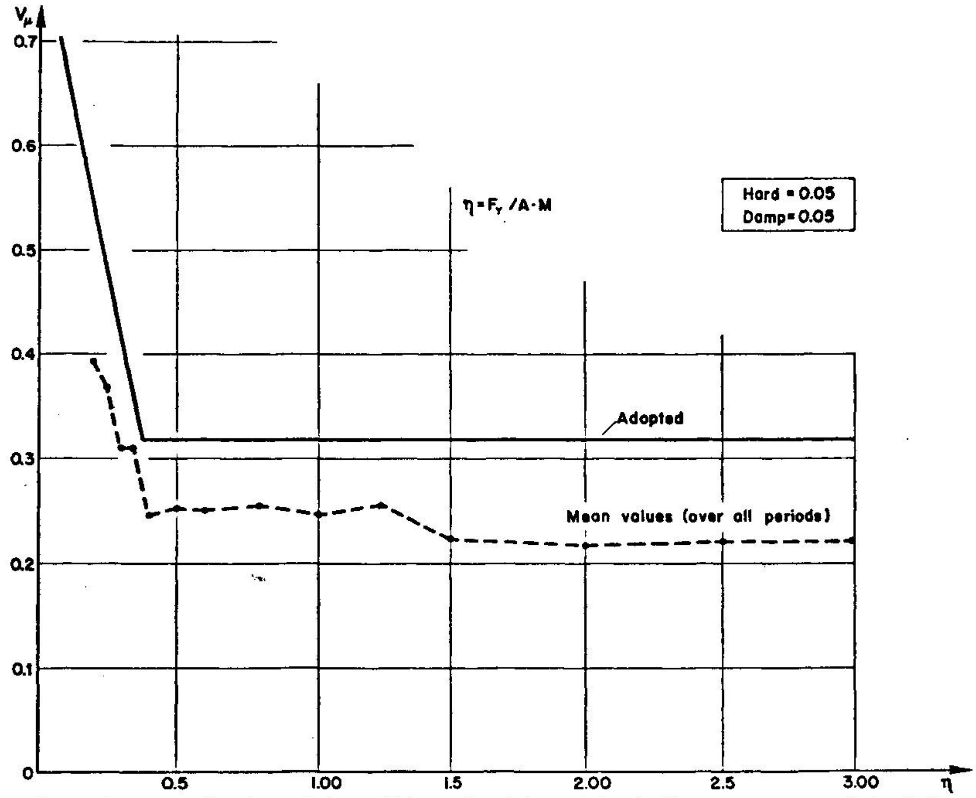
Mean ductility response spectra for several values of design factor η

Fig. 2



Coefficients of variation of the ductility response spectra for various values of the design factor

Fig. 3



Mean values over all periods of the coefficients of variations of the ductility response versus design factor η

Fig. 4

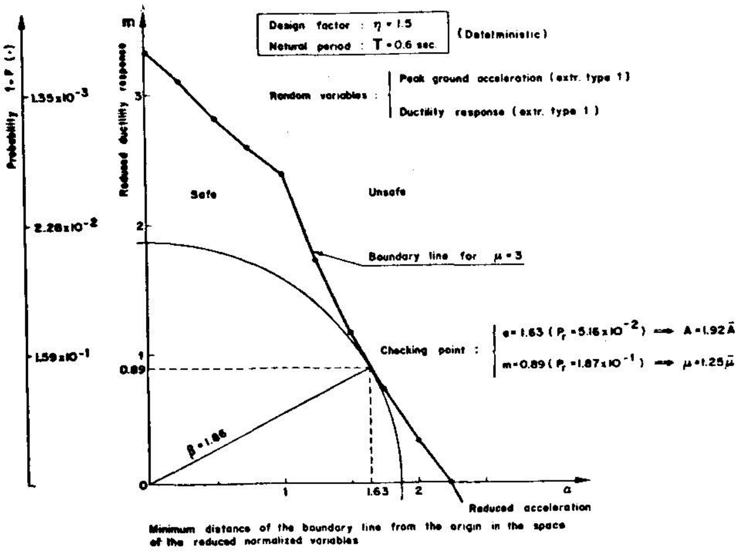
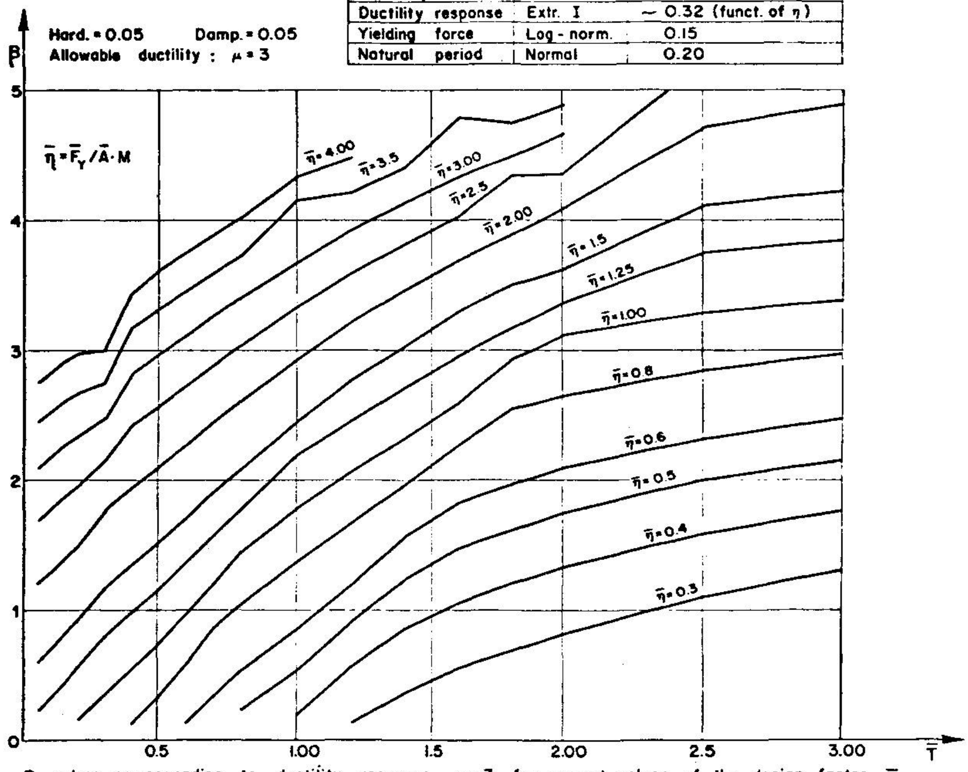


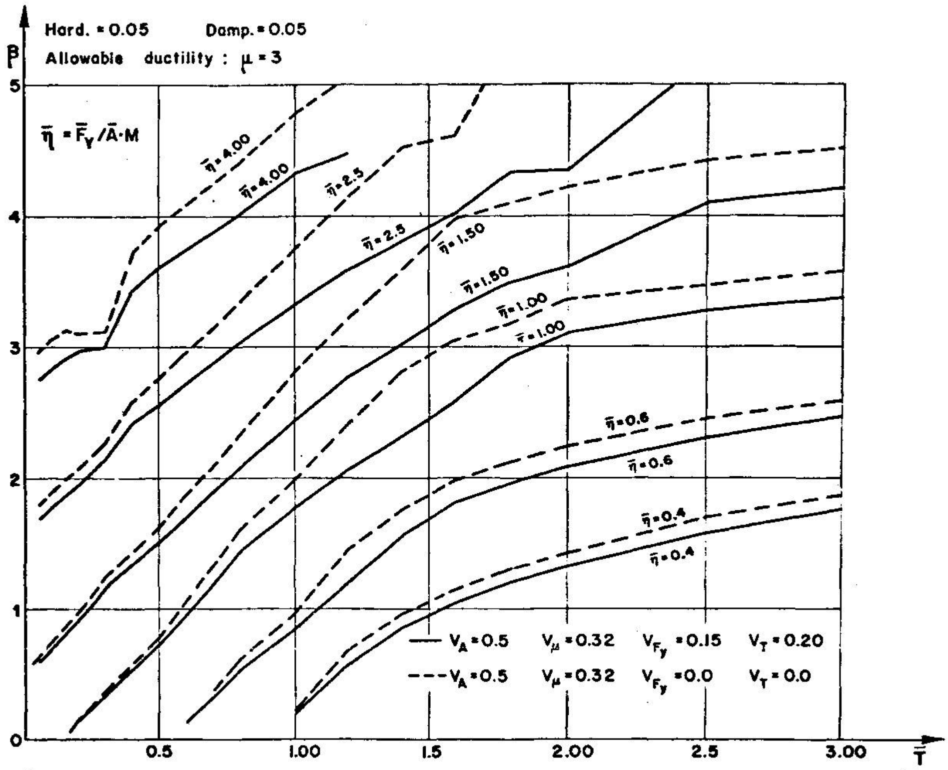
Fig. 5

Random variables	Distribution	Coefficient of variation
Ground peak accel.	Extr. I	0.5
Ductility response	Extr. I	~ 0.32 (funct. of η)
Yielding force	Log - norm.	0.15
Natural period	Normal	0.20



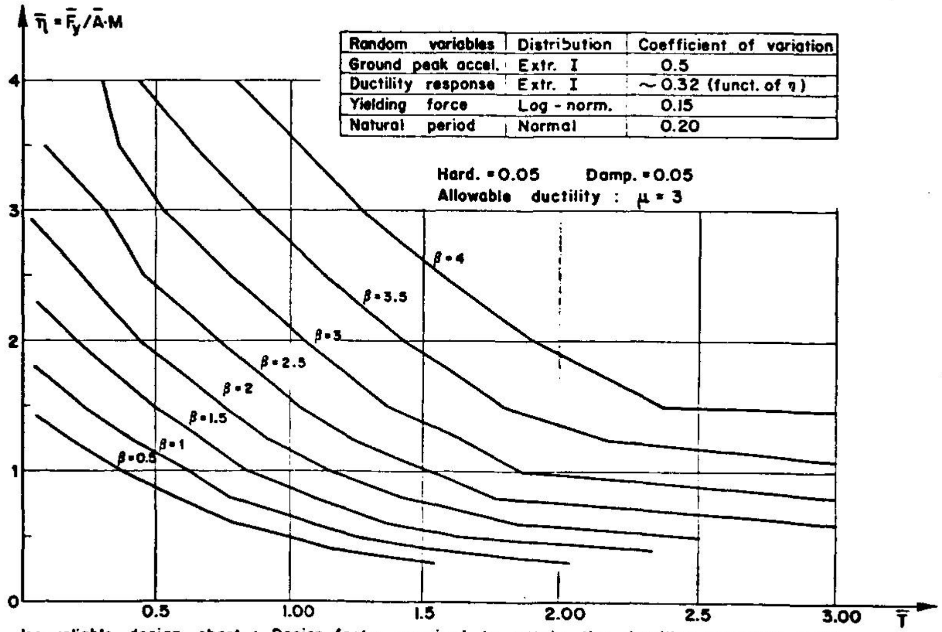
β values corresponding to ductility response $\mu = 3$ for several values of the design factor $\bar{\eta}$

Fig. 6



β values corresponding to ductility response μ = 3 for several values of the design factor η̄

Fig. 7



Iso-reliable design chart : Design factor required to attain the ductility response μ = 3 for several reliability levels

Fig. 8