A METHOD FOR THE ESTIMATION OF THE PROBABILITY OF DAMAGE DUE TO EARTHQUAKES

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SUMMARY

A probabilistic method for the determination of the chance of structural damage to plants due to the occurrence of earthquakes is described.

A Poisson distribution for the occurrence of earthquakes has been assumed and the sensitivity of using either a truncated linear magnitude frequency law or one based on extreme values over finite time intervals has been investigated. The analysis has been carried out in terms of site intensity, this being considered to be the parameter of interest related to damage levels, and suitable attenuation relationships have been employed to construct an annual probability distribution of the exceedence of a particular site intensity in terms of the intensity at the site. The sensitivity of the results to several analytical approximations to this distribution has also been investigated.

The manner in which the occurrence statistics have been linked to the probability of damage to the plant is fully described. Damage evaluation studies of past earthquakes have indicated that the actual damage to a structure appears to fit a log normal distribution for a given site intensity. By a process of normalisation and substitution, it has been shown that the effective resistance to loading of the structure is normally distributed with respect to a normalised site intensity. This normal distribution can be completely defined by assuming a value for the standard deviation and a value for the failure probability at the design level, enabling the overall risk level per year of failure to be computed for a given design level. A range of values for the standard deviation covering structures, pipework and equipment has been used in the analysis.

Improvements have also been made to the model by incorporating the effects of uncertainty on the seismicity parameters. Correction factors for this uncertainty have been derived and applied to the results although in general the effects are small.

Application of the work to typical United Kingdom conditions shows that a reasonable approximation to the probability of intensity exceedence curve can be made by assuming a truncated second derivative type distribution. The difference between assuming a truncated linear magnitude frequency law and one based on extreme values has also been found to be small. The analysis has demonstrated that the contribution from the more frequent but less severe earthquakes is far from negligible in relation to the total risk of structural damage.

1. INTRODUCTION

The final design of a power or process plant is the result of a complex optimum balance between the expected benefit to be derived from the plant and the cost of construction and operation of the plant. The total cost of the plant includes the cost of design, erection and maintenance as well as the possible costs due to damage or failure of all or part of the system. A second and equally important balance in the design is the optimisation of the overall safety of the plant both during operation and under shutdown conditions. Included in the assessment of the safety of the plant is the question of the seismic hazard and this paper is intended to describe a method for the estimation of the probability of damage to a plant due to the occurrence of earthquakes.

In order to discuss the relative importance of the seismic hazard to a plant, the following items of information are required. a) The distribution of epicentres throughout the area of interest. b) The statistical frequency of occurrence of seismic events throughout the area of interest. c) A measure of a parameter related to the potential for causing damage (e.g., intensity, peak acceleration etc.) d) An indication of how this parameter attenuates with distance. e) An estimate of the capacity or resistance of the structure to the applied loading.

A method is described using available United Kingdom data to determine the overall failure characteristics of typical plant items when subjected to seismic events.

2. GEOLOGICAL SETTING

The geological setting of the UK has been adequately described by Burton [1] and is briefly summarised for completeness. Due to the fact that North West Europe is situated within the Eurasian plate in the mid-plate position, the major earthquakes associated with plate boundaries are not experienced. The most striking feature of the UK is the Great Glen fault dividing the North West Highlands from the Central Highlands. There is clear correlation between the line of the Great Glen and local earthquake epicentres. Elsewhere it is not possible to demonstrate such clear correlation with the data currently available.

3. DATA COMPILATION

In order to carry out an investigation into the seismic hazard, the spatial and temporal distribution of earthquake foci is required. The history of data compilation in the UK has been described by Burton [1] and the way in which pre-instrumental data has been evaluated is explained. A data file containing the basic parameters of date, time, epicentre, depth and magnitude for each earthquake has been created by the Institute of Geological Sciences (I.G.S.) and the details have been described by Lilwall [2]. The resulting file covers the last 1000 years but the coverage naturally varies considerably. It is reasonable to assume from observation of the number of events per year that the period since 1800 represents a uniform completeness of the file.

4. FREQUENCY OF OCCURRENCE

The occurrence of earthquakes in space and time falls under the general heading of stochastic processes. The two most popular processes used to describe earthquake occurrences are the Markov and Poisson processes, and they differ in that the former assumes that the probability of future events is dependent on the past events, whereas the latter implies complete independence of each event.

Although more consideration has been given in recent times to the Markov process (Shah and Vagliente [3], Vagliente [4]), due to its general compatibility with the Elastic Rebound Theory, the Poisson model (Lomnitz [5, 6], Gardner and Knopoff [7]) is still the most popular model used for seismic risk evaluation, mainly due to its simplicity in concept and treatment. Adoption of the Poisson model necessarily precludes aftershock sequences from any analysis and also assumes that the pattern of seismic activity is constant with time.

The frequency of occurrence of earthquakes can be easily described by Richter's Law in the form

$$\mathrm{Log}\, N = a - b\, m_b \qquad (1)$$

where N is the number of events per year greater than or equal to m_b, and m_b is the body-wave magnitude, a and b are constants. Lilwall [2] has shown that for the whole of the UK the following relationship holds

$$\mathrm{Log}\, N = 4.13 - 1.09\, m_b \quad , \qquad 3.25 \leqslant m_b \leqslant 5.5 \tag{2}$$

The disadvantage of this cumulative frequency approach is that it does not directly lead to a prediction of a maximum value and that ideally it requires information on events down to the threshold of the analysis. To overcome this disadvantage, a different approach can be adopted based on the analysis of earthquake extreme values over equal time intervals. This approach automatically overcomes any objections due to the presence of aftershocks and also mitigates the influence of swarms of events. Gumbel's third distribution of extreme values [8] allows for the roll-off at the high magnitude range and also allows for an upper magnitude limit to the earthquake population. Using techniques described by Yegulalp and Kuo [9], Lilwall [2] showed that the distribution for five year extremes was

$$P_5 = \exp\left\{-\left(\frac{5.72 - m_b}{1.29}\right)^{2.22}\right\} \tag{3}$$

Using the relationship between annual extremes and n-year extremes of

$$P_n = P_1^{\,n} \tag{4}$$

we have $$P_1 = \exp\left\{-\frac{1}{5}\left(\frac{5.72 - m_b}{1.29}\right)^{2.22}\right\} \tag{5}$$

and the probability that m_b is exceeded in any year is given by

$$P = 1 - \exp\left\{-\frac{1}{5}\left(\frac{5.72 - m_b}{1.29}\right)^{2.22}\right\} \tag{6}$$

5. ATTENUATION RELATIONSHIPS

The parameter of interest related to damage level in this model is taken to be Modified Mercalli Intensity and it is necessary to have a relationship between intensity and focal distance for a given magnitude. Karnik [10] found for Northern Europe that the epicentral intensity was related to focal depth and magnitude in the following manner

$$I_o = 1.9 + 1.5M - 0.9 \ln h \tag{7}$$

where M is the surface wave magnitude and h (km) the focal depth. Esteva and Rosenbleuth [11] have suggested the following form for the attenuation of intensity with distance

$$I = B + 1.5M - 0.95 \ln\left(h^2 + r^2 + k^2\right) \tag{8}$$

where B is a constant, r is the epicentral distance (km) and k is an empirical constant to give better agreement over short distances. Esteva [12] has suggested a value of 20 for k and this value has been used in the analysis.

Eqs. (7) and (8) have been combined and the constant B eliminated to give consistency at the epicentre, leading to the following attenuation law given the epicentral value which is obtained from eq. (7)

$$I = I_o - 0.95 \ln\left(1 + \frac{r^2}{h^2 + 400}\right) \tag{9}$$

In order to convert the surface wave magnitude M to the short period body wave value m_b, the results of Marshall [13] have been used where

$$M = 2.08\, m_b - 5.65 \tag{10}$$

6. SEISMIC LEVEL ASSESSMENT

The general methodology used to establish the probability of exceedence of a particular site intensity level is based on the method developed by Cornell [14]. It is assumed that the seismicity is uniformly distributed throughout the UK and that future earthquakes will occur randomly with respect to location and time. The range of site intensity is divided up into a number of discrete intervals, denoted by I_i. The range of epicentral intensities is also divided up into a number of discrete intervals, denoted by I_{oj}. Associated with each value of I_i is an elemental surface area a_{ij} for an earthquake with epicentral intensity I_{oj} which can be determined from eq. (9). If the total area of the region being considered is A, then the probability that an arbitrary point A will experience a site intensity I_i when exactly one earthquake with epicentral intensity I_{oj} has occurred is:-

a_{ij}/A

Given that an earthquake with epicentral intensity I_{oj} has occurred, the probability that a point within A will not experience intensity I_i is:-

$1 - a_{ij}/A$

For the case of n earthquakes per year with epicentral intensity I_{oj}, the probability of not experiencing I_i is:-

$(1 - a_{ij}/A)^n$

We assume that the occurrence of earthquakes follows a Poisson distribution for the magnitude range under consideration and since for a given focal depth, the epicentral intensity can be related to the magnitude by a simple linear equation, then the probability of exactly n earthquakes with epicentral intensity I_{oj} occurring is

$$(\lambda_j^n/n!)\exp(-\lambda_j)$$

where λ_j is the expected number of earthquakes with epicentral intensity I_{oj} in one year. The probability that any point in the total area A will not experience an intensity I_i is therefore

$$\sum_{n=0}^{\infty}(1 - a_{ij}/A)^n \frac{\lambda_j^n}{n!}\exp(-\lambda_j)$$

$$= \exp(-a_{ij}\lambda_j/A)$$

Taking into account all the possible epicentral intensities I_{oj}, then the probability of not experiencing intensity I_i is therefore

$$\prod \exp(-a_{ij}\lambda_j/A) = \exp\left[\left(-\frac{1}{A}\right)\sum_{\text{all } j} a_{ij}\lambda_j\right]$$

The probability that a random site will experience an intensity greater than or equal to I_i in one year is therefore

$$P(I_i) = 1 - \exp\left[-\left(\frac{1}{A}\right)\sum_{\text{all } j} a_{ij}\lambda_j\right] \tag{11}$$

The value of a_{ij} is based on the epicentral distance and hence using eq. (9)

$$a_{ij} = \pi r^2 = \pi(h^2+400)\left(\exp((I_{oj}-I_i)/0.95)-1\right) \tag{12}$$

By combining eqs. (2), (7) and (10), the number of earthquakes occurring with epicentral intensity between I_{oj} and $I_{oj} + \delta I_{oj}$ is given by

$$\lambda_j = |\delta N_j| = 54.76(h)^{-0.724}\exp(-0.804 I_{oj})\,\delta I_{oj} \tag{13}$$

Inserting eqs. (12) and (13) into (11) and taking to the limit we have:-

$$P(I_i) = 1 - \exp\left[\frac{-54.76\pi(h^2+400)}{A(h)^{0.724}}\int_{I_i}^{I_L}\left(\exp((I_{oj}-I_i)/0.95)-1\right)\exp(-0.804 I_{oj})\,dI_{oj}\right] \tag{14}$$

where I_L is the upper limit of intensity, a predetermined value, or that value of I_{oj} that causes a_{ij} to equal the total area A, whichever is the smaller.

A similar exercise has been carried out using the extreme value distribution of eq. (6) in place of eq. (2) leading to an alternative expression:-

$$P(I_i) = 1 - \exp\left[\frac{-0.267\pi(h^2+400)}{A}\int_{I_i}^{I_L}\left(\exp((I_{oj}-I_i)/0.95)-1\right)(1.36-0.11\ln h-0.12 I_{oj})^{1.22} \times \exp\left(-(1.36-0.11\ln h-0.12 I_{oj})^{2.22}\right)dI_{oj}\right] \tag{15}$$

I_L is defined as before

Assuming a mean epicentral depth of 10 km and A = 230000m^2, eqs. (14) and (15) have been evaluated for a range of intensity levels and are shown in Fig. 1.

7. OVERALL RISK

In this section the uncertainty of the damage is linked to the uncertainty of the earthquake occurrence rate. The model developed here is based on the method described by Veneziano /15/. The seismic load at a site is characterised by the mean occurrence rate and the probability distribution of site intensity I.

If $F_I(i) = P\{I \leq i \mid \text{the occurrence of a seismic event}\}$ is defined as the cumulative distribution function of the site intensity whenever an earthquake occurs, then the mean rate of events with site intensity larger than i is:-

$$\lambda_i = \lambda[1 - F_I(i)] \tag{16}$$

Now if $F_{D|I}(d|i) = P\{D \leq d | I = i\}$ is defined as the cumulative distribution function of the damage caused by an earthquake with site intensity i, then the mean rate of events which cause damage in excess of d is:-

$$\lambda_D(d) = \lambda \int_{-\infty}^{\infty} [1 - F_{D|I}(d|i)]\, dF_I(i) \tag{17}$$

We are interested in the mean rate of events beyond some critical value, d_f say, which represents 'failure'. Therefore, the mean failure rate is given by:-

$$\lambda_f = \lambda_D(d_f) = \lambda \int_{-\infty}^{\infty} [1 - F_{D|I}(d_f|i)]\, dF_I(i) \tag{18}$$

where $[1 - F_{D|I}(d_f|i)] = P_f(i)$ is the probability of failure at intensity i.

There are several ways in which the failure probability can be evaluated and these have been discussed by Veneziano /15/. It is the intention here to base probability on actual damage statistics and related work. Benjamin /16/ found that for broad classes of buildings ranging from wooden frame dwellings to high rise buildings, the damage data for a given intensity fitted a log-normal distribution. It is assumed that the initiating event in a plant is likely to involve similar characteristics to those of conventional structures.

The probability of log d_f, the threshold value being exceeded is equal to the probability of failure, given that an event of site intensity I occurs and is given by:-

$$P_f(I) = 1 - \Phi\left[\frac{\log d_f - \log D_m}{\sigma_D}\right]$$

where $\Phi(.)$ is the normal cumulative distribution function, D_m is the mean value of damage and σ_D is the standard deviation. log D_m has been found to be essentially linear with intensity and the variance approximately constant with intensity. Therefore D_m can be expressed as $\log D_m = a_D + b_D I$ and $P_f(I)$ can be expressed therefore as

$$P_f(I) = 1 - \Phi\left[\frac{\log d_f - (a_D + b_D I)}{\sigma_D}\right] \tag{19}$$

which can be rearranged to give

$$P_f(I) = 1 - \Phi\left[\frac{\mu_R - I}{\sigma_R}\right] \quad \text{where } \mu_R = (\log d_f - a_D)/b_D \text{ and } \sigma_R = \sigma_D / b_D \tag{20}$$

The parameters μ_R and σ_R represent the mean value and the variance respectively of a 'resistance' parameter in units of intensity and by its form is also normally distributed.

In order to determine the overall probability of failure of a structure or system it is necessary to define the characteristic represented by eq. (20). This can be specified by giving a design level of site intensity I_{SSE}, a probability of failure at the design condition p, say, and a value of σ_R deduced from experience. μ_R is then determined from eq. (20).

Development of the method to determine overall failure probabilities is simplified by converting to normalised intensities, defined as $i_N = (i - \mu_R)/\sigma_R$. The overall failure probability can then shown to be:-

$$\lambda_f = \int_{-\infty}^{\infty} - \phi(i_N)\, \lambda_{i_N}\, di_N$$

and since i_N is normally distributed

$$\lambda_f = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{\infty} \exp\left(-\tfrac{1}{2} i_N^2\right) \lambda_{i_N} \, di_N \tag{21}$$

Eq. (21) would be difficult to evaluate without making some simplifying assumption about the form of λ_{i_N}. Following Veneziano /15/, different approximations were fitted to the data and to illustrate this two examples are given here, namely a simple linear approximation and a truncated second derivative law.

a) Simple linear approximation

This can be expressed as:-

$$\lambda_i = \lambda \exp\left(-\beta_I i\right) \tag{22}$$

The mean failure rate is then obtained from eq. (21) as

$$\lambda_f = \lambda \exp\left(-\beta_I \mu_R\right) \exp\left(\tfrac{1}{2} \beta_I^2 \sigma_R^2\right) \tag{23}$$

b) Truncated second derivative law

This can be expressed as:-

$$\lambda_i = \begin{cases} \lambda\left[\exp(-\beta_I i) - \exp(-\beta_I I_c) - \beta_I (I_c - i) \exp(-\beta_I I_c)\right], & i \leq I_c \\ 0, & i > I_c \end{cases} \tag{24}$$

where I_c is the cut-off intensity.

The mean failure rate is then obtained from eq. (21) as

$$\lambda_f = \lambda \exp(-\beta_I \mu_R)\left[\exp\left(\frac{\beta_I^2 \sigma_R^2}{2}\right) \Phi(i_1 + \beta_I \sigma_R) - \exp(-\beta_I \sigma_R i_1)\left[(1 + \beta_I \sigma_R i_1)\Phi(i_1) + \beta_I \sigma_R \phi(i_1)\right]\right], \quad i_1 = (I_c - \mu_R)/\sigma_R \tag{25}$$

Eqs. (22) and (24) are shown in Fig. 2 compared with eq. (15) evaluated for the extreme value magnitude distribution. Further analysis is confined to the use of the truncated second derivative relationship.

8. RESISTANCE CONSIDERATIONS

The calculation of the overall risk is very dependent upon the assumptions made for the characteristics of the structural resistance function. A study by Ang and Newmark /17/ on the Diablo Canyon Nuclear Power Plant has outlined a method of assessment based on many years experience in the field. The probability distribution for the resistance was prescribed to be log-normal and was based on peak accelerations as the independent variable.

$$P_f(a_g) = \Phi\left[\frac{\ln(a_g/\bar{r})}{\zeta_R}\right] \tag{26}$$

in which a_g is the design level of peak acceleration, $\bar{r}$ is the median level of peak acceleration, ζ_R is the variance given by $\zeta_R^2 = \ln(1+\Omega_R^2)$ and Ω_R is the coefficient of variation representing the uncertainty in the resistance. Eq. (26) can be shown to be similar to eq. (20) by assuming one of the relationships between intensity and peak acceleration (Ambraseys /18/, Trifunac and Brady /19/) in which log a = constant + 0.3I. From this it can be shown that $\sigma_R = 1.448\,\zeta_R$ and $\mu_R = I_{SSE}$ + (log FS)/0.3 where FS the factor of safety is given by $\bar{r}/a_{g_{SSE}}$ ie the ratio of the median value to the design peak acceleration.

According to Newmark /20/ structures and structural components of nuclear power plant have a factor of safety of between 4 and 6 up to damage of a yielding nature. For damage up to fracture or collapse the factor of safety has been taken to be 10. For pipework, this factor has been taken to be 12 and for equipment the factor has been taken to be 4. Values for the variance of these items have been taken from reference /17/ and these are respectively 0.59, 0.71 and 0.85. The data can be summarised as

a) Structures $\mu_R = I_{SSE} + 3.33$, $\sigma_R = 0.85$

b) Pipework $\mu_R = I_{SSE} + 3.6$, $\sigma_R = 1.03$

c) Equipment $\mu_R = I_{SSE} + 2.0$, $\sigma_R = 1.23$

Eq. (25) has been evaluated over a range of design levels of intensity for these three groups of plant items and the results shown in Fig. 3. It can be argued that for an accident sequence to lead to a major release, then two independent items of equipment would have to fail. In this case the overall failure probability would be λ_f^2 calculated from eq. (25) and this result is also shown in Fig. 3.

9. INFLUENCE OF UNCERTAINTY ON OCCURRENCE RATE

The model developed up to this stage is based on the available statistical evidence. However, since there are uncertainties inherent in the mean occurrence rate and the attenuation law, an improvement can be made by including some representation of these uncertainties. The method described by Veneziano [15] has been used and this implies a penalty factor on the overall failure probability. A normalised intensity i_d is defined such that the mean occurrence rate is independent of $\beta_I \sigma_{\bar{R}}$ and this is assumed to occur at an intensity of 4.5. The penalty factor is given by:-

$$PF = \frac{1}{\sqrt{1-\sigma_{\bar{\beta}_N}^2}} \exp\left\{\tfrac{1}{2}\beta_I^2\sigma_{\bar{R}}^2\sigma_{\bar{\beta}_N}^2/(1-\sigma_{\bar{\beta}_N}^2) + \tfrac{1}{2}\sigma_D^2 + \tfrac{1}{2} i_d \sigma_{\bar{\beta}_N}^2 (i_d + 2\beta_I\sigma_{\bar{R}})/(1-\sigma_{\bar{\beta}_N}^2)\right\} \tag{25}$$

Typical values of $\sigma_{\bar{\beta}_N}$ and $\sigma_{\bar{d}}$ are 0.2 and 0.3 respectively. The penalty factor has been applied to the results shown in Fig. 3, and are shown in Fig. 4.

10. DISCUSSION OF RESULTS

The results show that there is little difference in the probability of exceedence of an intensity at a site between choosing a linear magnitude-frequency law with cut-off and one based on extreme values. This is demonstrated in Fig. 1. The truncated second derivative approximation to the site intensity exceedence curve is shown to be reasonable in Fig. 2.

Figs. 3 and 4 show the overall failure probability distributions uncorrected and corrected for uncertainty in the occurrence rate and indicate the similarity between the behaviour of structures and pipework. A single piece of equipment would be dominant in the design if failure could lead to complete release of the inventory but, if as has been argued two independent items of equipment must fail before complete release is possible, then the behaviour of the pipework or structure is dominant.

11. CONCLUSIONS

A method for the determination of the probability of damage due to the occurrence of earthquakes given the failure characteristics of the plant items has been described. The method enables the selection of an intensity value for the design of a hazardous plant given the overall failure probability required. The intensity value can be converted to peak acceleration in order to select an appropriate design response spectrum, or enable an intensity related design response spectrum to be used.

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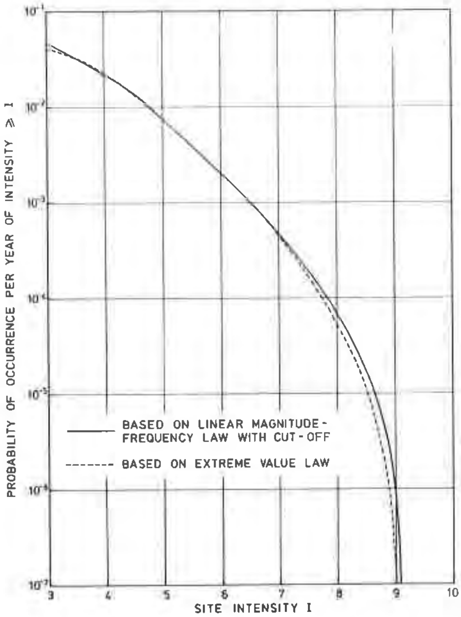

FIGURE 1. PROBABILITY OF EXCEEDENCE OF INTENSITY I vs. I

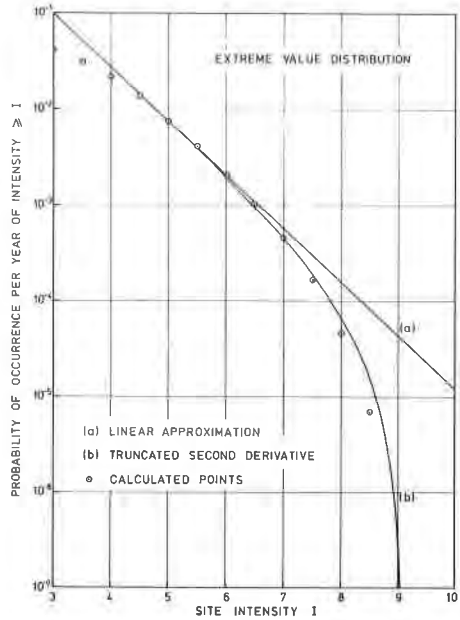

FIGURE 2. APPROXIMATIONS TO PROBABILITY OF EXCEEDENCE CURVE

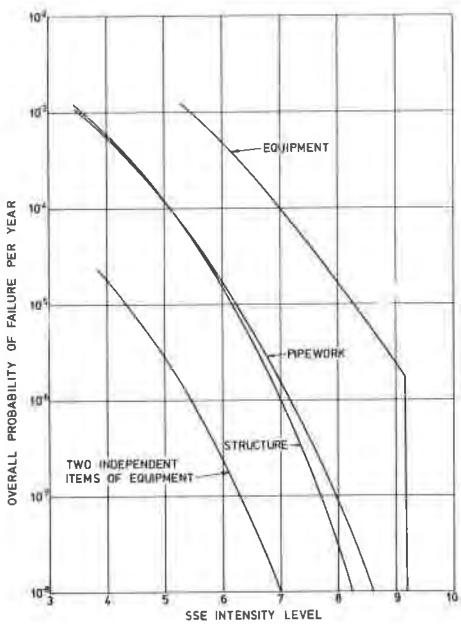

FIGURE 3. OVERALL PROBABILITY OF FAILURE vs. DESIGN INTENSITY

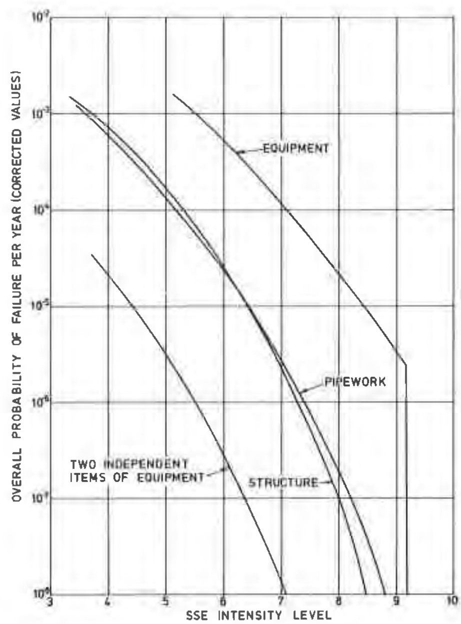

FIGURE 4. CORRECTED OVERALL PROBABILITY OF FAILURE vs. DESIGN INTENSITY