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Some Probabilistic Aspects of the Seismic Risk of Nuclear Reactors

California Univ, Los Angeles Dept of Energy and Kinetics

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16. Abstracts The potential for existing cracks in piping systems to grow to critical size and cause system failure during a severe earthquake was examined, and the critical crack size based on linear elastic mechanics was reviewed. A simple and crude method of relating the initial crack size, the range of strain produced in a component, and the number of cycles to failure, was proposed and discussed. The significance of piping failure due to ordinary crack growth other than the intergranular stress corrosion under seismic loading was found to be comparable with the potential for failure due to crack growth under severe seismic loads. A simplified piping system with only a few snubbers and hangers as seismic restraints was devised. The inverse of the maximum Von Mises ratios at a few nodal points of the piping system was used as the basis for indicating the change in safety factor due to piping restraint failures. The potential importance of seismic events leading to a core melt accident was examined by factoring the effect of design errors and system degradation into an estimate of the probability of a core-melt accident due to seismic events.

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TEH-MING HSIEH
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"A General Evaluation Approach to Risk-Benefit for Large Technological Systems and its Application to Nuclear Power

UCLA • SCHOOL OF ENGINEERING AND APPLIED SCIENCE

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Department of Energy and Kinetics
School of Engineering and Applied Science
University of California, Los Angeles

PREFACE

This report represents one aspect of a National Science Foundation funded study at UCLA entitled, "A General Evaluation Approach to Risk-Benefit for Large Technological Systems, and Its Application to Nuclear Power," (NSF Grants GI-39416 and OEP 75-20318). The objectives of this project can be defined to include the following:

1) To make significant strides in the provisions of improved bases or criteria for decision-making involving risk to the public health and safety (where a risk involves a combination of a hazard and the probability of that hazard).

2) To make significant strides in the structuring and development of improved, and possibly alternative, general methodologies for assessing risk and risk-benefit for technological systems.

3) To develop improvements in the techniques for the quantitative assessment of risk and benefit.

4) To apply methods of risk and risk-benefit assessment to specific applications in nuclear power (and possibly other technological systems) in order to test methodologies, to uncover needed improvements and gaps in technique and to provide a partial selective, independent assessment of the levels of risk arising from nuclear power.

Reports prepared previously under this grant include the following:

1. Mathematical Methods of Probabilistic Safety Analysis, G.E. Apostolakis, UCLA-ENG-7464 (September 1974).
2. Biostatistical Aspects of Risk-Benefit: The Use of Competing Risk Analysis, H.N. Sather, UCLA-ENG-7477 (September 1974).

3. Applying Cost-Benefit Concepts to Projects which Alter Human Mortality, J. Hirshleifer, T. Bergstrom, E. Rappaport, UCLA-ENG-7478 (November 1974).

4. Historical Perspectives on Risk for Large Scale Technological Systems, by W. Baldewicz, G. Haddock, Y. Lee, Prajoto, R. Whitley and V. Denny, UCLA-ENG-7485 (December 1974).

5. A Prediction of the Reliability of the Core Auxiliary Cooling System for a HTGR, K.A. Solomon, D. Okrent and W.E. Kastenberg, UCLA-ENG-7495 (January 1975).

6. Pressure Vessel Integrity and Weld Inspection Procedure, K.A. Solomon, D. Okrent and W.E. Kastenberg, UCLA-ENG-7496 (January 1975).

7. A Survey of Expert Opinion on Low Probability Earthquakes, D. Okrent, UCLA-ENG-7515 (February 1975).

8. On the Average Probability Distribution of Peak Ground Acceleration in the U.S. Continent Due to Strong Earthquakes, T. Hsieh, D. Okrent and G.E. Apostolakis, UCLA-ENG-7516 (March 1975).

9. The Effect of a Certain Class of Potential Common Mode Failures on the Reliability of Redundant Systems, George E. Apostolakis, UCLA-ENG-7528 (November 1975).

10. Risk-Benefit Methodology and Application: Some Papers Presented at the Engineering Foundation Workshop, September 22-26, 1975, Asilomar, California, D. Okrent, Ed., UCLA-ENG-7598 (December 1975).

11. A Computer-Oriented Approach to Fault-Tree Construction, S.L. Salem, G.E. Apostolakis and D. Okrent, UCLA-ENG-7635 (April 1976).

12. The Effect of Human Error on the Availability of Periodically Inspected Redundant Systems, G.E. Apostolakis and P.P. Bansal, UCLA-ENG-7650 (May 1976).

13. An Integrated Safe Shutdown Heat Removal System for Light Water Reactors, J.C. Ebersole and D. Okrent, UCLA-ENG-7651, (May 1976).

14. On the Failure Modes of Alternate Containment Designs Following Postulated Core Meltdown, C.K. Chan, UCLA-ENG-7661 (June 1976).

15. Probability Intervals for the TOP Event Unavailability of Fault Trees, Y.T. Lee and G.E. Apostolakis, UCLA-ENG-7663, (June 1976).

16 Statistical Models for Competing Risks Analysis, H. Sather, UCLA-ENG-7676, (August 1976).

17 On Risk Assessment in the Absence of Complete Data, W.E. Kastenberg, T.E. McKone and D. Okrent UCLA-ENG-7677, (August 1976).

18 Cost-Benefit Analysis and the Art of Motorcycle Maintenance, B. Fischoff, UCLA-ENG-7685, (August 1976).

19. On the Probability of Loss of DC Power Following AC Failure in a Nuclear Power Plant, J. Chun and G.E. Apostolakis, UCLA-ENG-76112, (December 1976).

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CHAPTER I. INTRODUCTION

This report is derived directly from the Ph.D. dissertation of T. Hsieh and, to facilitate early dissemination, has been reproduced using all the same figure numbers, table numbers, references, etc. This report is comprised of three of four studies by Hsieh related to the seismic safety of nuclear power reactors. The first part of the dissertation has already been published as a UCLA Engineering Report (UCLA-ENG-7516) and in Annals of Nuclear Energy (Vol. 2, p. 615, 1975).

In this report we first examine the potential for existing cracks in piping systems to grow to critical size and cause system failure during a severe earthquake. First, the critical crack size based on linear elastic mechanics is briefly reviewed. Using the critical size as a criterion, it is concluded that crack growth under seismic loading of SSE or smaller is likely to be insignificant ($\leqslant 10\%$), if general yield is not produced in a component. However, for some materials under seismic loading so severe that inelastic strain is induced in the component, the extent of crack growth could be very significant and finally lead to failure. A simple and crude method of relating the initial crack size, the range of strain produced in a component, and the number of cycles to failure, is proposed and discussed. The significance of piping failure due to ordinary crack growth other than the intergranular stress corrosion under seismic loading is found to be comparable with the potential for failure due to crack growth under severe seismic loads.

The failure of hydraulic snubbers and hangers represents another common deficiency of system components due to deterioration, design and construction error, or operation error. A simplified piping system with only a few snubbers and hangers as seismic restraints is devised, and the SAPIV code is used to calculate the responses under the assumption of various combination of failures of piping restraints. The inverse of the maximum Von Mises ratios at a few nodal points of the piping system is used as the basis for indicating the change in safety factor due to these piping restraint failures. A rough relationship is found for the reduction of safety factor between multiple and single failures.

Finally, the potential importance of seismic events leading to a core melt accident is examined. Subjective extrapolation of past records on design and construction errors, which may or may not be seismic related, is made and is used to infer the number of seismic-related design errors. Also, subjective values of safety-factor-reduction are assigned for these errors. Using the Newmark correlation between the probability of system failure and safety factor, the effect of design errors and system degradation is factored into an estimate of the probability of a core-melt accident due to seismic events. It is concluded that seismic risk could be more significant, than is estimated in WASH-1400 and that further study of the matter is warranted.

CHAPTER III. CRACK INDUCED FAILURE UNDER SEISMIC LOADING

III.1. Introduction

In this chapter we will examine the increase in potential failure of a piping system during a severe earthquake due to the presence of flaws prior to the earthquake. We will find that employing linear fracture mechanics and assuming that the yield point is not exceeded, crack growth due to seismic loading is small, just as it would be for a limited number of operational cycles. However, if general yield (i.e. plastic deformation in the component) is produced during seismic loading, crack growth is no longer negligible, at least for some materials. A simple methodology for relating the number of cycles to failure (critical size flaw) and the strain range is proposed. The significance of piping failure due to crack growth is also discussed.

The presence of defects, such as microcracks and even cracks of visible sizes, embedded in structural material during the process of manufacturing and being put into service, can be generally assumed. There are limits to the capability and efficiency of non-destructive testing such as ultrasonic test which is generally performed for vital components between manufacturing and reactor startup. It is well known that flaws may significantly reduce the allowable stress to a much lower level than that determined from conventional stress analysis without crack existence. Thus, the existence of cracks could significantly reduce the safety margin of a structure. A few well illustrated examples are given in Reference [32].

Although fracture mechanics can be applied as a tool for relating the fracture of materials with the general laws of applied mechanics and macro-scale material properties, the analysis involves many parameters. This greatly complicates the problem. Furthermore, depending on the degree of nonlinearity of the stress within the material and around the crack tip, linear elastic fracture mechanics (LEFM), or yielding fracture mechanics must be selectively applied. Only the former concept has been developed to the degree that it can have engineering application.

In this chapter, a crude and simple analysis of the critical crack size, crack growth, and their relationship to the failure probability of major components under seismic loading will be performed. Rigorous treatment is beyond the scope of this study.

III.2. Critical Crack Size

Based on linear elastic fracture mechanics (LEFM), the extent of the effect due to flaws can be quantified in the following manner. The main parameter, the stress intensity factor K, is related to the crack size "ℓ" (crack half length) according to the following equation

$$K = Q\sigma\sqrt{\pi\ell} \tag{28}$$

where Q is a flaw shape parameter and is dimensionless, σ is the nominal stress. K has the dimension of stress multiplied by the square root of length. When the value of the stress intensity factor K approaches that of the fracture toughness K_c of the material, the risk of brittle fracture arises. For a given structure or equipment with known K_c and σ, one can calculate the critical flaw size that can cause failure. Once a structure or equipment contains cracks of critical size, unstable crack propagation ensues, as long as the stress is maintained. An obvious limitation of this theory is that it is applicable only for relatively small cracks, since it is independent of the dimensions of the equipment.

LEFM applies well to materials without any notable ductility. It may also be applicable to ductile materials when the stress state is "plane strain," in which the applied stress is elastic and the crack tip plastic zone size is small compared to the thickness of the structure or equipment. For ductile materials with a relatively thin wall, empirical techniques have been used successfully. Usually, the geometry of the material can be approximated by a wide plate. For smaller piping, correction for bending stresses, which are not present in the plate, may be required. Examples of estimating the critical crack sizes are given below.

III.2.1. Critical Size of Surface Flaw or Embedded Flaw.

The applicable equation is [33], [34], [35]

$$a_{cr} = K_c^2 \left[\frac{\phi^2 - 0.212(\sigma/\sigma_y)^2}{1.21\pi\sigma^2} \right] \tag{29}$$

where

$$\phi = \int_0^{\pi/2} \left(1 - \frac{c^2 - a^2}{c^2} \sin^2\theta \right)^{1/2} d\theta$$

and a and c are the half depth and half length of the crack, respectively. Letting

$$\Psi = \left[\phi^2 - 0.212(\sigma/\sigma_y)^2\right]$$

we get

$$a_{cr} = \left(\frac{K_c}{\sigma}\right)^2 \frac{\Psi}{1.21\pi} \qquad \text{(surface flaw)}$$

$$a_{cr} = \left(\frac{K_c}{\sigma}\right)^2 \frac{\Psi}{\pi} \qquad \text{(embedded flaw)}$$

The Ψ values as functions of σ/σ_y and of a/2c are given in the references. Assuming an equipment made of a material with σ_y = 64 ksi and K_c 160 ksi$\sqrt{in}$, the critical surface flaw sizes are shown in Fig. III.1. Given a nominal stress, one can find the critical crack depth or length for different a/2c values on this figure. For embedded flaws, the corresponding critical half width (a) will be greater by a factor of 1.21. The underlying equation suggests that $a_{cr} \propto \frac{K_c^2}{\sigma^2}$ and is independent of the structure, equipment size, or shape.

III.2.2. Critical Size of Partially Through Wall Flaw

Based on the experimental data of the General Electric Co. [36] 6" carbon steel sch. 80 pipe (t = 0.432", σ_{ult} = 60 ksi), critical 2c/a versus stress for different a/t is shown in Fig. III 2. In this figure, t is the wall thickness of the pipe.

III.2.3. Critical Through Wall Flaw Size

Based on the empirical equation derived from full-scale pressurized pipe experiments at Battelle's Columbus Laboratories, the critical through wall flaw size can be calculated according to the equation [37]

$$\frac{K_c^2\pi}{8c\bar{\sigma}^2} = \ln\left[\sec\left(\frac{\pi}{2}\frac{M_t\sigma_t}{\bar{\sigma}}\right)\right] \qquad (30)$$

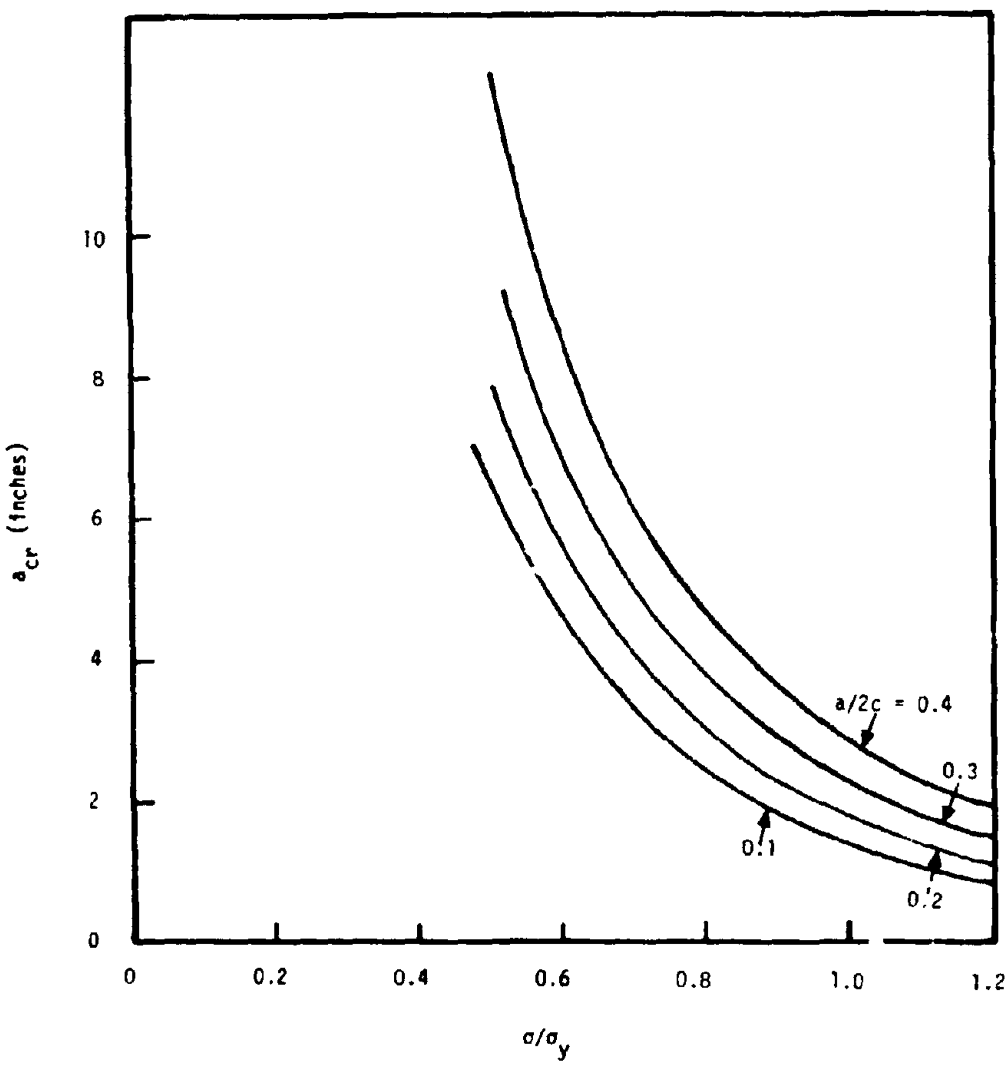

Fig. III.1 Critical Sizes of Surface Flaws for a Material with σ_y = 64 Ksi, K_c = 160 Ksi $\sqrt{in}$

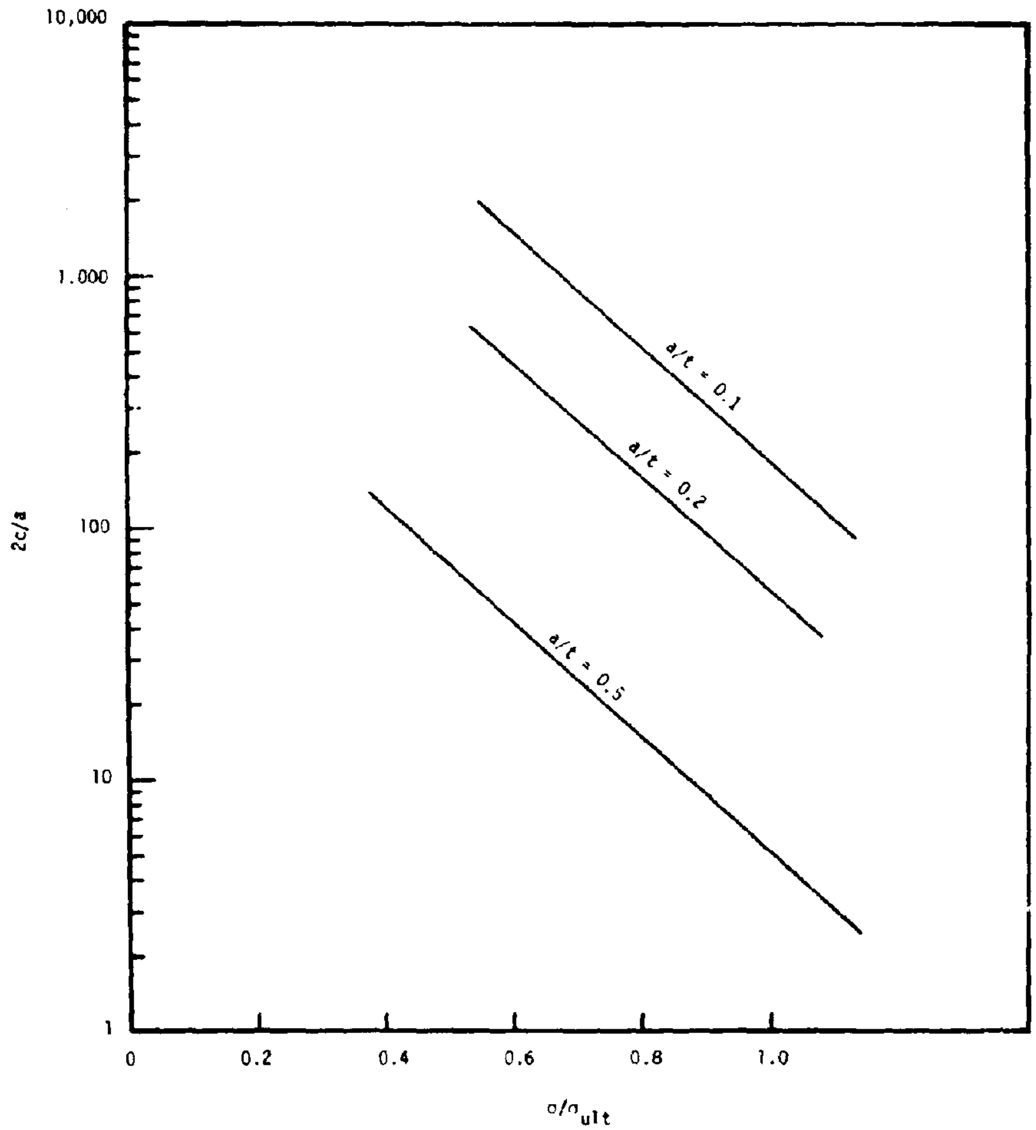

Fig. III.2 Critical Size of Partially-Through Wall Flaws in a Pipe.

where

σ_t = hoop stress

$\bar{\sigma}$ = flow stress

$M_t = \left(1+1.255\,\frac{c^2}{Rt} - 0.0135\,\frac{c^4}{R^2t^2}\right)^{1/2}$, the Folias correction factor

R = pipe radius

t = wall thickness

2c = through-wall flaw length

In the above the flow stress, the Folias correction factor is introduced in the equation to account for the ductility and the geometry effect of pipe materials. Another empirical formula [38]

$$K_c^{\,2} = \frac{\pi\sigma_t^{\,2}c}{\cos\theta}\left(1+0.4\,\frac{c^2}{Rt}\right) \tag{31}$$

can also be used. Here, $\theta = \frac{\pi}{2}\frac{\sigma_t}{\sigma_{ult}}$. Assuming a pipe with K_c = 160 ksi $\sqrt{in}$, σ_y = 73.6 ksi, σ_{ult} = 80 ksi, R = 7", t = 0.75", two curves, predicting critical through-wall flaw size, are shown in Fig. III.3.

III.3. Low Cycle Fatigue Crack Growth

In the previous section we have seen that the critical crack size could be very small ($\lesssim$ 1 inch), depending on the material and the loading. A subcritical flaw may grow to critical size during the normal service life via two principal mechanisms: stress corrosion and fatigue. It may also grow substantially during highly stressed seismic vibration. We will examine the rate of crack growth by low cycle fatigue, since it relates to flaw growth during an earthquake.

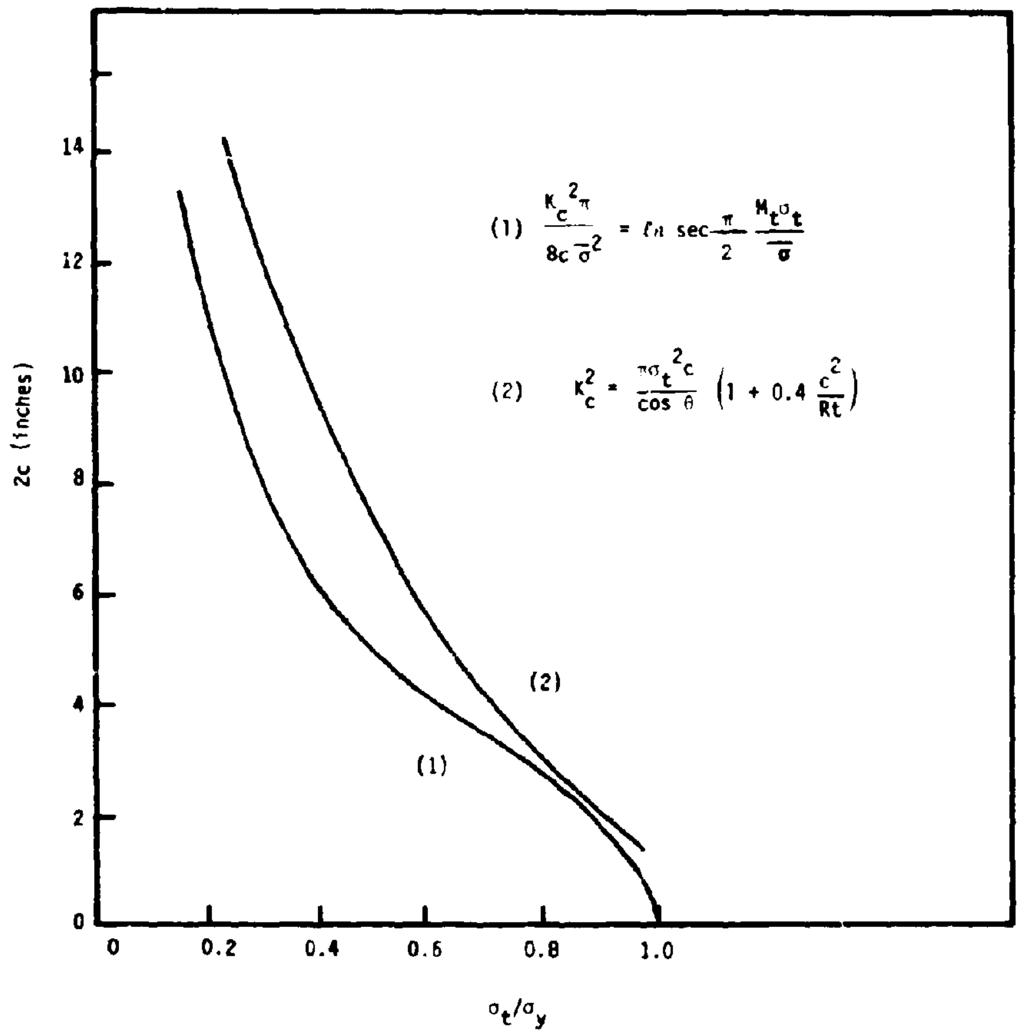

Fig. III.3 Critical Sizes for Through Wall Flaws
(for a Pipe or Vessel with K_c= 160 ksi $\sqrt{in}$, σ_{ult} = 80 ksi,
σ_y= 73.6 ksi, R= 7", t= 0.75").

The overall corrosion effect is ordinarily expected to be relatively small; if a rapid form of intergranular stress corrosion exists, it, of course, could lead to growth of large flaws which might lead to failure during an earthquake.

Operational heat-up and cool-down during the life of a power plant ($\sim$40 yrs) may be in the order of a few hundred cycles. However, the induced stress is expected to be smaller than the seismic loading and reports have indicated that fatigue growth due to operational cycles is negligible ($\leq$ 10%) [39, 40, 41].

Crack growth is sensitive to many parameters which can be related to the flaw configuration, the material, the environment and the loading. It is also recognized that the shape of a crack changes during its growth [42]. However, fatigue crack growth data are generally correlated well with the range of the stress intensity factor within the LEFM domain The basic form of the correlation is

$$\frac{d\ell}{dN} = c_o(\Delta K)^m \tag{32}$$

where $\frac{d\ell}{dN}$ is the crack growth per cycle; c_o, m are constants, and ΔK is the range of stress intensity corresponding to the cyclic load, and can be evaluated from Eq. (28). Alternativaly, the number of cycles after which an initial crack size of ℓ_i will grow to the critical crack size ℓ_{cr} can be expressed, after integrating Eq. (32) as

$$N_f = \frac{2c_1}{m-2}\Delta\sigma^{-m}\left[\frac{1}{\ell_i^{\frac{m-2}{2}}} - \frac{1}{\ell_{cr}^{\frac{m-2}{2}}}\right] \tag{33a}$$

for m > 2; and,

$$N_f = c_2 \Delta\bar{\sigma}^{2} \ell_n \frac{\ell_{cr}}{\ell_i} \tag{33b}$$

for m = 2.

Here $c_1 = \dfrac{1}{c_o Q^m \pi^{m/2}}$, and $c_2 = \dfrac{1}{c_o Q^2 \pi}$

In Figure III.4, we plot N_f versus the crack growth rate for $c_o = 3.69 \times 10^{-9}$, m = 2.4, [34] and $\Delta\sigma$ changes from 0 to the yield stress σ_y = 64 ksi. If the N_f considered is not to be more than 200 cycles,* the crack growth would be less than 10% according to the curves. For smaller σ/σ_y, the growth will be even smaller.

Begley [43] proposed the following crack growth formula:

$$\ell n \frac{\ell_f}{\ell_i} = \Delta\sigma^2 \frac{10}{E^2} Y^2 N \tag{34}$$

which relates the final crack size (ℓ_f) to the initial crack size (ℓ_i) by the variables: stress range ($\Delta\sigma$), Young's Modulus (E), crack shape factor (Y), and the number of stress cycles(N).

Assuming a strong ground motion effect with

$N = 200$ cycles

$\Delta\sigma = 6 \times 10^4$ psi

and Y = 2, it is found that

$$\frac{\Delta\ell}{\ell_i} = 3.3\%$$

*Actually a much smaller number of large peak ground accelerations velocities and displacements is expected in a strong earthquake. However, N_f values of 100 or 200 cycles are used for purposes of illustration.

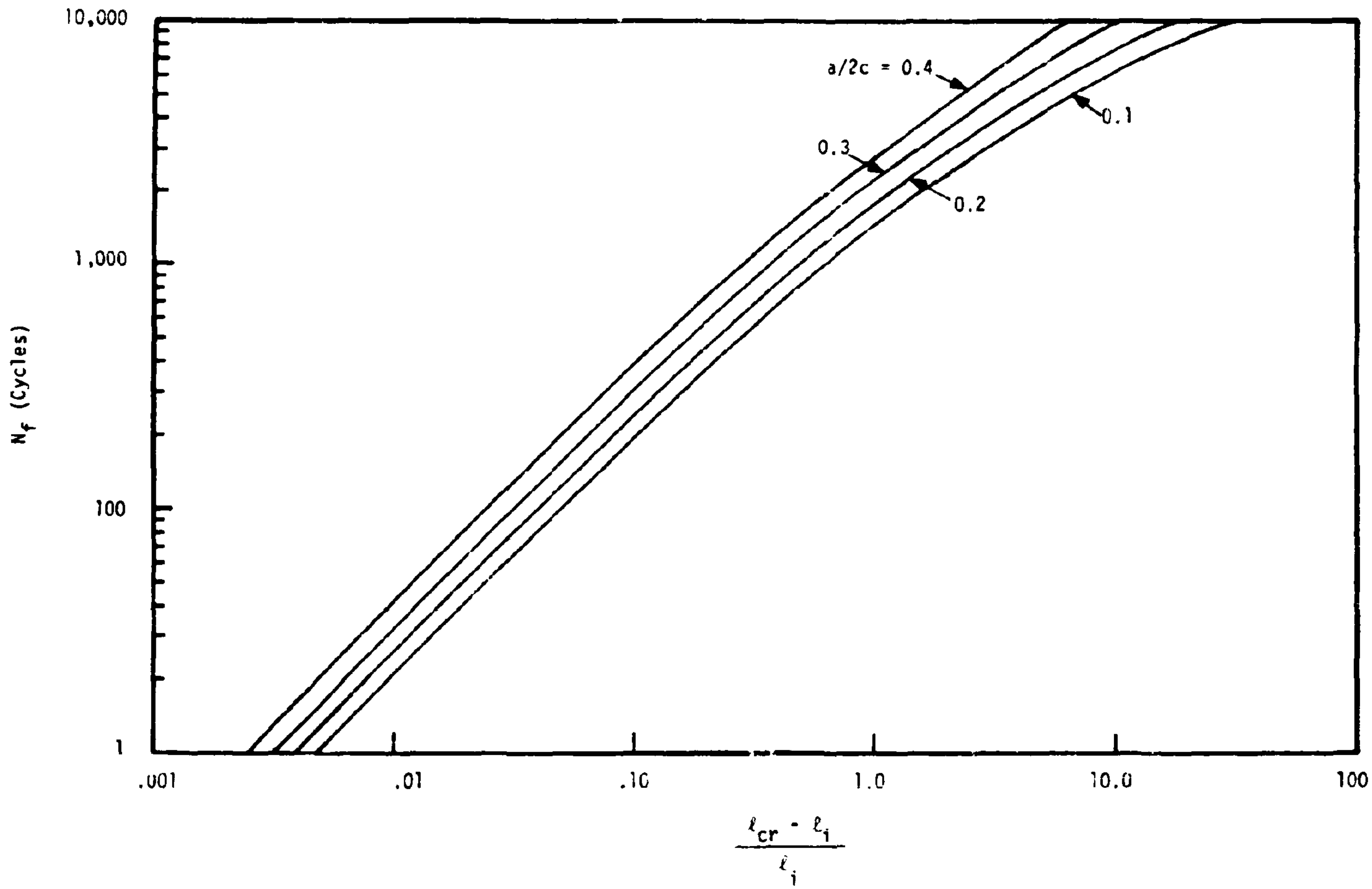

Fig. III.4 Relation Between Number of Stress Cycles and Percentage of Crack Growth to Critical Size

Thus, from the above, the growth of the initial crack is probably insignificant compared to the uncertainty of the critical crack size prediction, if we do not exceed yield.

Parameters other than the stress intensity factor have been studied for crack growth prediction, including crack tip opening displacement [44], absorbed hysteresis energy per cycle [45], etc. Generally speaking, more research and experimental data are needed before practical engineering application of these parameters can be realized. However, particularly for specimens which contain sizeable plastic zones, $\Delta\epsilon_p$, the applied plastic strain range has been widely used. A few examples are shown in Figure III.5. Curve No. 1, the Coffin-Manson relation [46]

$$N_f = \frac{1}{4}\left(\frac{\epsilon_f}{\Delta\epsilon_p}\right)^2 \tag{35}$$

relates the cycles to failure N_f to the applied plastic strain range $\Delta\epsilon_p$. In Equation (35), ϵ_f is the fracture strain. In Figure III.5, ϵ_f = 28.6%, approximately equivalent to an area reduction of 75%, a typical value for stainless steel. Curve No. 2 represents Coffin's law [47]

$$N_f^n \, \Delta\epsilon_p = c \tag{36}$$

where n is independent of the material, and to a first order approximation, $n \approx 0.5$ and $c \approx 0.65$ for a number of ductile materials. Curve No. 3 was based on Iino's crack growth rate experimental data [48] assuming that the compressive strain is relatively small. Curve No. 4 is a transposition of one of Das' data for stainless steel [49].

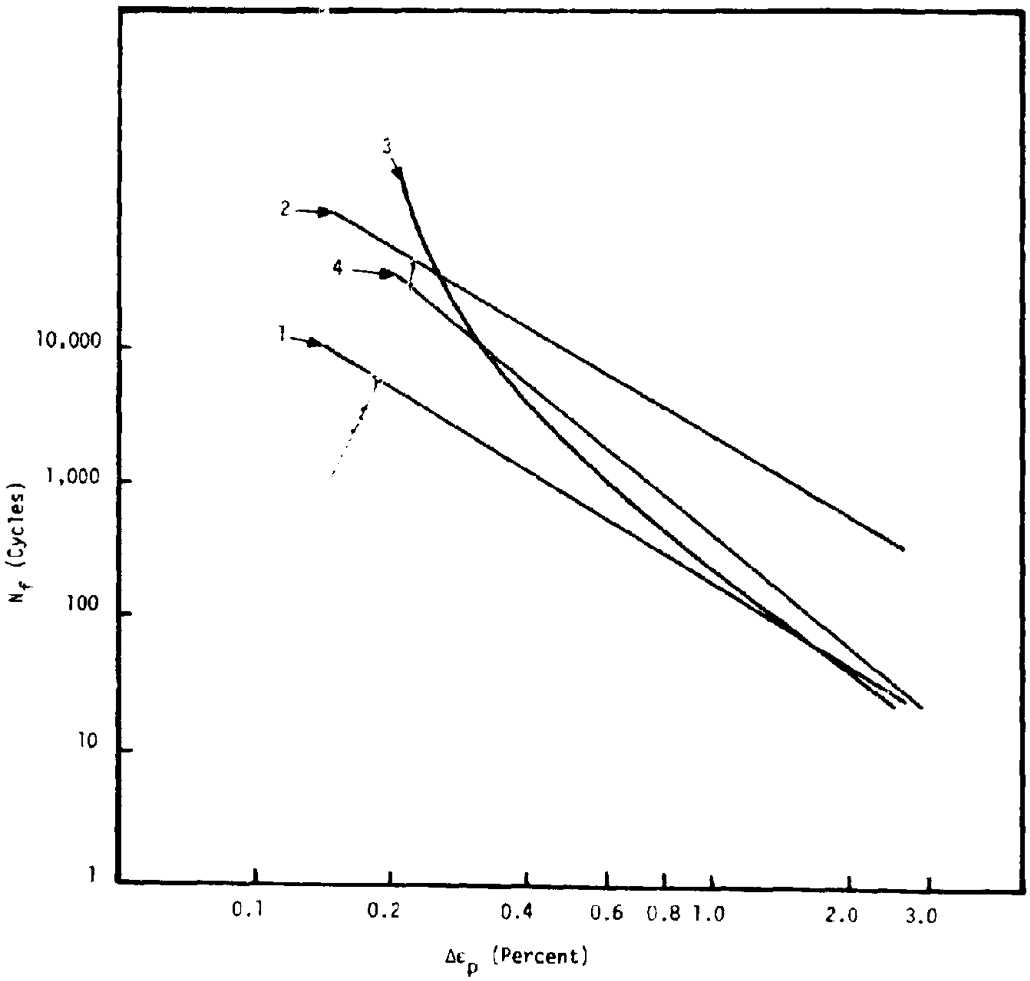

Fig. III.5 Number of Cycles to Failure (N_f) for Specimens With Sizable Plastic Strain

In addition to the stress intensity factor and fracture toughness, Cherapanov, et al., introduced a new material constant, β[50], which can be related to the specific dissipation energy of the material, for all materials manifesting plastic properties. The fatigue crack extension rate is obtained as

$$\frac{d\ell}{dN} = -\beta \left[\frac{K_{max}^2 - K_{min}^2}{K_c^2} + \ell n \frac{K_c^2 - K_{max}^2}{K_c^2 - K_{min}^2} \right] \tag{37}$$

in which β and K_c must be determined experimentally. Table III.1 shows some of the data presented in the reference.

Since the critical crack size could be as low as an inch, and the interesting number of cycles could be about 100 cycles, a crack growth rate greater than 10^{-2} in/cycle is significant. It can be seen from Table III.1 that a material with $\beta \geq 10^{-2}$ inches could be damaged during strong earthquake motion.

A crack growth law which takes the stress ratio R (the min. stress/max. stress during a cycle) into account and having a form like [45]

$$\frac{d\ell}{dN} = c(\Delta K)^m / \left[(1-R)K_c - \Delta K\right]^n \tag{38}$$

would also predict a rapid crack growth when ΔK approaches K_c in the case of $R = 0$.

III.3. Cycles to Failure as A Function of Initial Crack Size Under Plastic Deformation

In a nuclear power plant, components of Seismic Category I are expected to have some degree of plastic deformation under SSE influence. In the following, a methodology for estimating the number of cycles to

Table III.1. Crack Growth Rate (in/cycle) for Some Steels

Material	K_c ksi$\sqrt{in}$	β (in)	$\frac{K_{max}}{K_c}$			
			0.4	0.6	0.8	1.0
Martensitic aging Steel 250	200	7.87×10^{-3}	1.4×10^{-4}	7.9×10^{-4}	3.7×10^{-3}	1.5×10^{-2}
Martensitic aging Steel 300	190	3.54×10^{-3}	6.7×10^{-5}	3.9×10^{-4}	1.6×10^{-3}	4.7×10^{-3}
Stainless Steel 310	130	3.94×10^{-2}	7.9×10^{-4}	3.9×10^{-3}	2.0×10^{-2}	7.9×10^{-2}
Stainless Steel 301	197	1.57×10^{-1}	2.4×10^{-3}	1.4×10^{-2}	6.7×10^{-2}	3.5×10^{-1}
Ni-Mo-V Steel	146	1.18×10^{-3}	1.2×10^{-5}	7.9×10^{-5}	3.5×10^{-4}	1.6×10^{-3}
Steel HP9-4-25	130	7.87×10^{-4}	2.0×10^{-5}	1.3×10^{-4}	4.7×10^{-4}	7.9×10^{-4}
Steel B-95	56	2.31×10^{-2}	3.5×10^{-4}	2.0×10^{-3}	9.8×10^{-3}	3.9×10^{-2}

failure as a function of initial crack size is outlined for severely stressed components in which plastic deformation has occurred.

The critical crack size is determined first from the stress intensity factor and the fracture toughness as discussed in Section III.1, keeping the strain-amplitude constant for each cycle. However, the stress intensity factor range must be modified since the strain is beyond the domain of LEFM. Equation (9) of Reference [51] can be used for this purpose.

$$\Delta K' = \Delta K \left(1 + \frac{\Delta \epsilon_N^p}{\Delta \epsilon_N^e}\right)^{1/2} \tag{39}$$

where $\Delta K'$ is the modified stress intensity range, and superscripts p, e denote plastic and elastic, respectively. Assuming an initial crack size ℓ_i slightly smaller than the critical one ℓ_{cr}, Equation (37) is used to determine the cycles to failure. Since no closed form solution similar to Equation (33) can be found, a cycle-by-cycle crack growth process must be followed. N_f is obtained whenever ℓ_i becomes greater than ℓ_{cr} at the $(N_f - 1)$th cycle. For smaller ℓ_i's, the entire process is repeated until N_f is greater than the value predicted by the curves in Figure III.5.

As an example, an embedded crack growth history has been obtained for stainless steel 310 ($\beta = 3.94 \times 10^{-2}$ inch, $K_c = 130$ ksi$\sqrt{\text{in.}}$) and is shown in Figure III.6. Approximations made in the process are:

a. The stress-strain relation remains unchanged from cycle to cycle.

b. $K_{min}=0$, assuming that the crack has closed during compression with a negligible strain amplitude.

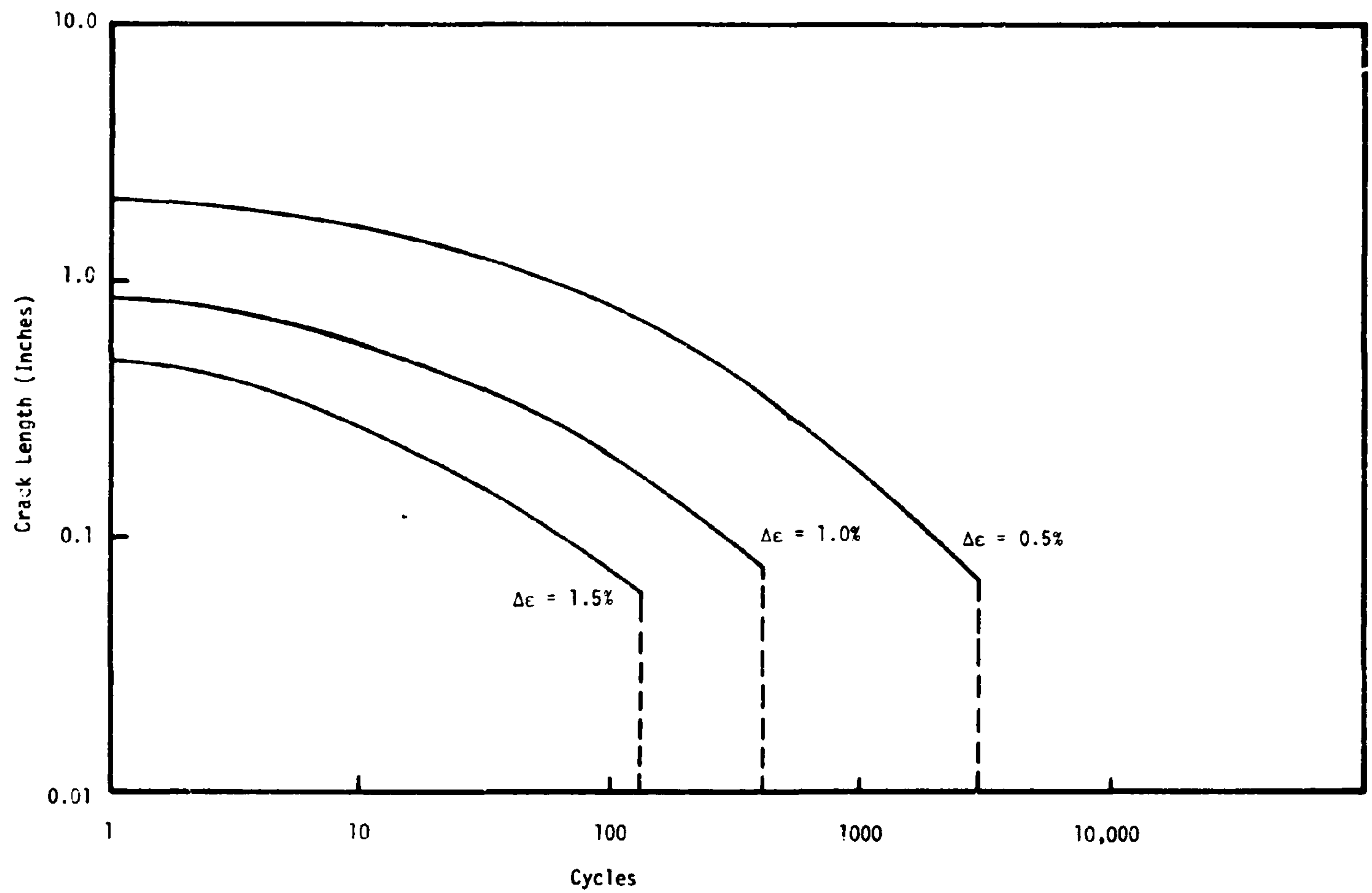

Fig. III.6 Number of Cycles to Failure as a Function of Initial Crack Length (for Stainless Steel 310)

Figure III.7 is based on information contained in Figure III.6. It shows that components with crack lengths less than ~0.06 inch could be regarded as not having cracks at all, since their sizes have no effect on the cycles to failure under plastic strain. It also indicates that at about 0.1 inch size, the crack starts to grow much more rapidly, which agrees with the experimental observation of Gowda, et al., [51].

III.4. Discussion

It appears that if the seismically induced stress is well within the LEFM domain, the crack growth phenomenon is unimportant. The initial size of undetected cracks alone determines the probability of component failure. However, when plastic strain does accompany cyclic loading, fatigue crack growth may be dominant for materials with high β values. Careful evaluation of the crack growth history is warranted for this case.

Several materials, e.g., stainless steel, exhibit the familiar stress-strain relation in which very little incremental stress is required beyond the yield strength to produce large plastic deformations. Thus, although Figure III.6 was plotted using the strain range as a parameter, the corresponding stress ranges may not be linearly proportional, and the absolute stress could be only slightly above the yield strength for most cases. An earthquake which contains a few tens of cycles that would induce strain beyond the elastic range, could cause the failure of some components. The probability of failure will be close to the probability of the earthquake which would produce such inelastic deformations, if the probability of significant flaws is large.

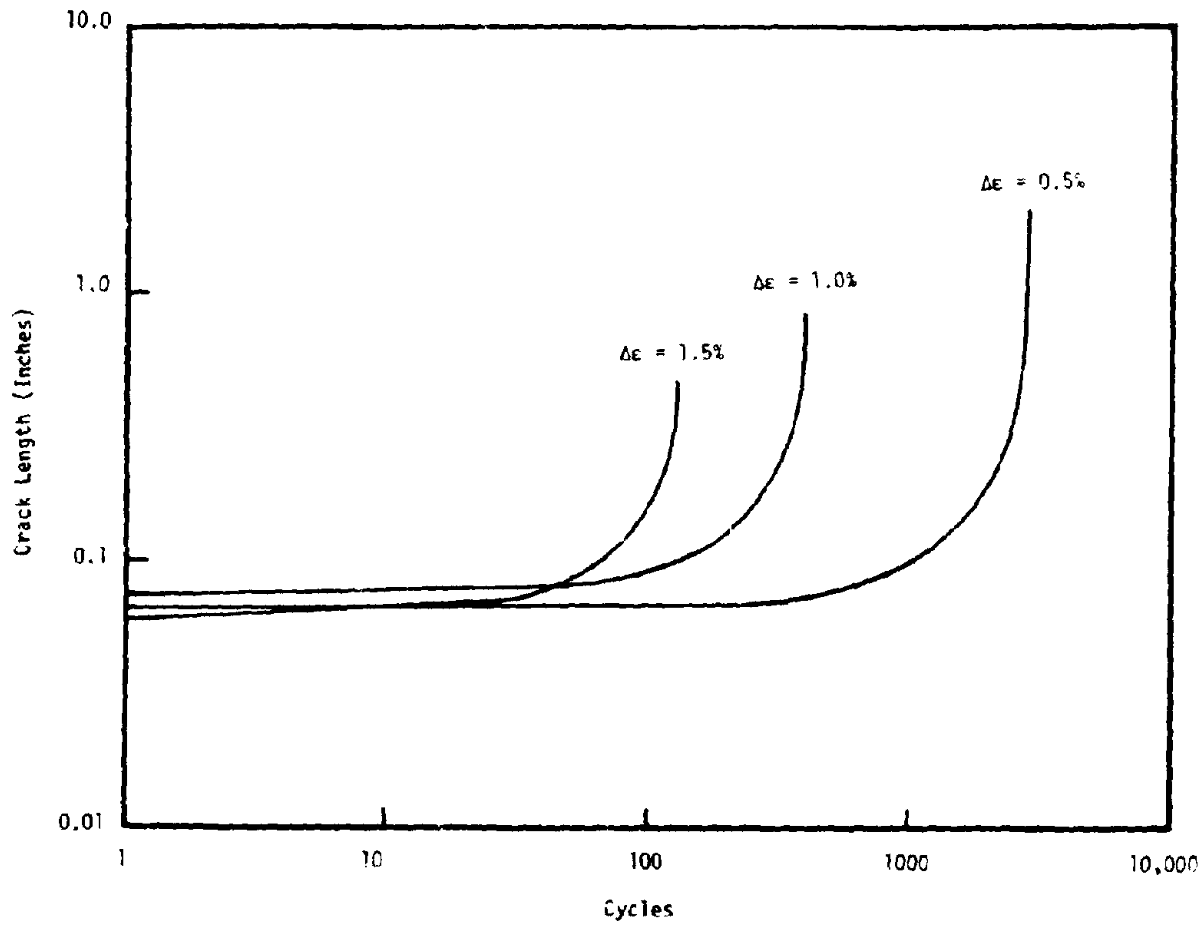

Fig. III.7 Crack Growth History (for Stainless Steel 310)

We will now do a simple calculation to illustrate the importance of seismically induced pipe failure due to crack growth when plastic deformation is involved.

One inference from the work of Wilson [36] is that the circumferential joint made in field construction is the weakest point with respect to crack detectability. The probability of an undetected crack with its length ranging from 1 to 8 inches and with a depth up to about 1/10 of the wall thickness is estimated to be about 5×10^{-2} per joint for the worst case. Assuming that 10 such joints exist for a typical piping system, and that at about 0.6g (or 3 times an SSE of 0.2g ground acceleration) the earthquake induces stresses that exceed the yield point, the probability of seismic failure of the piping system would be about $0.5 \times 2 \times 10^{-4} = 10^{-4}$ per year per piping system. In this example, 2×10^{-4} is the estimated annual probability of ground acceleration $\geq 0.6g$ (Table II-5) for an average reactor site.

According to Bush [52], experience from the operation of nuclear power plants indicates that the probability of rupture of piping larger than 4" diameter is about 10^{-5} to 10^{-6} per ft-year. For a 100 ft piping system, the frequency of disruptive piping failure might be of the order of 10^{-3} to 10^{-4} per year per piping system. The piping system failure frequency due to crack growth under severe seismic loading conditions would then be on the same order of magnitude or somewhat smaller. If the piping system has redundancy (such as a typical RHR system), and if one assumes independent random failure for each loop, failures of both loops from other than seismic causes ranges in frequency from 10^{-6} to 10^{-8} per year per piping system. On the other hand, if one assumes a common-mode failure under such a severe earthquake, the frequency of failure for the redundant systems would be essentially the same as for an individual system, and system failure due to a large seismic event (with undetected cracks) could be dominant.

Further examination of this matter appears to be warranted.

CHAPTER IV. HANGER AND SNUBBER FAILURE ANALYSIS FOR A PIPING SYSTEM

IV.1. Introduction

In this chapter, we shall examine how the failure of hangers and seismic snubbers reduces the safety factor for piping system stresses in an earthquake. First, we shall analyze a simplified piping system with only a few restraints. The structure analysis computer code SAP IV [53] will be used to calculate the change in the piping stresses. The change in safety factor within the piping system will then be calculated on the basis of the SAP IV output results.

The failure of hangers and snubbers in a nuclear power plant is not a rare event. In 1973, in a nuclear power plant sited in the Eastern U.S., several hydraulic snubbers were found to be devoid of fluid. Subsequent inspection of all snubbers in several operating power plants revealed that a large percentage of them would not be functional as designed. The main function of a snubber with fluid-filled cylinders is to increase the rigidity of the piping system, or other Seismic Category I components, through the incompressibility of the fluid during strong and rapid vibration. The main cause of failure is the loss of fluid, which is due to seal deterioration. Hangers, although usually do not contain fluid, have often been found damaged, broken, or pulled out of the wall due to design deficiency, installation error, water hammer or mechanical fatigue. The hangers are used in conjunction with the snubbers to minimize damage to the piping during transients such as earthquakes. For a heavy piping system with long spans, appropriate use of hangers is particularly important in minimizing the static bending moments induced by gravitational force.

The design of a Seismic Category I piping system is usually very conservative. A properly designed piping system should have a fairly high built-in safety factor. A single hanger or snubber failure is not expected to cause a catastrophic failure of the piping system, but will reduce the margin of safety. The main purpose of this chapter is to study the possible extent of this safety margin reduction.

In a modern nuclear power plant, there are many very complex piping systems. Different sizes and types of restraints are used in them at various locations. Hence, it would be futile to search for a "representative" piping system. Within a given piping system, the importance of a pipe restraint would not be uniform with respect to either the restraints themselves, or to the loading effects at various locations. On top of its complexity, a realistic three-dimensional mathematical model for a piping system would involve hundreds of degrees of freedom; therefore, its computer analysis would be too expensive, costing hundreds of dollars for each run. In order to keep the problem within manageable size and reasonable computer expenditure, (and at the same time, avoiding any infringement of proprietary rights), a suitable piping system model with only a few restraints was created.

By removing one or more restraints at a time from the piping system and modifying the inputs to a structural analysis computer program code (SAP IV), the effects of various failure combinations of the snubbers and hangers were studied. The statistical (mean or median) value of the importance of a single piping restraint can be obtained by examining the reduction of safety factors under various failure conditions at a few important nodal points in the piping system. The response spectrum method is used for the dynamic analyses. This allows for examination of the effects of earthquakes of different magnitudes (larger than the safe shut-down earthquake) in a straightforward manner.

IV.2 The Piping System

The piping system chosen for analysis had four snubbers and four hangers, and is shown in Figure IV.1. The nodal points are numbered sequentially, with a distance of 4.92 ft between them, except at the three identical 90° elbows, and between nodal points 3 and 4, where a heavy 2200 lb valve is located. The total length of each of the three equal arm elbows is 3.28 ft, with a bending radius of 1.64 ft. For simplicity, the pipe size and materials are kept the same throughout the entire piping system. The 14" outside diameter pipe, the mass per unit length, including the water content (131 lbs/ft), the 400 psi internal pressure, and the stainless steel material are taken from a residual heat removal (RHR) system of a recent nuclear power plant. There are three fixed ends where the floor response of the building transmits the earthquake motion.

Some ground rules for designing the piping system are the following:

1. The static force must minimized. To do this, the piping is broken up into sections between the hangers. The hanger loads are calculated by balancing the moments due to uniformly distributed loads (gravitational forces) and to concentrated forces (the valve and the hangers) for each individual section. The combined forces

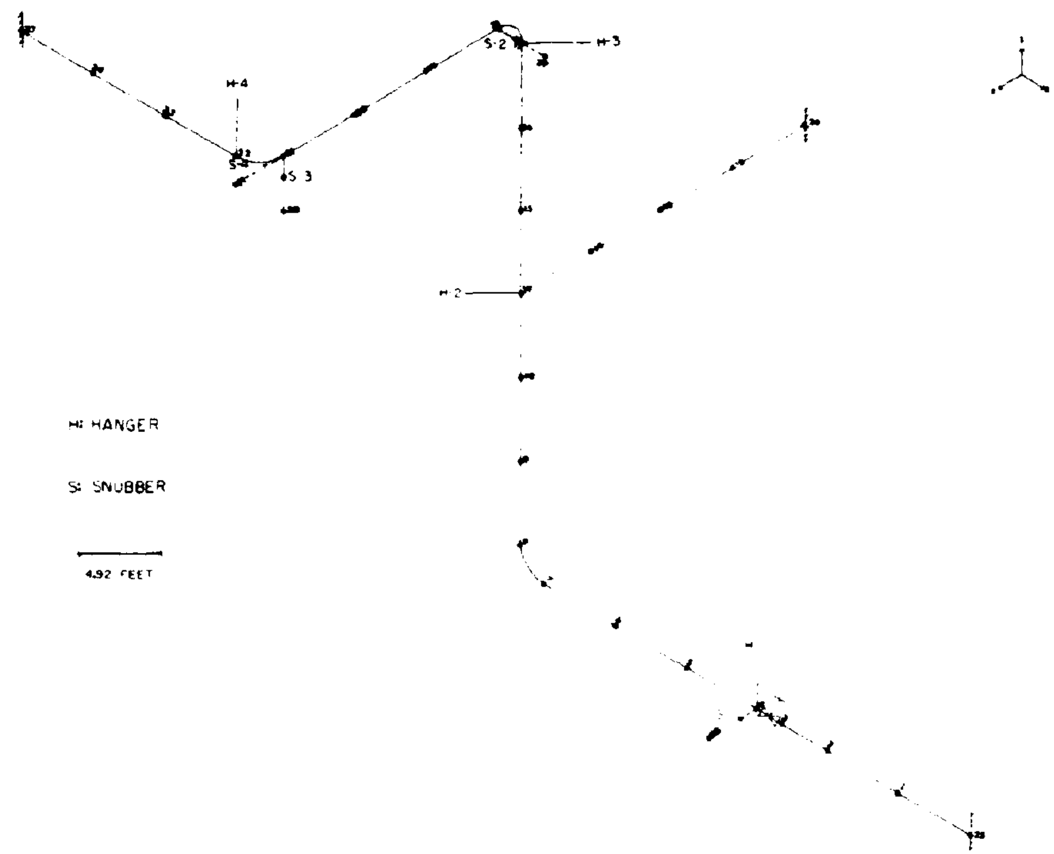

FIG IV.1 The Hangers and the Snubbers of the Piping System from two neighboring sections are then used for choosing a proper hanger [54]. To avoid any transfer of stress from support to support during transients and to simplify the analysis (to be discussed in more detail in the next section), constant force hangers are adopted. The maximum static deflection for the piping system with four such hangers was found to be less than 0.1 inch, which seems reasonable.

2. The practical maximum stresses in a well designed piping system under faulted conditions (due to safe shutdown earthquake), with all piping restraints functional, are generally of the order of 10,000 psi, or less than the allowable stress. Too high or too low stresses would indicate that the piping system is either underdesigned or overly conservative. Depending on the classification of a piping system, Section III of ASME Boiler and Pressure Vessel Code [55] permits maximum stresses of 36,000 to 45,000 psi under faulted conditions.

3. The sizing and the location of the four snubbers are such that the overall piping system is relatively rigid, with its first mode frequency reasonably above the peak response frequency of the floor response spectrum.

The maximum stress of the piping system was found to be 8843 psi at the tee connection (node 11 in Figure IV-1). The maximum Von Mises ratio (to be explained in the next Section) is 0.062. The first natural frequency of the piping system was about 2.5 times that of the peak floor acceleration (resonant frequency). Table IV.1 gives the sizes of the hangers and the snubbers and their locations are shown in Figure IV.1.

Table IV.1. The Hangers and the Snubbers of the Piping System (see Figure IV.1).

Hanger identification	Hanger load (lbs.)	Snubber identification	Snubber Damping (lbs/in/sec	Snubber Stiffness (lbs/in)
H1	4242	S1	2000	221000
H2	5475	S2	1000	110500
H3	2362	S3	1000	110500
H4	2362	S4	1000	110500

IV.3 The Analysis

The computer program SAP IV, created for the static and dynamic analysis of linear structural systems, is used to study the effects on the static and dynamic responses of the piping system due to failure combinations of the snubbers and the hangers. The response spectrum analysis mode is selected for the dynamic analysis with a typical floor response spectrum identical for both north-south and east-west horizontal directions; the vertical response is assumed to be 2/3 of that of the horizontal one for SSE = 0.17 g with 5% damping. (Figure IV.2)

In the static analysis only the hangers need to be considered, since the snubbers freely allow slow displacement, such as thermal exapansion. In the dynamic analysis, only the snubbers need to be considered, since constant support hangers are used, which would exert no dynamic forces in addition to that on the piping. As the deflection varies during vibration, the spring constant also varies, keeping the hanger load unchanged; hence there are no dynamic forces acting. Thus the failure of the hangers and of the snubbers can be studied separately by static

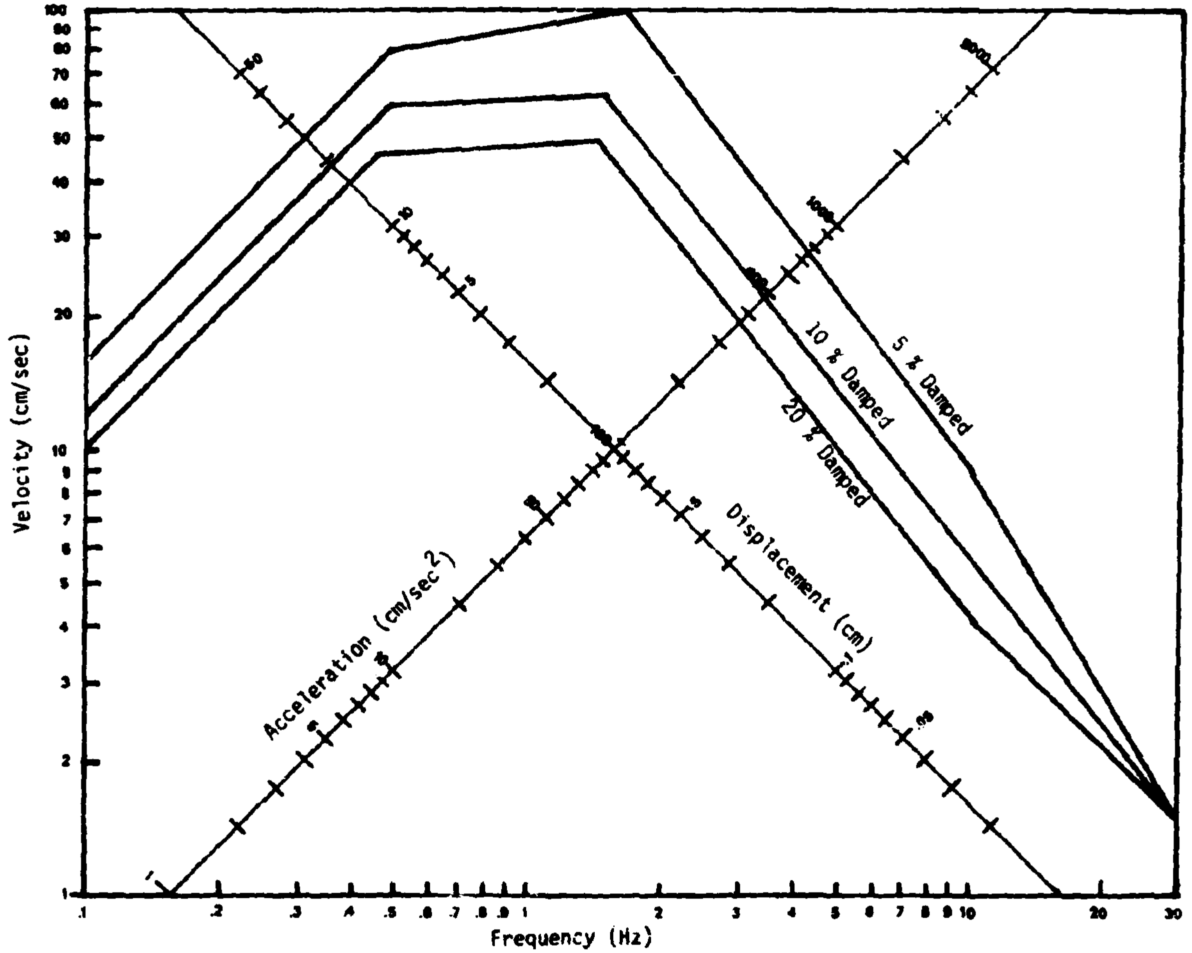

Figure IV - 2. The Floor Response Spectrum Used in the Piping System Analysis (SSE = 0.17 g) analysis and dynamic analysis, respectively. The number of computer runs required for all failure combinations is greatly reduced (from 256 to 32) by the introduction of the constant force hangers. SAP IV prints out axial forces, shear forces, torques, and bending moments for each nodal point. A special stress analysis computer program [56] developed by the Applied Nucleonics Corporation was used in a slightly modified form to calculate the maximum combined stresses, and the maximum Von Mises ratio, which is defined as

$$V_r = \frac{\sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2}{\sigma_y^2}$$

where σ_1 and σ_2 are the two principal stresses, and σ_y is the yield stress. Based on the maximum distortion energy theory, when $V_r \geq 1$, structural element failure can happen [57]. It is obvious that the inverse of the maximum V_r is a measure of the safety factor. Maximum combined stress calculations are performed for the three fixed ends and for the tee connection, because at these locations maximum structural responses were observed. The changes of the safety factors at these nodal points due to pipe restraint failure give a statistical indication of the importance of the restraints.

Lastly, the effects of different earthquake magnitudes and the resonant effect of the floor responses are also of interest. The former was calculated simply by scaling up the floor response spectrum, whereas the latter was done by moving the peak acceleration frequency of the floor response spectrum to coincide with the first mode frequency of the piping system. All piping restraints were assumed to be functional.

IV.4. Results and Conclusions

The following tables present the results of the SAP IV structural analyses and the combined stress analyses for the failure of one or more pipe restraints.

Table IV.2. Maximum Von Mises Ratios and Corresponding Safety Factors of the Complete Piping System

Nodal Point	Maximum Von Mises Ratio (V_r)	Safety Factor ($1/V_r$)
11	0.06204	16.12
25	0.03360	29.76
26	0.04667	21.43
27	0.02687	37.22

IV 3.1. Failures involving snubbers only; all hangers are functional.

Table IV.3. Safety Factor Reduction Due to Failure of One Snubber (Avg = 0.67; Median = 0.69)

Snubber No.	Node 11	Node 25	Node 26	Node 27
1	0.81	0.34	0.37	0.96
2	0.84	0.69	0.34	0.90
3	0.81	0.69	0.41	0.57
4	1.11	0.75	0.59	0.60

Table IV.4. Safety Factor Reduction Due to Failure of Two Snubbers (Avg = 0.57; Median = 0.51)

Snubber No.	Node 11	Node 25	Node 26	Node 27
1, 2	0.76	0.33	0.29	0.87
1, 3	0.69	0.32	0.30	0.53
1, 4	1.05	0.40	0.48	0.42
2, 3	0.79	0.68	0.33	0.63
2, 4	0.82	0.79	0.36	0.48
3, 4	0.95	0.70	0.45	0.34

Table IV.5 Safety Factor Reduction Due to Failure of Three Snubbers (Avg = 0.46, Median = 0.37)

Snubber No.	Node 11	Node 25	Node 26	Node 27
1,2,3	0.67	0.32	0.28	0.61
1,2,4	0.71	0.36	0.30	0.34
1,3,4	0.94	0.38	0.40	0.26
2,3,4	0.68	0.64	0.27	0.29

Table IV.6 Safety Factor Reduction Due to Failure of All Snubbers (Avg = 0.37, Median = 0.30)

Snubber No.	Node 11	Node 25	Node 26	Node 27
1,2,3,4	0.66	0.33	0.26	0.23

IV.3.2. Failures involving hangers only. All snubbers are functional.

Table IV.7. Safety Factor Reduction Due to Failure of One Hanger (Avg = 0.71, Median = 0.70)

Hanger No.	Node 11	Node 25	Node 26	Node 27
1	1.04	0.39	0.63	0.95
2	0.50	0.53	0.23	0.65
3	0.86	0.77	0.48	0.89
4	1.04	0.97	0.87	0.51

Table IV.8. Safety Factor Reduction Due to Failure of Two Hangers (Avg = 0.46, Median = 0.43)

Hanger No.	Node 11	Node 25	Node 26	Node 27
1,2	0.41	0.23	0.17	0.56
1,3	0.66	0.31	0.33	0.78
1,4	0.95	0.38	0.55	0.46
2,3	0.36	0.40	0.14	0.50
2,4	0.48	0.50	0.21	0.33
3,4	0.80	0.73	0.43	0.43

Table IV.9. Safety Factor Reduction Due to Failure of Three Hangers (Avg = 0.30, Median = 0.30)

Hanger No.	Node 11	Node 25	Node 26	Node 27
1,2,3	0.30	0.19	0.11	0.43
1,2,4	0.39	0.22	0.16	0.28
1,3,4	0.62	0.30	0.30	0.38
2,3,4	0.34	0.39	0.13	0.26

Table IV.10 Safety Factor Reduction Due to Failure of Four Hangers (Avg = 0.20, Median = 0.20)

Hanger No.	Node 11	Node 25	Node 26	Node 27
1,2,3,4,	0.28	0.18	0.11	0.22

IV.3.3 Failures involving both hangers and snubbers.

Table IV.11 Safety Factor Reduction Due to Failures of One Hanger and One Snubber (Avg = 0.51, Median = 0.44)

Snubber No.	Hanger No.	Node 11	Node 25	Node 26	Node 27
1	1	0.85	0.21	0.26	0.91
1	2	0.82	0.28	0.26	0.85
1	3	0.85	0.28	0.29	0.56
1	4	1.17	0.30	0.40	0.56
2	1	0.43	0.21	0.12	0.62
2	2	0.46	0.37	0.13	0.55
2	3	0.44	0.37	0.13	0.39
2	4	0.56	0.40	0.16	0.41
3	1	0.72	0.29	0.21	0.84
3	2	0.77	0.52	0.24	0.77
3	3	0.72	0.52	0.23	0.51
3	4	0.95	0.57	0.31	0.52
4	1	0.86	0.33	0.33	0.49
4	2	0.85	0.65	0.32	0.48
4	3	0.87	0.66	0.36	0.25
4	4	1.18	0.72	0.52	0.35

Table IV.12 Safety Factor Reduction Due to Failure of Four Snubbers and Four Hangers (Avg = 0.12, Median = 0.092)

Snubbers No.	Hanger No.	Node 11	Node 25	Node 26	Node 27
1,2,3,4,	1,2,3,4	0.23	0.11	0.067	0.077

IV.3.4 Safety Margin Reduction Due to Other Factors. All Piping Restraints Functional

Table IV.13 Safety Factor Reduction Due to Earthquakes Twice, Four Times, and Eight Times Greater than the SSE

SSE	Node 11	Node 25	Node 26	Node 27	Avgg	Median
x 2	0.39	0.29	0.16	0.82	0.41	0.34
x 4	0.12	0.093	0.043*	0.48	0.18	0.11
x 8	0.032*	0.025*	0.011*	0.19	0.063	0.028

*Van Mises ratio ≥ 1

Table IV.14 Safety Factor Reduction Due to Resonant Effect (Avg = 0.71, Median = 0.73)

Node 11	Node 25	Node 26	Node 27
0.83	0.63	0.40	0.98

From the tables given in subsections IV.3.1 and IV.3.2, one can see a general trend of safety factor reduction, with increasing number of pipe restraint failures. If F_s and F_h denote the relative reduction of the safety factor for a single snubber failure and for a single hanger, respectively, then the approximate relation for the failure of n_1 snubbers and of n_2 hangers is:

$$F_{n_1 s} \cong (F_s)^{n_1} \tag{40}$$

and,

$$F_{n_2 h} \cong (F_h)^{n_2} \tag{41}$$

where F_s = 0.74, F_h = 0.67 for this particular piping system. The tables in subsection IV.3.3 also suggest an approximate relation for the combined failures of n_1 snubbers plus n_2 hangers:

$$F_{n_1s,n_2h} = (F_s)^{n_1} \times (F_h)^{n_2} \tag{42}$$

Substituting the above values of F_s and F_h into Equation (42), we find that the value of $F_{1s,1h}$ falls between the average and the median values given in Table IV.11. However, the above relation would give a lower value for the simultaneous failure of four snubbers and four hangers than Table IV.12 (0.06 as compared to the median value of 0.09).

The reduction of the safety factor for the piping system due to earthquakes larger than SSE is shown in Table IV.13. For an earthquake twice the magnitude of the SSE and larger, the safety factor is reduced to 1/3 of its original value as compared to $\frac{1}{2}$ in Newmark's numbers [58]. It should be noted that SAP IV considers linear responses only. Nonlinear structural deformations could occur under very strong earthquakes, and the responses could be smaller. The safety factor reduction will then be smaller. It is also observed that if the peak acceleration frequency of the floor response spectrum coincides with the first mode frequency, the effect is roughly equivalent to the failure of one snubber for this particular piping system, as shown by the numbers in Table IV.14.

The tables in sub-section IV.3.1 and IV.3.2 show that the effect of a pipe restraint failure depends on the restraint involved and also on the location of the nodal point in the piping system. In some cases the failure of a certain pipe restraint may slightly increase the safety factor by a fortuitous cancellation of stresses. However, the net effect is always a reduction of the safety margin for the whole piping system.

In the above, we have presented the average and median values of the reduced safety factors due to variously combined failures of the hangers and the snubbers. The worst case, i.e., the maximum reduction of safety factor, is also of interest. As the tables show, the maximum reduced safety factors are 0.34, 0.29, 0.26, and 0.23 for failures of one snubber, two snubbers, three snubbers, and four snubbers, respectively; 0.23, 0.14, 0.11, and 0.11 for failures of one hanger, two hangers, three hangers, and four hangers, respectively; 0.12 and 0.07 for combined failures of one snubber and one hanger, and four snubbers and four hangers, respectively. These values are expected to greatly depend on the specific piping system and on the design and sizing of the hangers and the snubbers.

We based the safety factor definition directly on the failure criterion of the maximum distortion-energy theory, (namely, $\sigma_y^2 \leq \sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2$). If the safety factor is defined to be proportional to the yield stress divided by an equivalent stress, all the related numbers in the previous tables would be changed to the corresponding square root values. Under this new definition, the maximum von Mises ratios would not be greater than unity for earthquakes larger than SSE (up to 8 times of SSE, see Table IV-13); the reduced safety factors would become 0.58, 0.33, and 0.17 as compared to 0.50, 0.25, and 0.125 of Newmark's numbers for twice, four times, and eight times of SSE, respectively.

Finally, it is not appropriate to generalize the above conclusions for all piping systems, since the specific system used in this study may differ significantly from others. For more flexible piping systems with many concentrated loads attached (which is usually the case for most piping systems in a nuclear power plant), the effect of snubber or hanger failure could be greater than the one calculated for our example.

CHAPTER V. THE IMPORTANCE OF SEISMIC RISKS FOR NUCLEAR REACTORS

V.1. Introduction

In WASH-1400 (the Reactor Safety Study - An Assessment of Accident Risks in U.S. Commercial Nuclear Power Plants)[59], it was concluded that the earthquake risk was negligibly small compared to other reactor accident risks. The main argument was that the simultaneous failure of two systems is required to produce a core melt accident. Based on Newmark's work [58], the probability of damage to a safety system with a built-in safety factor of 20 is of the order of 1.5×10^{-3}. The joint failure probability of two safety systems, using log normal mean, would be of the order of 10^{-5}. When this damage probability (P_D) is multiplied by the earthquake probability, the probability of core melt per reactor-year, due to a design basis earthquake, would be of the order of 10^{-8}, and thus the above conclusions was reached.*

In estimating the conservatism in the nuclear reactor facility seismic design, Newmark suggested a safety factor of 20 for the SSE, which resulted from the conservatism in the predicted free field ground motion and in the predicted structural responses. Based on the assumption that all the related parameters follow a log-normal distribution, a correlation was established between the probability of failure due to seismic events and the safety factor. Newmark also indicated that a substantial margin to failure exists for earthquakes that are

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*A short summary of Newmark's definition of safety factor is given in Appendix III, as quoted from the Atomic Industrial Forum Program, Vol. 2, No. 1., "Reactor Licensing and Safety."

significantly larger than SSE, and that the reduced safety factor is inversely proportional to the magnitudes of earthquakes.

While it may well be true that a safety factor as high as 20 exists for many important components in a nuclear power plant, due to a very conservative approach in the seismic resistance design, deficiencies of component quality will probably reduce this safety margin. Table V.1 gives some examples of such deficiencies. [36] [59].

Other factors such as the operator's panic reaction, fires, etc. due to a strong earthquake, may also result in safety margin reduction effects.

There are many possible failure paths, or, in the terminology of the fault tree methodology, many minimal cut sets would be induced directly or indirectly by SSE or larger quakes, each of which could lead to a core-melt accident independently. For the convenience of this parametric risk assessment study, we shall limit ourselves to the following ten failure paths:

1) containment collapse
2) scram failure, with small primary system leaks
3) reactor building foundation failure (such as soil liquefaction)
4) complete loss of AC power
5) complete loss of DC power
6) component cooling water system failure
7) ultimate heat sink system failure
8) multiple failure in the primary coolant system
9) residual heat removal (RHR) system failure
10) loss-of-coolant accident (LOCA), plus loss of minimum engineered safety features (ESF)

All of these failure paths are self-explanatory in that when SSE or a larger quake occurs, the reactor should be shut down, and the core

Table V.1 Examples of Deficiencies of Component Quality Assurance

Quality Assurance Stage	Deficiency
Functional Requirement Definition	• Improper specification of fluid, operating temperature, pressure, number of operating cycles, performance and capacity, external and internal environment, etc.
Design and Analysis	• Improper application of codes and standards. • Errors in mathematical models, load definition, boundary conditions, computer codes, numerical techniques, etc. • Departure of the analytical prediction from the true responses. • Inadequate design of pipe, equipment, structure size, wall thickness, joints, etc. • Interference with adjacent piping, equipment, or structure. • Uncertainties involved in the iterative design process, and in the incompleteness of system definition prior to the detailed layout. • Mislocation of the restraints, supports, etc. • Functional interference between systems when one of them fails. • Outdated analysis.
Material or Component Selection	• Uncertainties in material properties - strengths, fracture toughness, corrosion resistance, etc. • Use of nonstandard materials or components without adequate qualification test program. • Incompatible materials.
Manufacturing/ Construction	• Undetected cracks. • Defects in welding, deficiency in the welding procedure. • Misalignment of components. • Installation different from design.
Testing & Maintenance	• Errors due to improper procedures, lack of training, etc.
Degradation	• Aging or deterioration • Fatigue due to cycling stresses • Corrosion, creep, erosion • Radiation embrittlement.

should be sufficiently cooled, which need electrical power and functioning equipment. We shall group together paths No. 1 and 2 as group A, paths No. 3, 4, and 5 as group B, paths No. 6 through 10 as group C, and assume that the overall safety factor for each failure path is not greater than the Newmark factor for SSE. Later, we shall assume that the Newmark factor of 20 is applicable for group A, a much lower safety factor (tentatively, 6) for group B, and an intermediate safety factor (tentatively, 10) for group C. As can be seen from the list of the ten failure paths, some paths involve no redundance at all. For those paths which do involve redundant subsystems, such as the RHR system, we shall assume that the safety margin is not improved by redundancy, since the redundance has a considerable chance of being lost under very strong earthquake vibration, which affects all the components at the same time.

Reasons for choosing lower safety factors for group B include the following:

1) A site in which soil liquefaction is unlikely under SSE, could undergo soil liquefaction if the design ground-acceleration is doubled. For example, at a sand site with a water table about 5 ft. below the ground surface, the liquefaction potential as a function of the relative density (D_r) of the soil is given below [60].

Table V.2. Liquefaction Potential for a Sand Site

Max. ground surface acceleration	Liquefaction very likely	Liqn. potential depends on soil type and earthquake magnitude	Liquefaction very unlikely
0.10g	$D_r < 33$	$33 < D_r < 54$	$D_r > 54$
0.15g	$D_r < 48$	$48 < D_r < 73$	$D_r > 73$
0.20g	$D_r < 60$	$60 < D_r < 85$	$D_r > 85$
0.25g	$D_r < 70$	$70 < D_r < 92$	$D_r > 92$

2) Concurrent occurrences of failure of the standby emergency power system and the loss of off-site power have been reported in the past [61], and loss of offsite power is a high probability event for a large earthquake.

The group C paths generally involve hundreds of components. The majority of these components could be in series in the reliability sense or equivalently, be connected by OR gates in a fault tree diagram. As an example, a typical RHR system loop is composed of piping (a few hundred feet long, many different sizes), pipe fittings (e.g., pipe reducers, flanged joints, etc.), and valves (e.g., gate valves, check valves, one letdown valve, one stop valve, two throttling valves, and relief valves), also a pump, a heat exchanger, and a cooling water system for the heat exchanger which involves many components. Valve operators, component structural supports and many others could be separately included in the loop. Failure of any of these components would disable the decay heat removal function. Furthermore, the comlexity and vulnerability to undetected cracks, fatigue corrosion, creep, etc., could render a reduced safety margin for the paths of group C.

A complete quantitative evaluation of quality assurance deficiencies and their effects on the safety margin is an extremely difficult, if not impossible task. For example, earthquake excitation can produce slope instability, large scale soil subsidence, and partial, or complete loss of supporting capacity of the foundation [62] (liquefied), all of which can seriously damage many important structures in a nuclear power plant. Factors, known to influence liquefaction potential include the soil type, the relative density or void ratio, the initial confining pressure, and many other dynamic soil parameters, as well as the intensity and the duration of the ground shaking.

Sufficient data are not available to predict the statistical errors committed in the design process, e.g., in the geological and seismological study, in the soil property analysis, and in the final plant construction. In order to determine the safety margin reductions, a detailed deficiency analysis should be performed for the many possible faults of each element within the system. Each such analysis would be similar to that performed for the failures of snubbers and hangers in the preceding chapter. This would be much too complicated. In the following, we shall therefore limit ourselves to some historical perspectives of design and construction errors. We shall also select tentative safety margin reductions for each of three categories of design errors to be defined later.

V.2. Some Historical Perspectives on Design Errors

In an effort to obtain a crude basis for estimating the probability that a design (or construction) error may be present and will lead to a serious accident, given an earthquake similar to the Safe Shutdown

Earthquake (SSE), or larger, a brief and incomplete list of design (or construction) errors was prepared as follows:

1. Errors specific to a boiling water reactor (BWR) plant (hereinafter called the "reference" plant), with MARK I containment.
2. Errors generic to most, or to several, BWR plants.

Most of the "engineering design errors," associated with the reference plant are, in essence, functional deficiencies discovered principally by a small, interdisciplinary design review team (2 to 4 engineers) working within a typical, functional design organization with several hundred engineers performing conventional architect-engineering services. A few deficiencies were revealed by field tests, but because of the low frequency of failure or challenge, most deficiencies would not have been discovered by this method. Some other errors were revealed by the licensing process. It may be argued that the history of design and fabrication errors pertaining to the reference plant is not necessarily representative of all nuclear plants, also, that this plant (designed between 1967 and 1971) is an old one and that the newer plants should benefit from previous experiences. However, a brief examination of the available design error records for the period 1973-75 indicates a wide disparity in the reported errors and shows that in some newer plants than the reference plant several design errors have been discovered and reported. It should also be noted that seismic design errors, if any, have had little opportunity to surface yet.

The design (and construction) errors have been arbitrarily classified into the following three categories listed in order of decreasing significance:

1. Errors of major importance to public safety. These can lead to a complete loss of the plant function or a large reduction in the original design margin.
2. Errors of considerable significance to public safety. These can cause appreciable loss in the original design margin, or a considerable reduction in the availability of an important system, or a hazard to systems.
3. Errors of moderate significance to safety. These rerpresent a relatively modest reduction in the margin or in the overall availability of the needed functions.

A. Examples of Reference Plant Design and Construction Errors

CATEGORY I

1. The local creek and the local drainage patterns would allow plant flooding in the event of a probable maximum precipitation.

2. The hypothesized failure of Class I raw water piping led to the possible loss of the diesel building structure (and hence to all diesel power) during SSE. (A required assumption of a single failure of Class 1 piping under the condition of the loss of normal power.)

3. Hypothesized basement flooding, from torus header or other source, required the redesign of the shutdown cooling (RHR) system.

4. Use of mercury switches in the CO_2 systems could deprive the diesel generators of all air cooling when actuated by a seismic event which destroyed the off-site power.

5. Containment penetration design:

   a) Not designed for probable fault currents due to LOCA-caused effects in containment.

   b) Leaks and arcs when wetted.

6. Cable separation:

   *a) Failure to implement the backup control concept in detail. Sneak circuits via fire-induced multiple "short" circuits.

*"Backup control" assumes the complete loss of the normal control room rather than a simple abandonment by the personnel. It is not a current NRC requirement. (See General Design Criterion 19)

*b) Failure to include provisions in the backup control system for coincident electrical division damages in areas other than the control and the spreading rooms.

7. Hypothesized failure of High Pressure Coolant Injection (HPCI) and Reactor Core Isolation Cooling System (RCIC) primary loop pipes and related valve failures in the secondary containment. This could result in an intolerable environment for needed equipment. (Also covered in the generic item list for BWR's).

8. Possibility of coupled failure of "redundant" piping, should failure of one pipe cause erosion of the local filled soil supporting contiguous essential water lines, and hence failure of redundant system.

9. Disregard of distant (but not local) dam failures as a source for flood damage.

10. Failure to consider missile and blast damage from large LOCA on:
    a) Instrument lines which supply information on pressure changes for reflood loop selection
    b) Valve actuating equipment
    c) Wiring
    d) Control rod drive supply and exhaust lines which are
        (1) Grouped for physical concenience and
        (2) Serve one core quadrant

11. Failure of a single impulse (static) line in containment started a cascade of events leading to drywell high pressure ECCS activation,

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*"Backup control" assumes the complete loss of the normal control room rather than a simple abandonment by the personnel. It is not a current NRC requirement. (see General Design Criterion 19)

semi-automatic blowdown, and, depending on the availability or loss of offsite power, resulted in core dryout or thermal shock from condensate inflow.

12. Interception of open-cycle discharge from critical service water systems by canal added as part of tower cooling system. The result was the recycling of hot water until an unacceptable suction uptake temperature occurred.

13. Tornado vulnerability, single failure, and seismic vulnerability of the original ventilation system for the control room and for other critical rooms.

CATEGORY II

1. Parapet roof drains were inadequate for probable maximum precipitation (PMP).
2. Hot-water flooding of the cold-water intake resulted from off-site power failure (due to elevated pond water storage from the condensers to the cooling tower supply pumps).
3. Syphon-flooding of reactor areas could occur due to non-seismic Class I pipe failures (with pumps tripped).
4. Fuel handling cranes could drop loads or over-run their traverses if the limit or the target switches fail. The fuel pool is vulnerable to cask drops.
5. Improper thermal insulation may stop pumps with sensitive bearing and the seal-cooling systems (hydroclone filters) plug the sump filters, or spray systems, in the event of LOCA.
6. Failure of one or more impulse lines in the secondary containment could jeopardize the functioning of the secondary containment. (It is also covered in the generic list).
7. Ventilation cross-ties that exist to "separate" rooms that are housing the redundant safety equipment. (Fusible links do not prevent excessive common ambient temperature).
8. Numerous unqualified piping penetrations in the secondary containment.
9. Inadequate consideration of the lateral impact of long missiles on exposed pumps and pipes.

10. Large pumps ($\sim$ 8000 HP) that can be isolated by suction-discharge valving and are only protected by non-safety grade interlocks. Burst pressures can be caused by friction heat.

11. The inability to trip the non-safety-grade power circuits under seismic conditions, resulting in continued pumping into presumably damaged pipes and structures. Also, the probability of fire. (The trip circuits are generally more vulnerable to seismic shock.)

CATEGORY III

1. Inadvertent closure of the raw water circulating water valves without pump trip would cause marginal stresses in the circulating water system.
2. Non-seismic raw water piping located above the critical power switchboards.
3. Hazard from broken propane, or hydrogen-containing pipes.
4. Valve stem shear. The lack of "safety grade" torque and of limit switches allows this to happen in the event of an excessive motor torque capability on some valves.
5. Valve disc separation from the stem. The load torque-thrust originally was carried by welds not designed for such loads.

B. Design and Construction Errors Generic to Many BWR's

CATEGORY I

1. Failure to properly specify the dynamic loads in the torus in the event of a LOCA involving large ruptures.
2. Inadequacies in the original phenomenological description and computer modeling of LOCA-ECCS.
3. The potential flooding of the safety systems due to the failures of non-safety grade components.
4. Failure of HPCI or RCIC and of the primary loop pipes in the secondary containment with certain coupled valve failures could result in an intolerable environment for the needed equipment.
5. Inadequate separation for fire protection.
6. Potential loss of the fuel pool water due to fuel cask dropping into the pool.
7. Stress corrosion cracking in the safe ends and elsewhere.

CATEGORY II

1. Incorrect estimate of the torus baffle loads in the event of steam relief.
2. Failure of one or more impulse lines in the secondary containment could cause unacceptable conditions via non-isolated primary system leakage.
3. Inadequate provision for the inspection of the pressure vessel and of the primary system piping.
4. Inadequate protection for pipe whip.
5. Incorrect reactivity worth of the control rods in scram.
6. Potential for pump overspeed and/or pump damage due to runout in the event of postulated LOCA's.
7. Potential for severe dynamic forces in the torus in the event of long-term steam discharge into the heated water.
8. Flow-induced vibration damage in the reactor vessel.
9. Feedwater nozzle cracking.
10. Damage to the steam line hangers.
11. Snubber failures.

From the preceding lists nineteen Category I errors, twenty-one Category II errors, and a lesser number of Category III errors were found to be applicable to the reference reactor. It is anticipated that the number of Category III errors are actually much larger than those in Category I or II. Many of the errors reflect a possible deficiency in interface or in "system" engineering. Little effort has been made here to obtain a more complete listing of Category III errors since they are expected to play a lesser role in the estimated effect of design errors on safety. In the following, it will be assumed that the Category III errors are two times as common as the ones in Category I.

In order to find a crude basis for estimating the number of errors which might occur in the seismic design, the following approach is used herein:

(1) Assume that in later plants a lesser number of non-seismic design errors exist than in the reference plant. (Take the factor to range from 0.5 to 0.75.)

(2) Estimate the ratio of seismic engineering design effort to other safety-related engineering design. (Take the factor to range from 0.1 to 0.4.)

(3) Estimate the relative probability of a design error in the seismic design to that in other areas of design per engineering hour. (Take the factor to range from 0.1 to 0.4.)

Combining the smallest values of these three factors with the number of design errors, we obtain an estimate of the "minimal" number of seismic design errors, (called minimal design errors for brevity). Similarly, if the largest values of the three factors are used, we obtain an estimate of the "maximal" number of seismic design errors. For the Category II errors, a safety margin reduction factor of 2/3 is assigned, which is roughly equivalent to the effect of one snubber failure in the previously analyzed piping system. For Category I errors, the effect is arbitrarily set to be twice that of the Category II errors. For Category III errors, a 10% safety margin reduction is assumed. All these are summarized in Table V.3.

Table V.3. Estimated Seismic Related Design Errors in a Nuclear Power Plant

Design Error Category	Minimal Design Errors	Maximal Design Errors	Reduced Safety Margin
I	0.1	5	1/3
II	0.1	5	2/3
III	0.2	10	9/10

V.3. Parametric Assessment of Earthquake Risks with Respect to Design Errors.

In the following assessment of earthquake risk, the number of design and construction errors, for both the minimal and for the maximal cases in each of the three categories shown in Table V.3, are assumed to be uniformly distributed among the ten failure paths. The reduced safety factor for each failure path, due to design errors in the three categories is estimated from the following expression:

$$F_{rj} = \prod_{i=1}^{3} (R_i)^{n_i} \qquad (43)$$

where R_i's are the ratios of the reduced safety factors to the Newmark factor; i refers to the category, and "n" designates the number of design errors. F_{rj} gives the overall ratio of the reduced safety margin for the paths of the jth group to Newmark's factor. This equation was derived on a more or less heuristic basis. However, the results from Chapter IV of this study, on the reduction of safety factor due to multiple pipe restraints failures, (Equations (40) through (42) provide some support for the use of Equation (43). Newmark's correlation between the safety factor and the probability of failure, reproduced in Figure V.1, is used to estimate the probability of damage $(P_j)^*$ to the reactor core, due to the path of the jth group. The conditional probability of damage leading to core-melt per reactor (P_D), assuming SSE or a larger earthquake, is calculated from Equation (44), below. This equation can be derived from the fact that each failure path is a minimal cut set whose occurrence would lead to an unacceptable event, e.g., a core melt down. This gives

$$P_D = 1 - \prod_{j=1}^{3} (1-P_j)^{m_j} \tag{44}$$

Here, m_j is the number of paths in the jth group. The following Tables present the results of the parametric evaluation.

*P_j is the probability of failure corresponding to a safety factor equal to F_{rj} multiplied by the original safety factor for the failure path in the jth group.

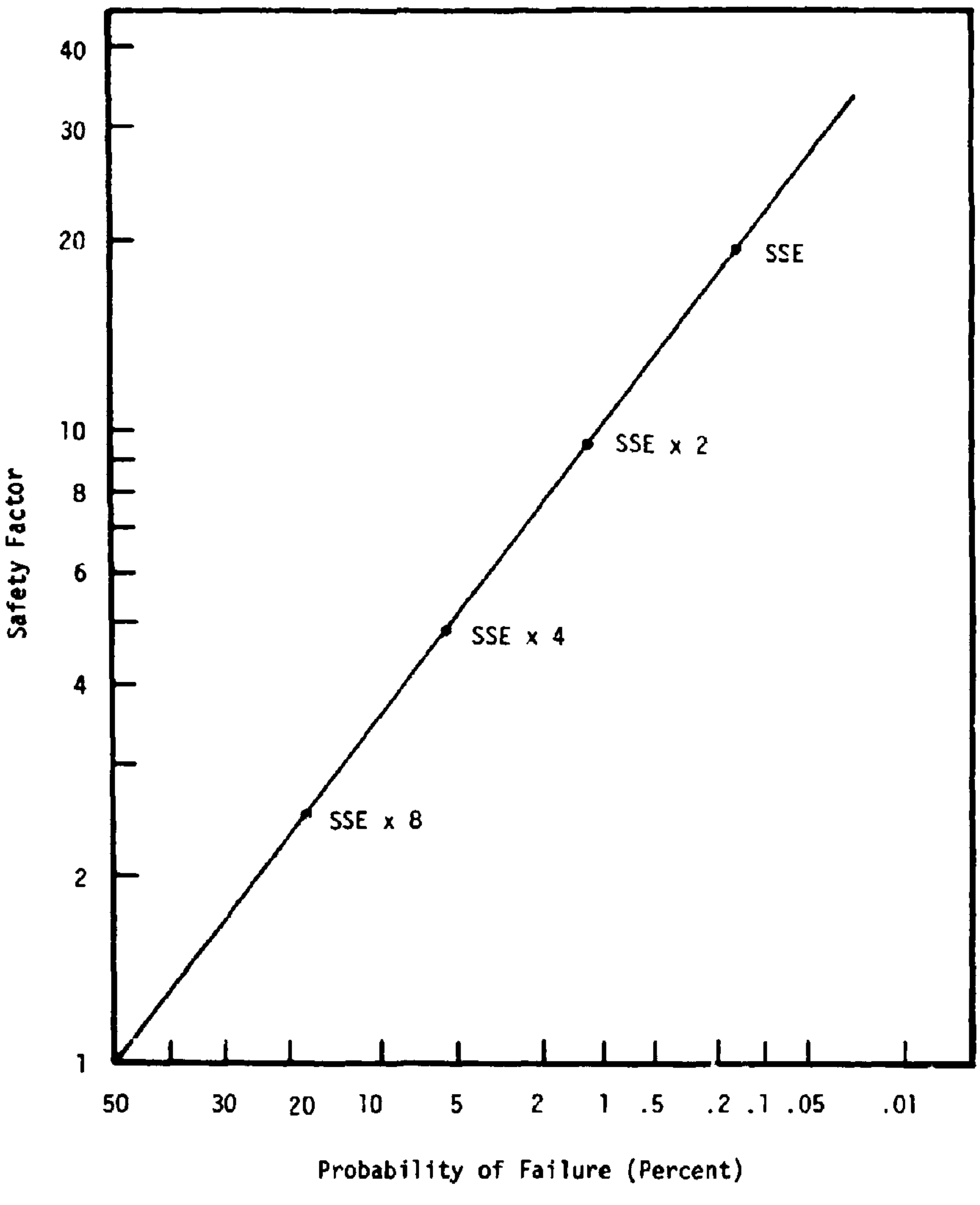

Fig. V.1 Newmark's Correlation Between Safety Factor and Probability of Failure for Nuclear Facilities

Table V.4. The Conditional Probability of Damage to the Reactor Core (P_D), Assuming that Each of the Ten Failure Paths Has a Built-in Safety Factor of 20 for Reactors Designed for SSE with 0.2g.

Ground Acc.	No Design Errors	Minimal Design Errors	Maximal Design Errors
0.2g	0.015	0.017	0.16
0.5g	0.18	0.20	0.72
1.0g	0.58	0.61	0.97

Table V.5. The Conditional Probability of Damage to the Reactor Core (P_D), Assuming Three Failure Path Groups With Built-in Safety Factors of 20, 6, and 10, Respectively.

Ground Acc.	No Design Errors	Minimal Design Errors	Maximal Design Errors
0.2g	0.16	0.17	0.60
0.5g	0.67	0.69	0.98
1.0g	0.96	0.98	~1.0

In the analysis, the failure probabilities were based on the assumption that the number of design errors are evenly distributed among the ten failure paths. If all design errors would occur to only one particular path at a time, the safety margin would be cut down to a very low value, and that particular path would become the weakest link in the reliability chain. However, this event is unlikely because the combinatorial probability is very low for such an uneven distribution.

By combining the conditional probabilities (P_D) with the earthquake probabilities, an estimated core melt per reactor year (P_{CM}), due to earthquakes, can be obtained. Table V.6 shows the estimated probability of earthquake damage resulting in core-melt for an average site in the Eastern United States. The values were taken from Ref.[54] for the frequency of earthquake occurrence with different peak accelerations.

Table V.6. The Probability of Core Melt per Reactor Year [P_{CM}] Due to Earthquake, for an Average Site in the Eastern United States Designed for SSE of 0.2g.

		No, or Minimal Design Errors		Maximal Design Errors	
Ground Acc.	Frequency of Ground Acc. Per Yr.	Uniform Safety Factor of 20	Three Safety Factor Groups	Uniform Safety Factor of 20	Three Safety Factor Groups
0.2g	7×10^{-4}	1×10^{-5}	1×10^{-4}	1×10^{-4}	4×10^{-4}
0.5g	5×10^{-5}	9×10^{-6}	3×10^{-5}	3×10^{-5}	5×10^{-5}
1.0g	1×10^{-5}	6×10^{-6}	1×10^{-5}	1×10^{-5}	1×10^{-5}
	Total*	8×10^{-5}		Total*	8×10^{-4}

Thus, the total earthquake risk, which is obtained by summing all accelerations ≥ 0.2g, would range from 8×10^{-5} to 8×10^{-4} per reactor year. This is equivalent to an average period of 1250 to 12500 reactor years between core melt-downs.

*Including P_{CM}'s for 0.3, 0.4, 0.6, 0.7, 0.8, 0.9g.

V.4. Discussion and Conclusions

The analysis suggests that earthquakes could be significant contributors to serious reactor accidents. With a uniform safety factor of 20 for each possible failure path which could independently lead to core melt, together with minimal or no seismic design errors, the probability of core melt per reactor year due to an earthquake is about the same as that estimated in WASH-1400 as due to a LOCA and to reactor transients. If significant (maximal) design errors prevail and, if at the same time, many failure paths have a reduced safety margin (less than the Newmark factor), the probability of a core melt per reactor-year would be one order of magnitude higher, and the seismic event would be the predominant cause. This inference contradicts the conclusions of the above reference. For the convenience of comparison and of disucssion, we shall reproduce the information contained in the two tables on page 67 of WASH-1400, together with the results of our analysis.

Table V.7, for the total probability of core melt per reactor year, should also include the probabilities for 0.3g, 0.4g, 0.6g, 0.7g, 0.8g and 0.9g, however, these seem to have been omitted in the reference analysis. If these probabilities were included, the total probability would become 2×10^{-6}, instead of 5×10^{-7}.

The large difference in total probability between our results and those of WASH-1400 can be attributed to the following considerations (which were neglected in the reference analysis):

Table V.7. Comparison of Probabilities of Core Melt Due to Earthquake for an Average Site in the Eastern United States with 0.2g SSE.

Ground Acceleration	Newmark (safety) factor	Earthquake Probability per year	Reference Analysis			Our Analysis			
			Prob. of damage single system	Prob. of damage two systems	Core melt per reactor year (P_{CM})	Prob. of damage due to one path	Prob. of reactor damage due to ten failure paths	Core melt per reactor-year: minimal or no design errors	Core melt per reactor-year: maximal design errors
0.2g	20	7×10^{-4}	0.001	3×10^{-5}	2×10^{-8}	0.0015 to 0.017*	0.016 to 0.16*	1×10^{-5}	1×10^{-4}
0.5g	8	5×10^{-5}	0.02	3×10^{-3}	2×10^{-7}	0.02 to 0.12	0.19 to 0.72	9×10^{-6}	3×10^{-5}
1.0g	4	1×10^{-5}	0.1	3×10^{-2}	3×10^{-7}	0.083 to 0.30	0.58 to 0.97	6×10^{-6}	1×10^{-5}
				Total	5×10^{-7}		** Total	8×10^{-5}	3×10^{-4}

* The range of the numbers corresponds to "no design errors to maximal design errors."

** Including P_{CM}'s for 0.3, 0.4, 0.6, 0.7, 0.8, 0.9g.

(1) There are more than ten independent failure paths that could lead to a core melt accident.

(2) An earthquake would affect all the components simultaneously, and hence would have a stronger potential for common-mode failure.

(3) Seismic design and construction errors should be anticipated.

(4) In some systems or failure paths, the built-in safety factor could be lower than the Newmark factor.

Our analysis also indicates that using the assumed numerical values of lower safety factors for some failure paths, the resulting risk will be similar to the risk obtained by applying maximal design errors. (Tables V.4. and V.5). If we use a safety factor of 3 instead of 6 (which seems to be a more realistic value) for the group B failure paths, and keep the other factors the same, the core melt probability per reactor (P_D) for the case of no seismic design error would become 0.60, 0.88, ~1.0 for 0.2, 0.5 and 1.0g, respectively, or about equivalent to the worst case in our study.

If one assumes a complex system consisting of 100 independent components (in series in the reliability sense), each of which has a seismic safety factor of 20, or, correspondingly, a failure probability of 15×10^{-4} at SSE, the total failure probability of the system under SSE condition would be $1-(1-0.0015)^{100} \approx 0.14$. This is equivalent to a safety factor of 3, according to Figure V.1. If this is the case for group C, discussed above, the core melt probability per reactor (P_D) would be somewhat higher than that based on a safety factor of 10, as given in Table V.6.

For ground accelerations greater than that for SSE, we follow the Newmark rule, i.e., the safety factor will be reduced by 2 for every doubling of SSE, although as discussed in the snubber and hanger failure analysis, the reduction could be by a factor of 3, and the probability of core damage would then be higher.

In conclusion, the seismic risk of nuclear reactors could be significant, and the matter should be investigated in greater depth than has been done to date. Earthquake excitation combined with the existence of design errors and of reduced safety factors could be a major contributor in nuclear safety. Our analysis has been parametric in nature; a more definitive and refined analysis is needed.

It must be emphasized that the quantitative results reported herein depend heavily on the assumptions; for example, the use of earthquake probabilities from Ref. [64] (UCLA-ENG-7516) may lead to higher than expected accelerations at many sites. Similarly, the assumptions concerning reduction in safety margin for each category design error are arbitrary, as are the assumptions concerning original safety margin for some failure paths. Hence, the failure probabilities estimated herein could readily be calculated to be at least an order of magnitude lower.

The purpose of the study is primarily to illustrate a possible method for the incorporation of design errors in reliability analysis and safety assessment, and to look in some detail at the results reported for seismic risk in WASH-1400.

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APPENDIX III

(An abstract from the paper by N. Newmark, entitled "Overview of Seismic Design Margins," as given in Atomic Industrial Program Report Vol. 2, No. 1, Reactor Licensing and Safety).

Earthquake resistant design requires selection of earthquake hazards and structural strengths, either explicitly or implicitly, as an integral part of the design procedure. Unless the various factors affecting the site earthquake motions and the structural responses up to the allowable limits of response are selected in a consistent manner, the design procedures, whether they involve rational analytical methods or the use of building codes and specifications, may become grossly uneconomical, or at the opposite extreme dangerously unsafe, and in either case, irrational. The various decisions that must be made concerning the parameters to be used as a basis for earthquake resistant design may be selected on a probabilistic or a deterministic basis. The latter basis, using upper bounds or extreme conditions, leads to highly unreasonable requirements, and may alter the behavior of the structure in such a way that the end result can be a structure that has a lower capability for other design conditions that are important in the behavior of the facility.

The purpose of this paper is to review the bases for consistency in earthquake resistant design, beginning with the selection of the earthquake hazard and proceeding to the selection of resistance parameters, using design limits for the facility based upon its intended use and the consequences of its failure. Although the treatment in this paper is somewhat heuristic, it is based on a long series of studies and observations; when applied to ordinary buildings designed in accordance with current building codes, it appears to give results that are consistent with observations of the probability of failure of such buildings when they are subjected to their design earthquake.

Although this study is not yet completed, it appears that, for the same hazard conditions, an ordinary building, designed in accordance with current building codes, may have a static resistance 1 to 2 orders of magnitude lower than that of a nuclear power plant facility. For the same earthquake motion intensities, the ratio between the static strengths may be of the order of 5 to 10 because of the differences in allowable ductility and energy absorption in the response.

It is also shown in the paper that the probability of failure exceeding the design limit, defined herein as "failure", is of the order of 3 percent in the case of an ordinary building subjected to its design earthquake, and is of the order of 0.01 percent to less than 0.0001 percent for a nuclear power plant facility subjected to its design earthquake. In either case, to obtain the probability of failure, these probabilities must be multiplied by the probability of the design earthquake occurring. That probability may be of the order of 1 percent to 1 part per thousand per year for ordinary buildings and of the order of 1 part in ten thousand to 1 part in 10^5 per year for a nuclear facility.

Hence, the net probability of failure per year under seismic conditions will be less than about 3 parts in 10^4 for ordinary buildings designed in accord with the better current building codes, and considerably less, of the order of 1 part in 10^8 or less, for nuclear power plants.

Earthquake Hazard

In general, two procedures are available to define the earthquake hazard. In the first, where there is an extensive history of earthquake activity and geological and tectonic investigations are feasible, estimates can be made of the possible magnitude and location of future earthquakes affecting a site. In many instances, such earthquakes occur along well-defined faults. One can then make estimates of the earthquake motion intensity propagated to the site, taking into account the experimental and observational data available for this purpose. Finally, one can modify the intensities of motion in accordance with the geologic and stratographic conditions pertaining to the site.

The result of this exercise is a characterization of the motions at the site in terms of peak ground acceleration, sometimes also including the peak ground velocity; and a measure of the nature of the motions, either from a time history describing probable motions of the specified intensity at the particular site, or a response spectrum for the basic ground motion values.

The second procedure for developing the earthquake hazard at a particular site is used when occurrence of earthquakes in the particular region considered is not generally associated with surface faulting, or when insufficient data are available from records and measurements. Under these conditions, use is made of the general relationships that have been developed for relating ground motions, generally velocities but also accelerations, to the Modified Mercalli Intensity determined by observations of a qualitative nature. Although these relations do not appear to be as readily subject to mathematical determination as the relationships for earthquake shock propagation, there are sufficient observations to permit useful data to be obtained.

Resistance Parameters

In defining the resistance parameters for the structure, the first step is to consider the changes in the basic ground motion values that are caused by the interaction of the structure and its foundation. Then modified input motions to the structure are determined although this determination may depend in part on the structural response itself.

The behavior of the structure is determined by analyses which make use in some way of the response spectrum for the structure. This response spectrum can be determined for basic earthquake motions and used as a design input. However, in developing the response spectrum, the energy absorption in the structure caused by damping, inelastic action, or changes in properties with stress level, such as cracking of concrete., is involved.

Factor of Safety

The required resistance of the structure is then determined, taking into account the allowable deformation, based on the importance of the structure or the consequences of its failure. In order to resist the particular hazard chosen, with a "factor of safety" that appears appropriate for the structure or facility. There are actually factors of safety involved in all of these considerations: a factor of safety for the earthquake hazard, another factor of safety for the required resistance, and a net factor of safety which is essentially the product of these two.

The various quantities that enter into the selection of the design parameters are probabilistic in nature. If one were to select the median value of the various items constituting the earthquake hazard, there would be a 50% probability that this hazard would be exceeded in an actual earthquake. Also, if one were to select the resistance parameters at their median value, there would be also a 50% probability that these values would not be reached in half of the instances. If one were to design the earthquake to have a median resistance equal to the median hazard, one would have essentially a 50% probability of exceeding the design level. However, all of the parameters are chosen with regard to factors substantially reducing the probability level of exceeding the hazard, and increasing the probability of the structural resistance being greater than the selected increased value of hazard. The factor of safety defined herein is the ratio of the actual value used for whatever probability level is chosen, to the median value.