# TOWARDS AN IMPROVEMENT ON LOADING CRITERIA

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## SUMMARY

A different approach to loading criteria which involves the notion of combined strength-ductility requirements, and thus relates to energy balancing during seismic motion, is presented.

This approach leads to a design philosophy where one can have in mind a probabilistically defined structural response and damage estimation.

Data gathered from nonlinear response of single degree of freedom systems to real accelerograms is shown, to substantiate the methodology, allowing immediate design applications.

## INTRODUCTION

Today's design methods for earthquake loading specifications, involve the idea that using the displacement ductility concept, one can use results from an elastic analysis to infer about nonlinear response. This notion is based on the assumption that there will be no "large" ductility demands, which can not be guaranteed a priori.

It is desirable that any approach to the loading definition could be:

a) Consistent both with strength and ductility requirements or in other words more related to the energy aspects of structural response.
b) Probabilistic by nature, including in the assessment of damage.
c) Easy to implement in design practice.

In a similar way to the methodology followed to develop the response spectrum idea, the response of nonlinear single degree of freedom (SDOF) systems was studied, to recognize patterns of structural behavior in some of the response parameters. It is believed that this may provide a basis for a different design methodology.

## MODEL DESCRIPTION

Several SDOF systems were subjected to a set of accelerograms to study the response in terms of required peak and accumulated ductility, energy dissipation and peak displacements distribution.

The elasto-plastic system with different initial elastic periods and yield strength ratios (YSR) was used. Viscous damping was kept constant to 5% of the critical (fig. 1a).

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The values used for the yield strength ratio were 0.25, 0.50, 0.75 and 1.00 where $YSR = \frac{\text{Horizontal acceleration to produce yielding}}{\text{Maximum peak ground acceleration}} = \frac{Fy/M}{PGA}$

Comparing YSR values with the current SEAOC lateral force design requirements, (YSR)(PGA)(M)=ZIKCSW or (YSR)=ZIKCS(g/PGA). For common values ( Z=1.0, I=1.0, K=0.67, C=0.07, S=1.5) and for PGA=0.20g, the correspondent YSR value is 0.35.

The initial periods were selected to cover the spectrum range of interest, 0-5 seconds, with closer intervals for lower periods.

The modified Clough stiffness degrading system ( fig 1b) and the bilinear system with k1/k0 = 0.1 (fig 1c) were used with some of the accelerograms to compare the results with the ones obtained for the E.P. system.

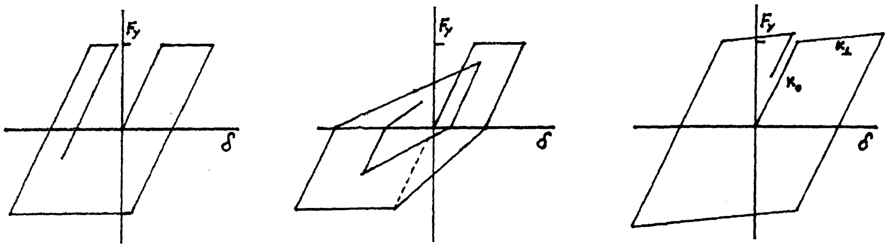


Fig. 1 a)Elasto-plastic b)Stiffness degrading c)Bilinear

The program used, performs dynamic analysis of SDOF systems subjected to arbitrary support accelerations, by means of a direct time step-by-step linear acceleration integration.

The 36 accelerograms used, are the CALTEC corrected accelerograms constant on TABLE I (ref 1), which were catalogued according to local site conditions (soil or rock) and type of probability distribution for the ranked peak values in the accelerogram ( Exponential, Rayleigh or Weibull ).

| | | |
|---|---|---|
| SOIL | Exponential | A001(S00E), A001(S90W), A004(N21E), A004(S69E), A008(N79E), B024(S90W), B029(N66E), B032(N86W) |
| | Rayleigh | A008(N11W), B024(S00W), B029(N04W), B033(N65E), C048(N00W), D058(S00W), D065(S90W), V315(WEST) |
| | Weibull | A009(N46W), B021(N82W), B034(N05W), B034(N85E), B035(N50E), B035(N40W), U299(N45E) |
| ROCK | Exponential | A015(N10E), B025(S90W), B040(N33E), D056(N69W), G106(S90W) |
| | Rayleigh | A015(S80E),B025(S00W) |
| | Weibull | B037(N65W), B037(S25W), C041(S16E), C041(S74W), D056(N21E), W334(S25W) |

TABLE I - CALTEC accelerograms used

According to DeHerrera (ref 2) the kind of distribution for the peak values in the accelerogram, has geophysical interpretation.

Weibull-Exponential distributions are likely to occur for high frequency motions which are more probable for small source to site distance, short segment rupture, short rise time, high stress drop and orientation in the fault rupture direction. Rayleigh distributions are related to low frequency motions.

## OBJECTIVES

Let us assume that the initial period of the structure and the equivalent viscous damping are known within a confidence range and that the structural behavior can be approximated by any of the structural models presented (Elasto-plastic, bilinear or stiffness degrading). Also the expected PGA and local site conditions are well defined.

The loads the system is to be designed for, can then be specified in terms of the yield strength ratio, which is the designer's choice, depending on the previously mentioned seismic hazard and structural parameters and on the type and system of construction.

It is then the designer's task to verify that the system can perform in terms of serviceability the loading it was designed for. Usually this is performed by means of structural detailing, so that the structure has the required capacity to dissipate the energy input to it. This verification can be done by means of damage estimation, that has been related to peak and accumulated ductility capacity (ref 3).

It is the purpose of this study to show that there are patterns for the way a certain structure with known characteristics subjected to a given earthquake dissipates the energy. This will be done by estimating the required cyclic ductility, accumulated ductility, number of non-linear excursions and peak displacements as well as their values, by means of a probability distribution (which relates to the way energy is dissipated through hysteretic damping in the time domain) and recognition of the relative importance of the two alternative ways of energy dissipation (hysteretic and viscous damping).

This estimation will be done for each initial period and yield strength ratio allowing the construction of spectra of Hysteretic Energy Capacity.

Parallel studies can be found on Refs. 4,5,6 and 7.

## RESPONSE STATISTICS

As mentioned, the ranked peak values of the ground motion acceleration follow an exponential type of distribution (Exponential, Rayleigh or Weibull) (ref 2). From the theory of random vibrations assuming a narrowband stationary process, one knows that the peaks above a certain level are Rayleigh distributed. On the other hand, the input signal itself is already a filtered response of the original source motion, and so one would expect the structural response signal to have the same frequency dependent kind of distribution as observed for the input itself. Indeed this is the case.

Systems with low initial periods tend to have the peak displacements Weibull distributed and increasing period (decreasing frequency) changes the distribution to Exponential and Rayleigh. Figs 2a and 2b show a plot of the ranked peak displacement distribution of the response to the N S component of the 1940 El Centro earthquake for two different initial periods.

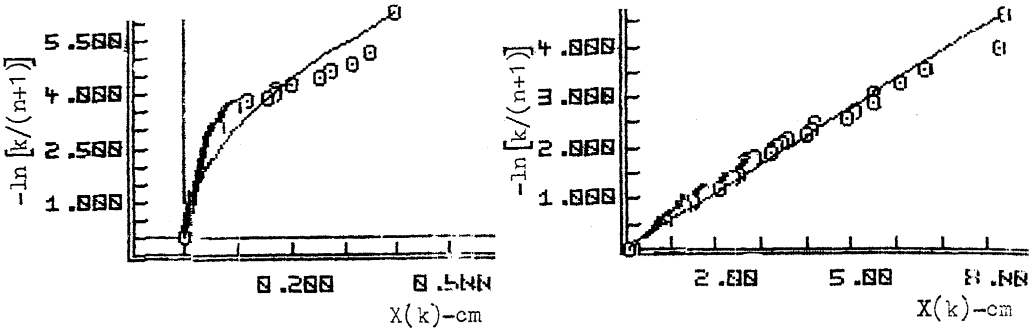


Fig 2 - Exponential plot for the peak displacements, YSR=0.50
a) T=0.1 b) T=1.0

Also the type of initial distribution for the peak accelerations affects the response distribution. Weibull inputs are likely to produce Rayleigh outputs for a wider range of large periods, and the opposite also applies for small periods. This suggests that the type of distribution is dependent not only on the frequency of the system but in the frequency of the system as related to the frequency of the input. Fig 3 shows the observed types of distributions for different sites (inputs) and system periods. Transition zones where both two distributions match the data, justify the superposition for certain period ranges.

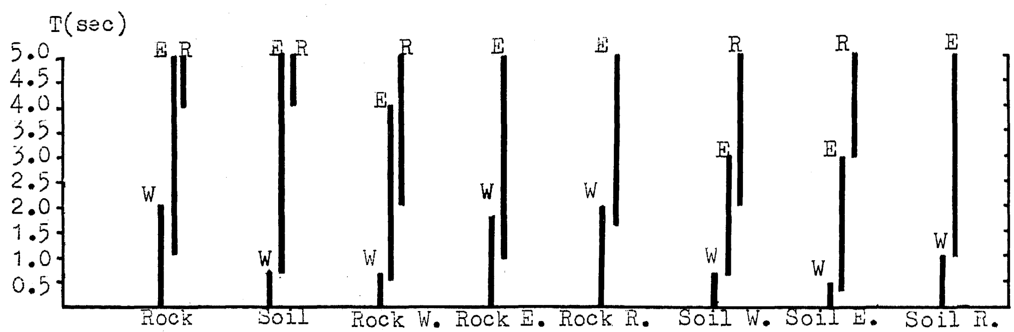


Fig 3 - Types of distribution for the peak displacements.
W- Weibull E- Exponential R- Rayleigh

The distribution of the inelastic excursions (inelastic component of the peak displacements) tends to be Weibull-Exponential which is in accordance with other analitical works (ref 7).

Known the ductility requirements in terms of the largest peak or the total energy dissipated, the number of inelastic excursions (n) and the type of distribution, it is possible using extremal statistics to know what is the expected value, deviation and probability of exceeding a certain value, for the i th response peak (i=1,n). The usefulness of this to damage estimation is obvious. Fig. 4 shows the expected number of inelastic excursions for a rock site.

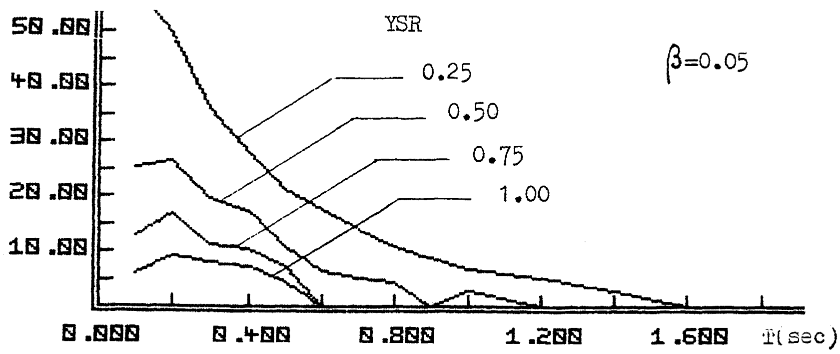


Fig. 4 - Expected number of inelastic excursion, rock site.

## DUCTILITY REQUIREMENTS AND HYSTERETIC ENERGY CAPACITY

Some authors have related damage to excessive peak ductility demand, others to the accumulation of all the inelastic excursions (accumulated ductility demand). According to the obtained results these two measures of ductility are somehow correlated, because one is proportional to the largest peak, and the other to the sum of the peaks, both of them easily obtained given the expected number of peaks and the $1/\lambda$ parameter of the peak distribution.

Figs 5a and b show the peak cyclic ductility demand (maximum positive to negative) for elasto-plastic systems, respectively for soil and rock sites, as a function of the initial period and the yield strength ratio. The values shown are the average plus one standard deviation for all the records used.

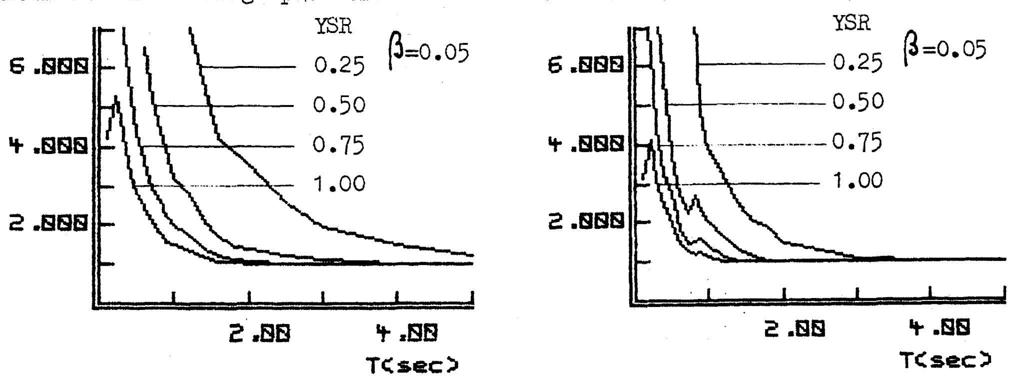


Fig 5 - Peak cyclic ductility demand
a) soil site b) rock site

From there the following conclusions can be made:

a)Soil sites require larger ductility capacity then rock sites for the same PGA and YSR.

b)Required ductility capacity decreases with period increase.

c)For large YSR the response is allways elastic except for small periods.

d)For small YSR the response is allways inelastic except for very high periods.

e)Low YSR and small periods tend to produce excessive ductility demand.

Table II shows the period values for which the system remains elastic and exceeds a peak cyclic ductility demand of 6.

| | | ELASTO-PLASTIC | | | | | | | | ST. DEGR. | | BIL. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | YSR | Rock | Soil | R.E. | R.R. | R.W. | S.E. | S.R. | S.W. | R.E. | S.E. | S.E. |
| a) System remains elastic | 1.00 | >1.0 | >2.0 | >1.0 | >0.6 | >0.7 | >1.4 | >2.0 | >1.8 | >1.0 | >1.4 | >1.4 |
| | 0.75 | >1.4 | >3.0 | >1.2 | >0.7 | >1.6 | >2.5 | >3.0 | >2.0 | >1.2 | >1.8 | >2.5 |
| | 0.50 | >1.8 | >4.0 | >1.6 | >0.8 | >2.0 | >4.0 | >4.0 | >3.0 | >1.6 | >4.0 | >4.0 |
| | 0.25 | >4.0 | -- | >4.0 | >1.2 | >2.5 | -- | -- | >5.0 | -- | -- | -- |
| b) Ciclic ductility >6 | 1.00 | -- | -- | -- | -- | -- | -- | <0.2 | -- | -- | <0.2 | -- |
| | 0.75 | <0.2 | <0.3 | <0.2 | <0.1 | <0.2 | <0.4 | <0.3 | <0.2 | <0.2 | <0.4 | <0.4 |
| | 0.50 | <0.3 | <0.6 | <0.4 | <0.2 | <0.3 | <0.5 | <0.6 | <0.6 | <0.4 | <0.5 | <0.5 |
| | 0.25 | <0.8 | <1.2 | <0.8 | <0.4 | <0.6 | <1.2 | <1.2 | <1.0 | <0.8 | <1.0 | <0.9 |

TABLE II - Initial periods for which the system a) remains elastic and b) exceeds a ciclic ductility demand of 6. T(sec).

Figs 6a and b show the normalized hysteretic energy demand (sum of the yield excursions normalized to the yielding displacement). Average plus one standard deviation values are presented.

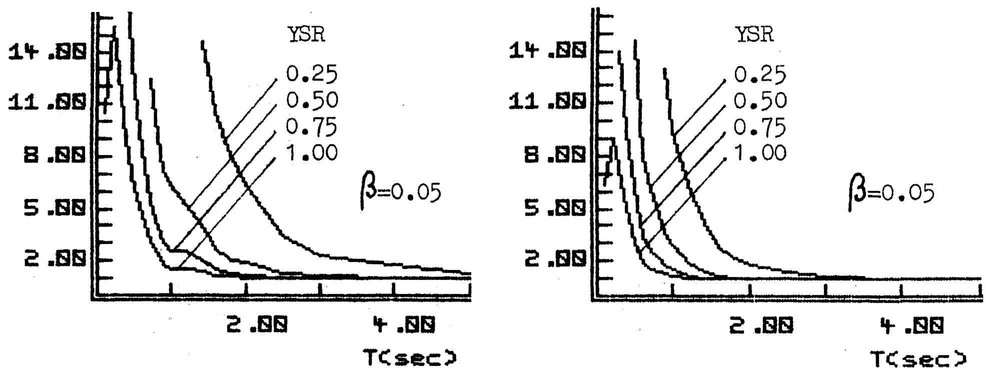


Fig 6 - Normalized hysteretic energy
a) soil site
b) rock site

All the previous conclusions apply here, showing the correlation between the two measures of inelastic demand.

The previous results were presented irrespectively of the type of distribution for the input earthquake. Figs 7a and b show for YSR=0.50 the ductility demand respectively for soil and rock sites but dependent on the type of input distribution.

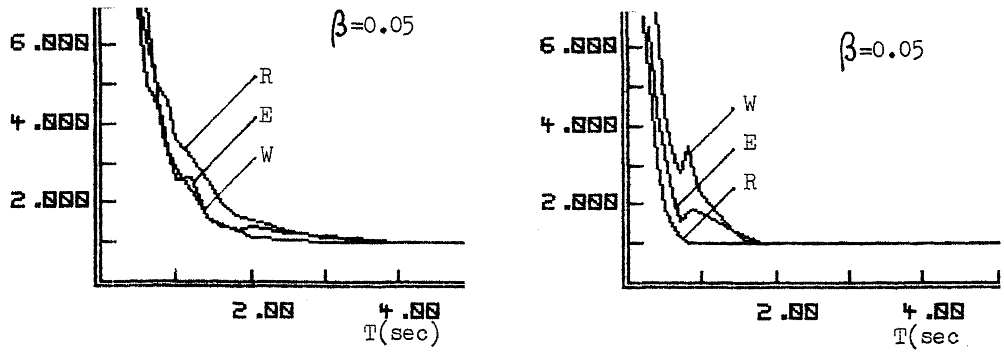


Fig 7 - Peak cyclic ductility demand, E-Exponential R-Rayleigh W-Weibull
a) soil site b) rock site

From these figures one can observe that:
a)For rock sites larger ductility demand comes from Weibull distributed input.
b)For soil sites the difference is not as marked, but higher ductility demand comes from Rayleigh distributed input.

To compare with the elasto-plastic, the other two models where subjected to the set of soil exponential records. Figs 8a and b show the comparison in terms os required ductility for systems with YSR respectively 0.75 and 0.50 .

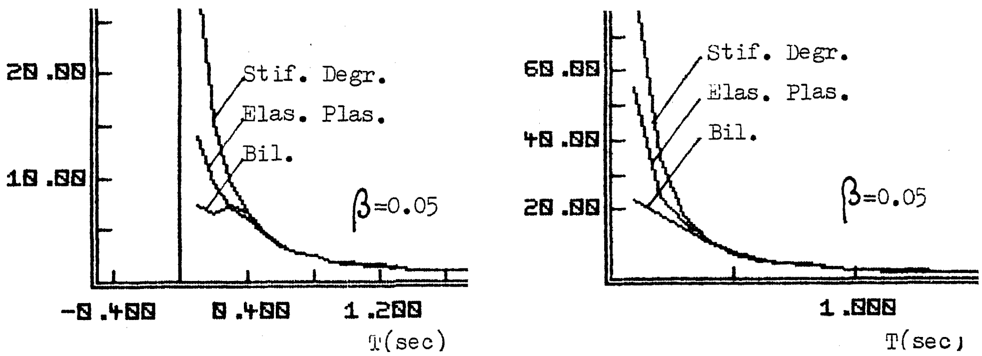


Fig 8 - Required peak cyclic ductility, soil-exponential
a) YSR=0.75 b) YSR=0.50

It is remarkable how the three systems behave so similarly. Only for very small periods (less than 0.4 sec) the stiffness degrading system dissipates more energy in hysteresis, than the elasto-plastic and bilinear respectively.

Figs 9a and b compare the energy dissipated by hysteretic and viscous damping. The remainder up to 100% is stored under strain and kinetic energy and dissipated by viscous damping after the end of the quake.

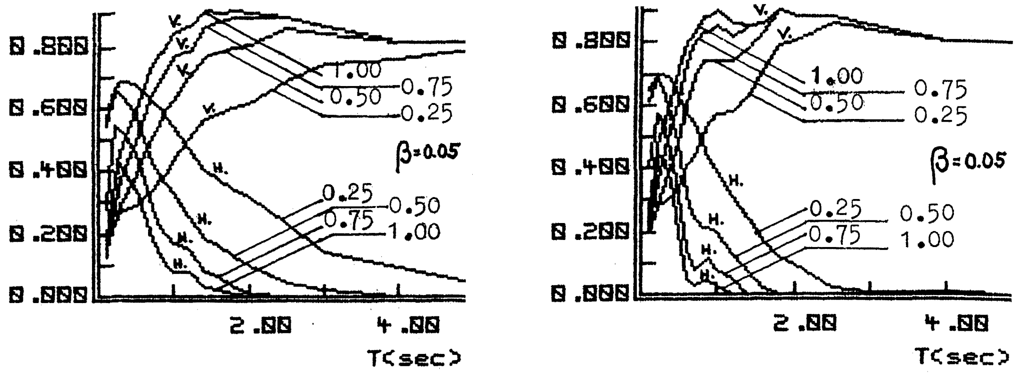


Fig 9 - Percentage of energy dissipated by hysteretic and viscous damping
a) soil site b) rock site

## CONCLUSIONS

Future research is necessary to expande this methodology to other structural models ( more stiffness degrading models, inclusion of gravity effects, etc), and to a wider range of some of the parameters like equivalent viscous damping. Use with multidegree of freedom structures needs to be carefully thought, but the fact that in most of the cases the great part of the dissipated energy is associated with the first mode (ref 4) allows it's immediate applicability. It is felt that to think structural analysis in terms of energy requirements and a probabilistic damage estimation is a more genuine approach.

## ACKNOWLEDGMENTS

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