the accuracy of the assumptions made and parameters used may lead to a false sense of security. If the specifications are unduly conservative the design may be forced into a type that is strong but less ductile than is desirable. It is difficult to avoid differences in the degree of conservatism among different types of structures, and in some cases it is undesirable to do so. Some materials by their nature, including their variability or lack of adequate control of properties, may require a greater factor of safety than other materials the properties of which are more accurately determinable and controllable. The margin between incipient failure and complete collapse may differ for different materials and may therefore involve a difference in the factor of safety required in the design. It is desirable, in the development of the basis for a performance criterion, that the designer's approach not be too greatly constrained. For example, it may be unwise to prescribe limits for both strength and ductility in such a way that the balance between the two cannot be adjusted to take account of new material properties or new structural types as they are developed. A trade-off between ductility and strength should be available in the methods that are permitted, so as to achieve economy without the sacrifice of safety. But whether one is interested in achieving strength or ductility, or both, the materials have to be used in an appropriate fashion, and adequate methods of inspection and control of construction are needed to insure that their use is proper.

The most desirable type of design code or specification is one which puts the least restrictions on the initiative, imagination and innovation of the designer. Such a code might involve only criteria for: (1) the loading or environment; and (2) the level of response, the stresses and deformation, or the performance of the structure under the specified loading or environmental conditions. Such an approach need not, and preferably should not, indicate how the designer is to reach his objective, provided he can demonstrate -- through documentation of adequacy -- that he has achieved a structural capability to resist the specified environmental conditions. This approach is generally the one now used for the design of nuclear reactor power stations. Experience over the past several years in approaching seismic design criteria in this way has indicated a number of problems, but also has been reasonably successful in avoiding constraints due solely to the specifications themselves, although there have been constraints based on the environmental conditions and the stress and deformation levels allowed.

## EARTHQUAKE HAZARD

An earthquake that has a relatively high probability of occurrence is appropriately considered as a loading for which the design must provide in such a way that the cost of the minor repairs required is not excessive. Major strengthening of a structure to resist intense forces is expensive, and the cost of such design provisions must be weighed against the possible cost of repairs in order to decide whether the additional design strength or ductility is economically justified.

Design standards should consider the dual threshold values of permissible damage associated with relatively small or common is cracked, where the elements on the two sides of the crack can move relative to one another with the absorption of energy at the faying surface, will also absorb considerable energy. On the other hand, a homogeneous solid structure or a welded steel structure has relatively small amounts of lost energy, and a concrete beam before cracking has a relatively small amount of energy losses except those within the material itself.

The importance of damping is indicated by the fact that the dynamic response of a structure in an earthquake may be affected to as great a degree by damping as by almost any other parameter. This is especially true in those instances when long sustained nearly harmonic motions are involved. It is because of this reason that great difficulties are found with design specifications in which the design forces do not properly or realistically reflect the differences in damping associated with different materials, different types of framing, and different levels of allowable deformation and stress.

## DESIGN SPECTRUM - ELASTIC

In either analysis or design for earthquake resistance it is convenient to use the concept of the response spectrum. A response spectrum developed to give design coefficients is called a "Design Spectrum". Detailed descriptions of the response of simple elastic systems, or more complex structure and elements, subjected to dynamic loading and especially to seismic loading, are given in Refs. 1-4.

For any given area or site, estimates might be made of the maximum ground acceleration, maximum ground velocity, and maximum ground displacement. The lines representing these values can be drawn on the tripartite logarithmic chart of which Fig. 1 is an example. The lines showing the ground motion maxima in Fig. 1 are drawn for a maximum ground acceleration, a, of 1.0g, velocity, v, of 48 in/sec, and displacement, d, of 36 in. These data represent motions more intense than those generally considered for any postulated design earthquake hazard. They are, however, approximately in correct proportion for a number of areas of the world, where earthquakes occur either on firm ground, soft rock, or competent sediments of various kinds. For relatively soft sediments, the velocities and displacements might require increases above the values corresponding to the given acceleration as scaled from Fig. 1. It is not likely that maximum ground velocities in excess of 4 to 5 ft per second are obtainable under any circumstances.

As part of a continuing study made recently by us for the U.S. Atomic Energy Commission and referred to in Ref. 5, values were determined for the horizontal and vertical directions of excitation for various degrees of damping. The current interim amplification levels for 50, 75, and 90 percentile levels of horizontal response are presented in Table 1. A value of 90 percentile means that one could expect 90 percent of the values to fall at or below that particular amplification.

The construction of the elastic design spectra is straightforward as illustrated in Fig. 1. For more detailed discussion of such constructions see Refs. 1-4.

factors. For frequencies between about 2 up to about 6 cps, the best relationship appears to be to equate the energy in the various curves, or to say that energy is preserved, with a corresponding relationship between deflections and accelerations or forces. There is a transition region between 6 and 20 to 30 cps, depending on the damping ratio. Above 20 to 30 cps, the force or acceleration is nearly the same for all ductility ratios.

## DESIGN SPECTRUM - INELASTIC

To use the design spectrum to approximate inelastic behavior, the following suggestions are made. In the amplified displacement region of the spectra, the left-hand side, and in the amplified velocity region, at the top, the spectrum remains unchanged for total displacement, and is divided by the ductility factor to obtain yield displacement or acceleration. The upper right-hand portion sloping down at $45^{\circ}$, or the amplified acceleration region of the spectrum, is relocated for an elasto-plastic resistance curve, or for any other resistance curve for actual structural materials, by choosing it at a level which corresponds to the same energy absorption for the elasto-plastic curve as for an elastic curve for the same period of vibration. The extreme right-hand portion of the spectrum, where the response is governed by the maximum ground acceleration, remains at the same acceleration level as for the elastic case, and therefore at a corresponding increased total displacement level. The frequencies at the corners are kept at the same values as in the elastic spectrum. The acceleration transition region of the response spectrum is now drawn also as a straight line transition from the newly located amplified acceleration line and the ground acceleration line, using the same frequency points of intersection as in the elastic response spectrum.

In all cases the "inelastic maximum acceleration" spectrum and the "inelastic maximum displacement" spectrum differ by the factor $\mu$ at the same frequencies. The design spectrum so obtained is shown in Fig. 5.

The solid line $DVAA_o$ shows the elastic response spectrum. The heavy circles at the intersections of the various branches show the frequencies which remain constant in the construction of the inelastic design spectrum.

The dashed line $D'V'A'A_o$ shows the inelastic acceleration, and the lines $DVA''A''_o$ shows the inelastic displacement. These two differ by a constant factor $\mu = 5$ for the construction shown, but A and A' differ by the factor $\sqrt{2\mu - 1} = 3$, since this is the factor that corresponds to constant energy for an elasto-plastic resistance.

Of course, the elasto-plastic or other inelastic response spectra can be used only as an approximation for multi-degree-of-freedom systems.

## COMBINED AND SPECIAL EFFECTS

Since the ground moves in all three directions in an earthquake, and even tilts and rotates, consideration of the combined effects of all these motions must be included in the design of important structures.

TABLE 1. VALUES OF SPECTRUM AMPLIFICATION FACTORS

| Percentile | Damping, % | Amplification D | V | A | Faring frequency, hertz |
|---|---|---|---|---|---|
| 50 | 0.5 | 1.97 | 2.58 | 3.67 | 40 |
| | 2.0 | 1.68 | 2.06 | 2.76 | 30 |
| | 5.0 | 1.40 | 1.66 | 2.11 | 20 |
| | 10.0 | 1.15 | 1.34 | 1.65 | 20 |
| 75 | 0.5 | 2.66 | 3.41 | 4.65 | 40 |
| | 2.0 | 2.24 | 2.68 | 3.36 | 30 |
| | 5.0 | 1.83 | 2.10 | 2.48 | 20 |
| | 10.0 | 1.47 | 1.66 | 1.89 | 20 |
| 90 | 0.5 | 3.28 | 4.16 | 5.53 | 40 |
| | 2.0 | 2.74 | 3.23 | 3.90 | 30 |
| | 5.0 | 2.21 | 2.51 | 2.82 | 20 |
| | 10.0 | 1.75 | 1.94 | 2.11 | 20 |

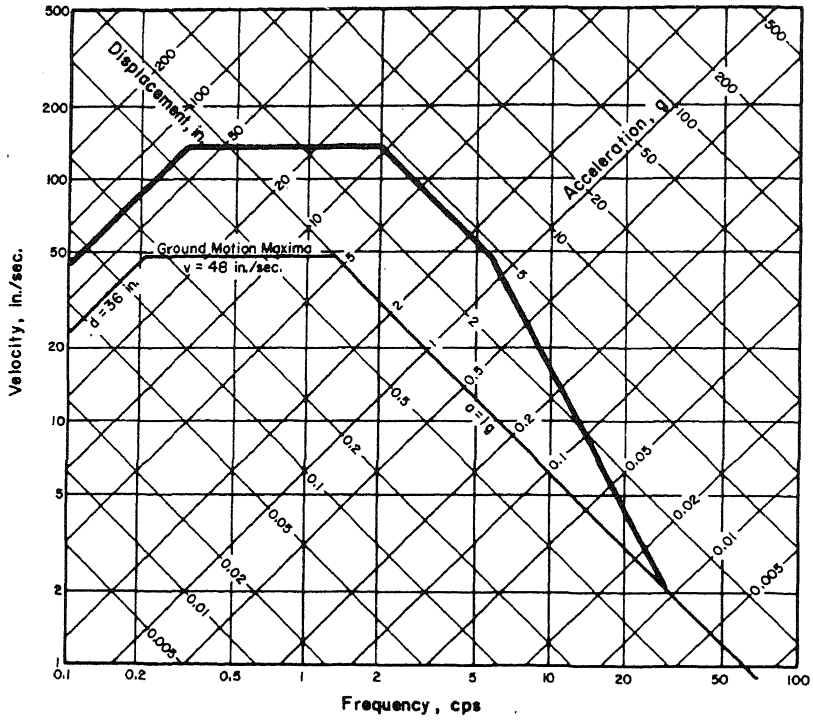


FIG. 1 BASIC DESIGN SPECTRUM NORMALIZED TO 1.0 g FOR 2 % DAMPING, 90 PERCENTILE LEVEL

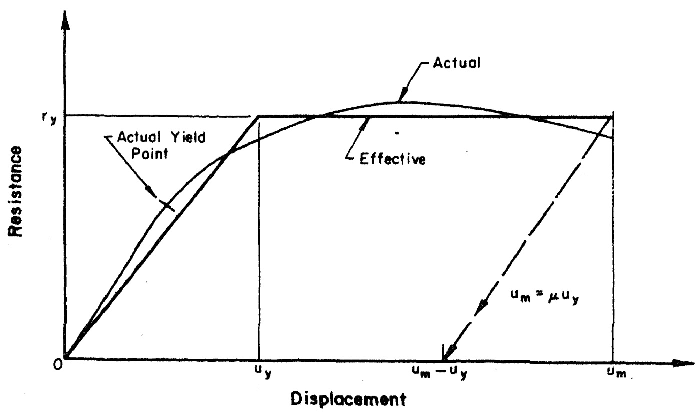


FIG. 2 RESISTANCE — DISPLACEMENT RELATIONSHIP

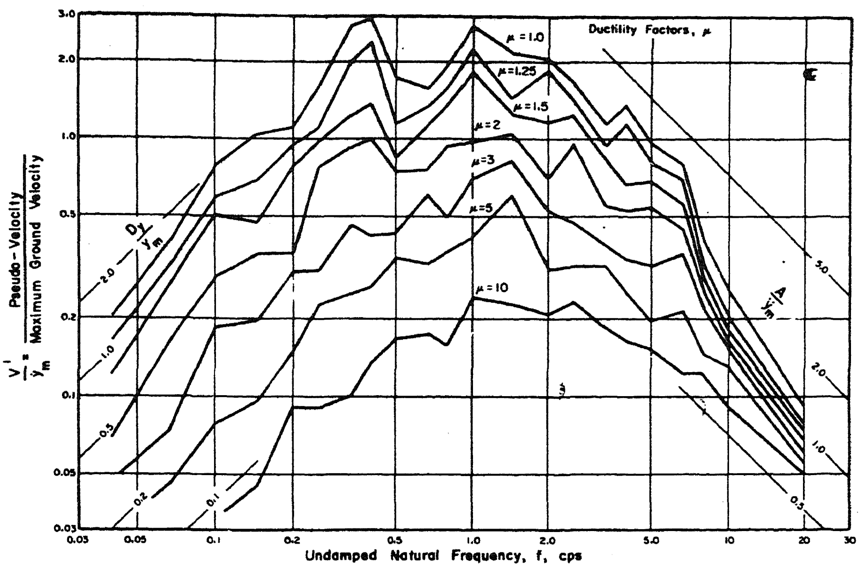


FIG. 3 ACCELERATION SPECTRA FOR ELASTOPLASTIC SYSTEMS, TWO PERCENT CRITICAL DAMPING, EL CENTRO 1940 EARTHQUAKE

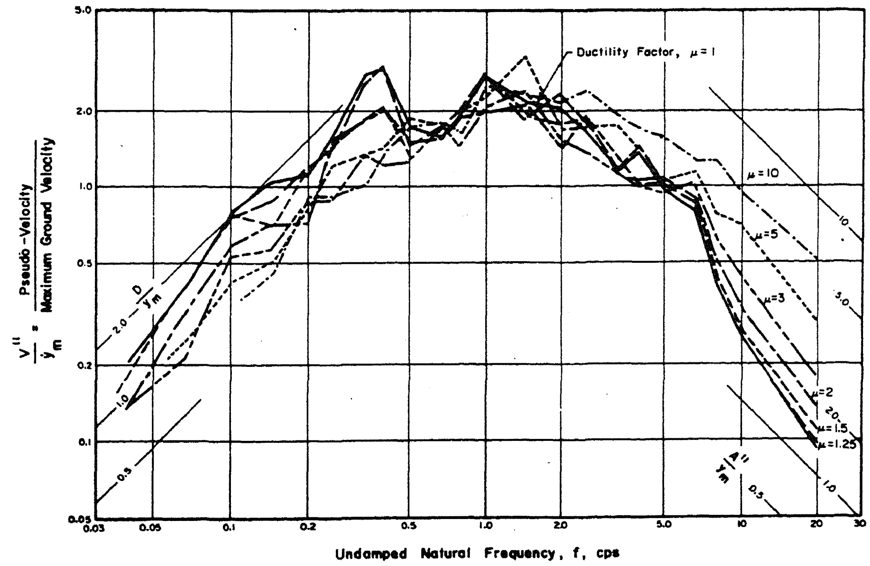


FIG. 4 TOTAL DISPLACEMENT SPECTRA, ELASTOPLASTIC SYSTEMS, TWO PERCENT CRITICAL DAMPING, EL CENTRO 1940 EARTHQUAKE

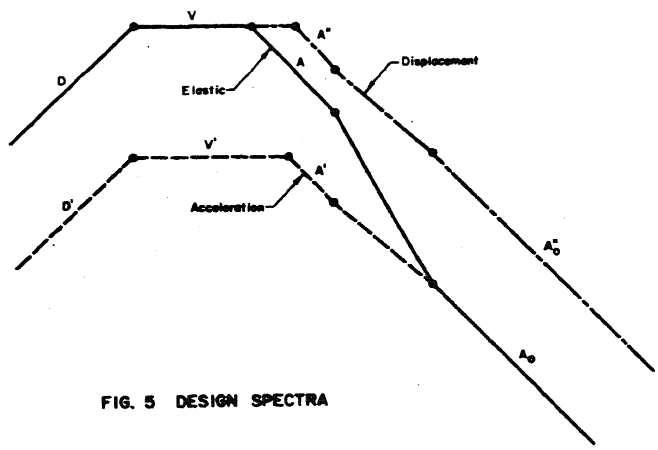


FIG. 5 DESIGN SPECTRA

## DISCUSSION BY F. NOVOA M. (I)

to the Paper N° 283, A Rational Approach to Seismic Design Standards for Structures, by N.M. Newmark and W.J. Hall.

A) This paper must be heartily welcome as a most important contribution and guide to the field. It is not by chance, nor for reasons of vogue that the method by the authors (1), as soon as published, was adopted by us as the basis for our specifications for the high voltage electrical equipment (see 2 and 3).

We emphatically agree with the authors on the basic function of the Specifications, as this is particularly important in our special field, wher the tendency to develop equipment for different conditions on standard elements has been established for the last one or two decades.

We have been trying to guide the composition and design of earthquake resistant equipment through a simplified analysis which has brought satisfactory explanation to the observed facts of experience with the equipment up to 220 kV (see 3 and 4).

In this simplified analysis and the results of several recent studies (se, by ex., 5 and 6), we can see the germs of a more developed earthquake analysis, applicable not only to the more complicated electrical equipment necessary for more than 220 kV, but also to several industrial installations.

The analysis would consist in the separate consideration of the design earthquake, as a transient random signal, and of the structural arrangement as a graph of transfer functions, each representing the single elements, with feedback loops representing the couplings. This kind of analysis would left in front of the designer the essential characteristics of his chosen structural arrangement, before confusing them with the random characteristics of the signal (see also the discussion to 6)

Do the authors have a view on this specific matter?.

B) In our simplified analysis, much importance is given to the fact that a sufficiently rigid element will not have a response higher than that of the ground, nor represent an amplification of the ground excitation when emplyed as a support for another oscillator. From our experience, and considering the elastic design spectrum in (1), we have chosen a natural frequency of 15 Hz to define such a rigid element or support.

(I) Author of Paper N° 69.

From the new design spectrum in fig. 1 and Table 1, we can see that the authors define a "faring frequency" of 20 to 40 Hz. As the matter is most important from the economical point of view, we would like to know the opinion of the authors.

It is perhaps interesting to add, that our experience seems to indicate that even lower frequencies than 15 Hz have not yet been detected to give significant amplification. Some other facts, as the absence of failures on higher modes, when these could have got amplification on account of critical natural frequencies (see, by ex., that the failures observed in ref. 5 correspond only to the fundamental mode of the towers), could also serve as a corroboration.

C) The selection of a 1.0 g earthquake we understand it corresponds to the maximum credible earthquake. We are not, however, very surprised by the increase in intensity it represents for the design, which we suppose provoked by the San Fernando earthquake. This, because, in spite not having yet registered very great intensities, we have already noticed that the statistical breakage of capacitor columns in Concepción, 1960 (see case a in ref. 3), demand a maximum ground acceleration of 0,7 to 0.85 g to be explained. with a 7.5 Richter earthquake at 90 km distance. We are investigating, however, the ground there, to try to detect any local amplification, if present.

## References.


(1) See ref. 1 of the Paper.

(2) F. Novoa : Earthquakes and the Substation Equipment Arrangement and Specification, CIGRE 1970, report 23-02.

(3) F. Novoa : Earthquake analysis and Specification of the HV Electrical Equipment, 5 WCEE, paper N° 69.

(4) F. Novoa : Further remarks to (3).

(5) T. Konno, E. Kimura : Earthquake Effects on Steel Tower Structures Atop Building, 5 WCEE, paper N° 21, and discussion by F. Novoa.

(6) H. Sato, K. Suzuki : On the Response Analysis of the Structure System subjected to Multi-seismic Motions, 5 WCEE, paper N° 19, and discussion by F. Novoa.