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ABSTRACT

This report describes a method for the assessment of the reliability of low-rise shear wall structures, which are often used in nuclear power plants. The shear walls are modeled by stick models with beam elements, and are subjected to dead load, live load and earthquake during their lifetimes. The earthquake load is assumed to be a segment of a stationary Gaussian process with a zero-mean and a Kanai-Tajimi power spectral density function. The seismic hazard at a site, represented by a hazard curve, is also included in the reliability analysis.

Both shear and flexure limit states are analytically defined. The flexure limit state is defined according to the ACI strength design formula, while the shear limit state is established from test data. The reliability analysis methodology is described in detail. Illustrative examples are given to demonstrate the method and the applications. This reliability analysis method can also be used to generate the fragility curve of the shear wall structure.

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EXECUTIVE SUMMARY

This report details an addition to the probability-based analysis methodology for structures that has been developed by the Structural Analysis Division of Brookhaven National Laboratory (BNL). Up to the present time the reliability analysis methods were confined to safety and fragility evaluations of reinforced concrete containment structures. This report deals with the reliability analysis method developed for shear wall structures.

Shear walls are used extensively in nuclear power plant structures and many are classified as seismic category I structures. The reliability analysis method described in this report can be utilized to evaluate the safety of shear walls under dead load, live load and earthquake excitation. Both shear as well as flexure limit states are analytically defined. The flexure limit state is defined according to the ACI strength design formula, and the shear limit state is established from test data. All of the formulations together with the reliability analysis methodology and several sample solutions are detailed in the text of the report. This additional extension of the reliability method can be applied by NRC for safety evaluations involving reliability and fragility of shear wall structures.

1. INTRODUCTION

Shear walls have been used extensively in nuclear power plants, and many of them are classified as seismic category I structures. Their functions are to provide shielding against radiation, protection from missile impact, and resistance against lateral loads such as earthquake. In addition to lateral loads, shear walls are also required to support dead and live loads acting on floor slabs supported by the walls. Although the reliability of a shear wall depends on the combination of all the load effects which may act on the wall, it is the intention of this report to assess the reliability of the shear wall subjected to the loads acting in the plane of the shear wall structure, namely, vertical dead and live loads combined with the in-plane seismic load.

In the following sections, the shear wall structures used in this study are described first. Then the probabilistic nature of the loads and material strengths is presented. Next, the limit states for shear and flexure are discussed in detail. The reliability analysis method for shear walls is also presented. Illustrative examples are given to demonstrate the method and its applications.

2. DESCRIPTION OF THE SHEAR WALL STRUCTURES

The shear walls in nuclear power plants usually exist in the shape of a closed or open box or as a simple rectangular wall. For the purpose of analysis and design, a flanged shear wall is often used instead of the box-shaped shear wall. This is a reasonable simplification if earthquake excitations in two horizontal directions are considered separately. The height-to-length ratio of these walls ranges from 1/2 to 1-1/2. The ratio is low in contrast to a much larger value for the walls used in conventional buildings. The thickness of the wall may range from 2 to 5 feet. However, these wall thicknesses are, generally, not based on the strength requirement but rather on the safety requirement against radiation and tornado-borne missiles. In order to assess reliability based on the strength consideration, the shear walls designed according to the strength requirement of ACI code[1] should be used. In this study, three-story low-rise shear walls with and without flanges are considered and a sketch of such a shear wall is shown in Fig. 1.

3. PROBABILISTIC REPRESENTATION OF LOADS

The loads considered in this study consist of dead, live and seismic loads. Their probabilistic nature is discussed in the following sections.

3.1 Dead Load

Dead load consists of the weight of the structural elements, attachments, and permanent equipment installations. In general, dead load is modeled as a random variable and remains on the structure throughout the life of a plant as shown in Fig. 2a. The variability of the dead load arises from the variations of the unit weights of construction materials and the variation in dimensions of the structural components deviated from the dimensions used for design. Additional uncertainties are also introduced from items that are averaged in the dead load rather than calculated explicitly by the designer. For ordinary building constructions, the mean value, $\bar{D}$, has been assumed to be 1.0 to 1.05 times the nominal value, D_n, and coefficient of variation, CoV (D), has been taken as 0.07 to 0.10.[2] For nuclear structures, a mean value $\bar{D} = 1.0D_n$ and a CoV (D) = 0.07 have been suggested.[3] These values are used in this study together with the assumption that the distribution of the dead load is normal.

3.2 Live Load

Live loads in nuclear power plants represent any temporary loads resulting from human occupancy, movable equipment and other operational or maintenance conditions. Significant live loads might arise from temporary equipments or materials stored on floors during maintenance or repair within the plant. Live load is modeled as a Poisson renewal rectangular pulse process as shown in Fig. 2b. The essential parameters for this model are mean occurrence rate, mean duration and the probability distribution of the point-in-time intensity.

Measurements of live loads in nuclear power plants were not readily available. Statistical data on live loads were obtained from a limited number of responses to a questionnaire used as part of a consensus estimation survey of loads in nuclear power plants.[4,5] The live load data from the consensus estimation survey were analyzed as shown in the Appendix A of Ref. 3. Considering both PWR and BWR plants, the mean value of the maximum live load to occur in 40 years is 0.81 times the nominal live load and its coefficient of variation is 0.37.[3] This mean value is consistent with the mean value of maximum live load during 40 years for ordinary building construction[2], which were obtained by analyzing measurements of live load rather than responses to a survey. With a mean duration of three months, several statistics for the point-in-time live load corresponding to different occurrence rates were presented.[3] In this study, the occurrence rate is taken to be 0.5 per year. Thus, the point-in-time live load intensity is modeled by a gamma distribution with a mean value of 0.36 times the nominal design value and a coefficient of variation of 0.54.

3.3 Earthquake Load

The seismic hazard at a site of a nuclear power plant is described by a seismic hazard curve, a plot of the annual exceedance probability $G_A(a)$ vs. the peak ground acceleration as shown in Fig. 3. The probability distribution $F_A(a)$ of the annual peak ground acceleration A is assumed to be a Type II extreme value distribution[6],

$$1 - G_A(a) = F_A(a) = \exp[-(a/\mu)^{-\alpha}] \qquad (1)$$

where α and μ are two parameters to be determined. The average value of α for the U.S. is estimated to be 2.7.[7] The parameter μ is computed based on this α value and the assumption that the annual probability of exceeding the safe shutdown earthquake (SSE) at a site is 4 x 10^{-4} per year.[8]

The lower and upper bounds of peak ground acceleration are required in the analysis. The lower bound, a_0, indicates the minimum peak ground acceleration for any ground shaking to be considered as an earthquake. a_0 is assumed to be 0.05 g. The upper bound, a_{max}, represents the largest earthquake possible at a site. However, the state-of-the-art in seismology can not precisely determine the value of a_{max}. The effect of a_{max} has been demonstrated in NUREG/CR-3876,[7] when we dealt with concrete containments. It was shown that a_{max} has a significant effect on the results of reliability analysis. In this study, a_{max} is chosen to be three times the peak ground acceleration of the safe shutdown earthquake a_{SSE}.

Earthquakes are modeled by a Poisson renewal process as shown in Fig. 2c. The ground acceleration, on the condition that an earthquake occurs, is idealized as a segment of a zero-mean stationary Gaussian process, described in the frequency domain by a Kanai-Tajimi power spectral density[9],

$$S_{gg}(\omega) = S_0 \frac{1 + 4\zeta_g^2(\omega/\omega_g)^2}{[1 - (\omega/\omega_g)^2]^2 + 4\zeta_g^2(\omega/\omega_g)^2} \qquad (2)$$

where the parameter S_0 is a random variable which represents the intensity of an earthquake. Parameters ω_g and ζ_g are the dominant ground motion frequency and the critical damping, respectively, which depend on the site soil conditions.[6] In this study, ω_g and ζ_g are taken to be 8π rad/sec and 0.6, respectively. In addition, the mean duration μ_{dE} of the stationary phase of the earthquake acceleration is assumed to be 20 seconds.

The value of S_0 can be determined based on the peak ground acceleration of an earthquake, A_1, as follows:[10]

$$A_1 = p_g \sigma_g \qquad (3)$$

where p_g is the peak factor which is assumed to be 3.0 in this study. The standard deviation of the ground acceleration, σ_g, computed by integrating the Kanai-Tajimi spectral density function with respect to ω, is

$$\sigma_g = \sqrt{\pi\omega_g(\frac{1}{2\zeta_g} + 2\zeta_g)} \sqrt{S_0} \qquad (4)$$

Hence, Eq. 3 can be rewritten as

$$A_1 = \alpha_g \sqrt{S_0} \qquad (5)$$

where

$$\alpha_g = p_g \sqrt{\pi\omega_g(\frac{1}{2\zeta_g} + 2\zeta_g)} \qquad (6)$$

If the earthquake occurs in accordance with the Poisson law at a rate λ_E per year, the probability distribution $F_A(a)$ is related to the probability distribution $F_{A_1}(a)$ in the following fashion:

$$F_A(a) = \exp\{-\lambda_E[1 - F_{A_1}(a)]\}$$

or (7)

$$F_{A_1}(a) = 1 + \frac{1}{\lambda_E} \ln F_A(a)$$

Since a_o indicates the minimum peak ground acceleration for any ground shaking to be considered as an earthquake, $F_{A_1}(a_o) = 0$ and hence, $\lambda_E = - \ln F_A(a_o)$. From Eqs. 1 and 7, we obtain

$$F_{A_1}(a) = 1 - (a/a_o)^{-\alpha} \; ; \; a \geqslant a_o \qquad (8)$$

Combining Eqs. 5 and 8, and writing Z for $\sqrt{S_o}$, we obtain the probability distribution and density function of Z in the following forms, respectively,

$$\begin{aligned} F_Z(z) &= 1 - (\alpha_g z/a_o)^{-\alpha} \\ f_Z(z) &= \alpha(\alpha_g/a_o)(\alpha_g z/a_o)^{-(\alpha+1)} \end{aligned} \qquad \text{for } z \geqslant a_o/\alpha_g \qquad (9)$$

4. PROBABILISTIC REPRESENTATION OF MATERIAL PROPERTIES

The resistance of shear wall structures depends primarily on the strength of the construction material, namely concrete and reinforcing steel. The strength of concrete and reinforcing steel are probabilistic in nature. Based on Ref. 11, the material properties of concrete and reinforcing steel are summarized as follows.

4.1 Concrete

The unit weight of concrete is taken to be 150 lb/ft^3. Young's modulus and Poisson's ratio for concrete are 3.6 x 10^6 psi and 0.2, respectively. The concrete compressive strength, f'_c, is assumed to be normally distributed with a coefficient of variation, CoV (f'_c) of 0.14 and a mean value at the age of 1 year, $\bar{f}'_c$,

$$\bar{f}'_c = 1219 + 1.02 \, f'_{cn} \quad (\text{psi}) \qquad (10)$$

in which f'_{cn} is the specified compressive strength of concrete at 28 days. For f'_{cn} = 4000 psi, the mean value of concrete compressive strength is 5299 psi.

4.2 Reinforcing Steel

The yield strength f_y of ASTM A 615 grade 60 deformed reinforcement is assumed to have a lognormal distribution with a mean value of 71.0 ksi and CoV (f_y) equal to 0.11. Young's modulus and Poisson's ratio are taken to be 29.0×10^6 psi and 0.3, respectively.

5. LIMIT STATES

A limit state (failure mode) represents a state of undesirable structural behavior. In general, a limit state is defined from the actual structural behavior under loads. For a particular structural system, it is probable that more than one limit state may have to be considered. For example, limit states of a low-rise shear wall may include flexure, shear, sliding and buckling. In this study, the sliding and buckling failures of a shear wall are discounted. A typical shear wall in nuclear structures is massive and low. Thus, the buckling failure is very rare. Resistance to sliding failure is provided by aggregate interlock and dowel action of vertical reinforcement and boundary elements. For a low-rise massive shear wall with proper boundary elements, the sliding failure of such a shear wall is also negligible. In this study, the shear and flexure limit states are considered and defined below.

5.1 Flexure Limit State

The flexure limit state for shear walls is analytically defined according to the ultimate strength formula. Figure 4 shows typical strain and stress distributions for a shear wall. On the basis of these strain and stress distributions, the flexure limit state is defined as follows: At any time during the service life of the structure, the state of structural response is considered to have reached the limit state if a maximum concrete compressive strain at the extreme fiber of the cross-section is equal to 0.003, while the yielding of rebars is permitted. Based on the above definition of the limit state, for a given geometry and rebar arrangement, a limit state surface can be constructed in terms of the axial force and bending moment of a cross-section. A typical limit state surface which is approximated by a polygon is shown in Fig. 5. In this figure, point "a" is determined from the stress state of uniform compression. Points "c" and "c'" are the so-called "balanced points", at which a concrete compression strain of 0.003 and a steel tensile strain of f_y/E_s are reached simultaneously. Points "e" and "e'" are determined from zero axial force. Furthermore, lines abc and ab'c' in Fig. 5 represent compression failure and lines cde and c'd'e' represent tension failure.

The flexure limit state surface essentially represents the flexural capacity of a shear wall. Since the flexural capacity is calculated using the ultimate strength formula of reinforced concrete, the variability of the capacity is considered to be primarily caused by the variations of concrete compressive strength and rebar yield strength as described in Section 4.

5.2 Shear Limit State

Shear limit state is reached when either concrete is crushed by diagonal compression or rebars are fractured by diagonal tension after the formation of the diagonal cracks in a shear wall.

The unit ultimate shear strength of a shear wall, v_u, is expressed as

$$v_u = v_c + v_s \tag{11}$$

in which v_c and v_s are the contributions of concrete and web reinforcement to the unit ultimate shear strength.

Barda, et al.,[12] conducted tests on eight specimens representing low-rise shear walls with boundary elements and suggested that for shear walls with height-to-length ratio h_w/ℓ_w between 1/4 and 1, v_c could be given as,

$$v_c = 8.3\sqrt{f'_c} - 3.4\sqrt{f'_c}\left(\frac{h_\omega}{\ell_\omega} - \frac{1}{2}\right) + \frac{N_u}{4\ell_\omega h} \quad ; \quad \frac{1}{4} \leq \frac{h_\omega}{\ell_\omega} \leq 1.0 \tag{12}$$

in which h_w and ℓ_w are the overall height and overall length of the shear wall. h is the wall thickness and N_u is the axial force taken as positive in compression. Barda, et al., also concluded that for shear walls with a height-to-length ratio of 1/2 and less, the horizontal wall reinforcement, which is effective for high-rise shear walls, did not contribute to shear strength. On the other hand, vertical wall reinforcement was effective as shear reinforcement in shear walls with height-to-length ratio of 1/2 and 1/4. However, it was less effective if height-to-length ratio was equal to 1.

In view of these findings, i.e., the contribution of the horizontal and vertical reinforcements varies according to different height-to-length ratios, the following equation for v_s is recommended[13],

$$v_s = (a\rho_h + b\rho_n)\, f_y$$

where ρ_h and ρ_n are horizontal and vertical web reinforcement ratios, respectively. The constants a and b are determined as follows:

$$b = \begin{cases} 1 & ; \frac{h_\omega}{\ell_\omega} < 1/2 \\ 2 - 2\frac{h_\omega}{\ell_\omega} & ; \frac{1}{2} \leq \frac{h_\omega}{\ell_\omega} \leq 1 \\ 0 & ; \frac{h_\omega}{\ell_\omega} > 1 \end{cases} \qquad (14)$$

and

$$a = 1 - b$$

Both horizontal and vertical rebars are actually still somewhat effective outside the given limits, but Eq. 14 is not sensitive to these limits as long as both kinds of rebars are used.

Gergely[14] also suggested that a low-rise shear wall would result in diagonal concrete crushing if the shear stress is larger than the following unit ultimate shear strength.

$$v_u = 0.25\ f_c' \qquad (15)$$

In this study, all the suggestions are taken into consideration, and thus, the unit ultimate shear strength is the smaller one determined from Eqs. 11-14 or Eq. 15. The total ultimate shear strength, V_u, is computed as

$$V_u = v_u\ h\ d \qquad (16)$$

where h is the wall thickness and d is the effective depth which is taken to be 0.8 ℓ_w for rectangular walls or the web length between flanges for wall with flanges. From Eq. 16, a shear limit state surface can be constructed for each shear wall cross-section. A typical shear limit state surface is shown in Fig. 6. In this figure, lines 9 and 12 are governed by Eqs. 11-14 and lines 10 and 11 are governed by Eq. 15.

From simulation results, Ellingwood[11] suggested that the variation of shear resistance can be treated as

$$V_u = B\ \bar{V}_u \qquad (17)$$

In this formulation, a random variable B is employed to describe inherent randomness and modeling uncertainty. B is a lognormal random variable with unit mean value and coefficient of variation of 0.19. $\bar{V}_u$ is the mean value determined from Eq. 16 with mean values of f_c' and f_y. The variation of shear strength expressed in the form of Eq. 17 is used for the reliability assessment of the shear walls.

6. RELIABILITY ANALYSIS METHODOLOGY

For reliability analysis of shear walls, the methodology employed in this study follows the same approach as described in Ref. 10. The limit state probability, P_f, is defined as the probability that the structural response will reach the limit state during the lifetime. In this study, the shear wall is considered to be subjected to three loads, i.e., dead load (D), live load (L) and earthquake (E). Thus, the wall is subjected to one of the following mutually exclusive load combinations in its lifetime: D, D+L, D+E and D+L+E. With the assumption that the limit state probability under D or D+L is zero, the limit state probability P_f can be expressed as

$$P_f = P_f^{(D+E)} + P_f^{(D+L+E)} \tag{18}$$

The limit state probability for a load combination q, i.e., $P_f^{(q)}$, can be computed approximately by

$$P_f^{(q)} = T \, \lambda^{(q)} \, P^{(q)} \tag{19}$$

in which T is the lifetime of the structure. $\lambda^{(q)}$ is the occurrence rate of the load combination (q) and is determined by the formula suggested by Wen.[15] The conditional limit state probability given the occurrence of the load combination (q), i.e., $P^{(q)}$, is the probability that the combined load effects exceed the structural resistance.

Analytically, the limit state surface, as established in Section 5, can be expressed as

$$R_j - \{A_j\}^T \{F\} = 0 \qquad j = 1,2,\ldots,N_s \tag{20}$$

where N_s is the number of segments of a polygon. $\{F\}$ is the vector of the internal force resultant such as the axial force (N), shear (V) and moment (M). R_j and $\{A_j\}$ are coefficient and coefficient vector, respectively. The limit state surface essentially represents the boundary of the safe region, that is, the structural resistance is larger than the load effects inside the limit state surface. The conditional limit state probability $P^{(q)}$ can be determined as the probability of the load effects outcrossing the limit state surface. Thus, using Eq. 20, $P^{(q)}$ is bounded as:

$$\max\{P_j\} < P(q) < \sum_{j=1}^{N_s} P_j \tag{21}$$

where

$$P_j = P_r[R_j - \{A_j\}^T \{F\} \leq 0] \tag{22}$$

On the basis of the symmetry of the limit state surface and the assumption that no axial force results from earthquake, $P^{(q)}$ can be computed by the following formula instead of Eq. (21),

$$P^{(q)} = 2 \max\{P_j\} \tag{23}$$

In this study, the force vector $\{F\}$ is at most the sum of three vectors; i.e.,

$$\{F\} = \{F\}_D + \{F\}_L + \{F\}_E \tag{24}$$

where $\{F\}_D$, $\{F\}_L$ and $\{F\}_E$ are the forces due to D, L and E, respectively. The vector $\{F\}_D$ is time-invariant, while $\{F\}_L = L\,\{F\}_{L=1}$, in which $\{F\}_{L=1}$ are the forces due to an unit live load. As discussed in Section 2.2, the point-in-time intensity of live load, L, follows a gamma distribution.

The force vector due to an earthquake $\{F\}_E$ can be obtained by using the modal analysis and random vibration theory.[10]

$$\{F\}_E = Z[c]\{v_0\} \tag{25}$$

$$[c] = [B][\phi^{(e)}][L_q] \tag{26}$$

in which, $Z = \sqrt{S_0}$ and $[B]$ is the element force transformation matrix, i.e., $\{F\} = [B]\{u_0\}$ and $\{u_0\}$ is the displacement vector due to $Z = 1$. $[\phi^{(e)}]$ is the mode shape matrix containing the modal displacements of the element nodes. $\{q_0\}$ is a generalized coordinate vector and is obtained from $\{u_0\} = [\phi^{(e)}]\{q_0\}$, $\{v_0\}$ is a transformed generalized coordinate vector, given by $\{q_0\} = [L_q]\{v_0\}$ such that the covariance matrix of $\{v_0\}$ becomes an identity matrix.

Using the above expressions, Eq. 24 can be rewritten as

$$\{F\} = \{F\}_D + L\,\{F\}_{L=1} + Z[c]\{v_0\} \tag{27}$$

Substituting Eq. 27 into Eq. 20, the limit state equation becomes,

$$R_j - \{A_j\}^T\{F\}_D - L\{A_j\}^T\{F\}_{L=1} - Z\{A_j\}^T[c]\{v_0\} = 0 \tag{28}$$

In this study, the variations of the structural resistance and the dead load are included in the reliability analysis by using a Latin hypercube sampling technique.[7] For a given set of samples, the dead load effect is deterministic and hence, R_j and $\{A_j\}^T$ are a constant and a constant vector, respectively. Under this condition, Eq. 28 can be re-written as

$$r_j - d_j\,L - Z\{n_j\}^T\,\{v_o\} = 0 \tag{29}$$

where

$$\{n_j\} = \{\bar{A}_j\}/|\bar{A}_j| \text{ with } \{\bar{A}_j\} = \{A_j\}^T\,[c]$$

and

$$r_j = [R_j - \{A_j\}^T \{F\}_D]/|\bar{A}_j|$$

$$d_j = [\{A_j\}^T \{F\}_{L=1}]/|\bar{A}_j|$$

The probability distribution of

$$X_{mj} = \max|\{n_j\}^T \{v_o\}| \quad ; \quad 0 \leq t \leq \mu_{dE} \tag{30}$$

is given by[10]

$$F_{X_{mj}}(x) = \exp\{- \nu_{jo} \mu_{dE} \exp(-1/2\ x^2)\} \tag{31}$$

in which

$$\nu_{jo} = \frac{1}{2\pi} \sqrt{\sum_{a=1}^{m} \sum_{b=1}^{m} n_{aj} n_{bj}\ E[\dot{v}_{oa} \dot{v}_{ob}]}$$

and n_{aj} is the a-component of $\{n_j\}$ and $E[\dot{v}_{oa}\dot{v}_{ob}]$ is the a-b component of the covariance matrix $[V_{\dot{v}_o\dot{v}_o}]$ of $\{\dot{v}_o(t)\}$.

For the case that the shear wall structure is subjected to dead load and earthquake, $P_j^{(D+E)}$ can be computed by substituting Eqs. 29 and 30 into Eq. 22, i.e.,

$$P_j^{(D+E)} = P_r\ \{r_j - Z\ X_{mj} \leq 0\} \tag{32}$$

Assuming the shear wall will not fail under dead load alone, then r_j is positive and $P_j^{(D+E)}$ can be evaluuuud as follows:

$$P_j^{(D+E)} = \int_{Z_{min}}^{Z_{max}} \left\{1 - \exp[- \nu_{jo}\mu_{dE} \exp\left\{- 1/2\left(\frac{r_j}{Z}\right)^2\right\}]\right\} f_Z(z)dz \tag{33}$$

where $f_Z(z)$ is the density function of Z as shown in Eq. 9.

For the case that the shear wall is subjected to dead load, live load and earthquake simultaneously, $P_j^{(D+L+E)}$ can be computed by following the same approach for computing $P_j^{(D+E)}$:

$$P_j^{(D+L+E)} = \int_{Z_{min}}^{Z_{max}} \int_{L_{min}}^{L_{max}} \left\{ 1 - \exp[- \nu_{jo}\mu_{dE} \exp\left\{- 1/2 \left(\frac{r_j - d_j \ell}{Z}\right)^2\right\}]\right\} f_L(\ell) f_Z(z) d\ell dz \tag{34}$$

where $f_L(\ell)$ is the density function of the point-in-time intensity of the live load, which is assumed to follow a Gamma distribution.

7. ILLUSTRATIVE APPLICATIONS

Several examples are delineated in this section to illustrate the reliability analysis methodology for shear wall structures.

Case 1. A rectangular shear wall, which is fixed at the base, is first considered. The height of the shear wall is 75 feet and the width is 100 feet. Three floors are supported on the wall at 25, 50 and 75 feet above the ground. It is assumed that the superimposed dead load on each floor is 12 K/ft and no live load acts on the wall. The safe shutdown earthquake (SSE) for design of the wall is taken to be 0.5 g. The shear wall is designed according to the current ACI-349 code for both shear and flexure requirements. The specified concrete compressive strength is 4000 psi and yield strength of the reinforcing bar is 60,000 psi. Assuming that the wall thickness is 1 foot, then the required horizontal and vertical reinforcement ratios are determined to be 0.003 and 0.00765, respectively.

The probabilistic characteristics of the load and material strengths are described in Sections 3 and 4. They are summarized in Tables 1 and 2, respectively. The limit states for flexure and shear are defined in Section 5. In this study, the variations of structural resistance and dead load are included in the analysis by using a Latin hypercube sampling technique. The sample size is chosen to be ten. Table 3 shows ten values of f'_c, f_y, D and B, which are chosen according to their distributions and each value has equal probability. Table 4 gives the ten sets of the Latin hypercube samples. Each set of the sample is used to evaluate the conditional limit state probability $P^{(D+E)}$, of the shear wall. For flexure and shear limit states, the average values of these ten conditional limit state probabilities are 4.204 x 10^{-4} and 2.144 x 10^{-4}, respectively. The limit state probabilities for flexure and shear computed by using Eq. 19 are respectively 3.370 x 10^{-3} per 40 years (or 8.425 x 10^{-5} per year) and 1.718 x 10^{-3} per 40 years (or 4.295 x 10^{-5} per year).

Both shear and flexure limit states are reached at the base of the shear wall. For the flexure limit state, the limit state probability P_f comes from lines 1 and 8 of Fig. 5. Furthermore, for the shear limit state, P_f is from those associated with lines 9 and 12 of Fig. 6. These results are expected since the axial force from the dead load is relatively small and consequently, the above mentioned lines will be critical.

Case 2. In this case, it is assumed that live load acts on the shear wall in addition to the loads in Case 1. The probabilistic characteristics of live load are also shown in Table 1 with the design value taken to be half of the superimposed dead load (i.e., 6 K/ft at each floor). The reliability analysis results are shown in Table 5. For a lifetime of 40 years, the limit state probabilities for flexure and shear are 3.584×10^{-3} and 1.890×10^{-3}, respectively. Comparison of the reliability result of Case 2 to that of Case 1 shows that the live load has no significant effect on the reliability of the shear wall used in this study. The reliability analyses with different statistics for the live load are also performed. It is concluded that the limit state probability is not sensitive to the statistics of the live load.

Case 3. The design condition is the same as that for Case 1, except that a flanged shear wall is used instead of a rectangular one. The flange is assumed to have the same thickness as the web thickness (i.e., 1 foot). The flange width is taken to be 8 times the web thickness. The design is carried out according to the ACI-349 code. Furthermore, it is assumed that the flexural reinforcement will be concentrated in the flanges and the shear reinforcement is distributed in the web. The required vertical flexural reinforcement ratio is determined to be 0.01919. The required shear reinforcement ratios in both horizontal and vertical direction are 0.00275. The probabilistic characteristics for load and material strength are the same as described in Case 1. For a lifetime of 40 years, the limit state probabilities for flexure and shear are 4.960×10^{-3} and 2.617×10^{-3}, respectively.

A summary of the above three cases is shown in Table 6. It is noticed that the above shear walls are designed according to the flexure and shear strength requirements specified in the ACI-349 code. The actual shear walls used in the nuclear power plants need to consider other requirements such as the protection from radioactivity and tornado-borne missiles. Thus, the actual shear walls in the nuclear power plants are more massive and thus, their limit state probabilities are probably much lower than those in this report.

8. FRAGILITY CURVES

The fragility, $P(A_1)$, is defined as the conditional limit state probability, given a peak ground acceleration A_1. Hence, referring to Eqs. 23 and 33, the fragility is determined in approximation as:

$$P(A_1) = 1 - \exp[- 2\nu_{jo}\mu_{dE} \exp(- 1/2 (\frac{\alpha_g \gamma_j}{A_1})^2)] \qquad (35)$$

where the crossing rate ν_{jo} has been doubled to account for the symmetry of the limit state and the assumption that no axial force results from the earthquake load.

Fragility curves have been generated for the three-story rectangular shear walls with different thicknesses. The wall thickness of 12, 15, 18, 24, 30 and 36 inches are used. For each thickness, the height of the wall is 75 feet and the length is 125 feet. Both horizontal and vertical reinforcement ratios are 0.0025 for each case.

Dead load, which consists of the weight of the wall and the additional superimposed load of 16 k/ft, is assumed to be normally distributed. The mean value is equal to the design value and the coefficient of variation is taken to be 0.07. An earthquake is represented by a stationary random process with a Kanai-Tajimi Spectrum. The parameters for this spectrum, i.e., ω_g and ζ_g are taken to be 5 rad/sec and 0.6, respectively. The mean duration of an earthquake is assumed to be 20 seconds. The material properties are the same as described in Section 4.

For the shear limit state, the fragility of each shear wall is evaluated by using Eq. 35. The fragility curves are shown in Fig. 7 and the range and the median value are tabulated in Table 7. As expected, the fragility level is increased as the wall thickness is increased.

9. CONCLUDING REMARKS

This report describes a reliability analysis method for shear wall structures. In the method, the shear wall, which has low height-to-length ratio, is modeled by beam elements. In addition to the dead and live loads, it is assumed that shear walls will also be subjected to in-plane earthquake. The probabilistic models for these loads are described in the report. Both shear and flexure limit states are considered. The flexure limit state is defined according to the ACI ultimate strength formula, while the shear limit state is established from test data. Based on the above information, the limit state probability can be evaluated for the lifetime of the shear walls. This reliability analysis method can be used to evaluate the reliability level of existing shear walls as illustrated in the report. It also can be used to derive load factors for design of shear walls.[16]

The reliability analysis method is analytically formulated and hence, the results are affected by the data used to determine the parameters of the analytical models. An extensive effort was made to collect the available data and to formulate the analytical models. However, it is desirable to have an independent assessment of the available data and to update these data so that the quality of the reliability analysis results can be ensured. In addition, a sensitivity study should be carried out to examine the effect of the variability of the parameters.

The buildings in a nuclear power plant such as the auxiliary building usually consist of many shear walls. In this report, engineering judgement is utilized to separate these shear walls into individual rectangular or flanged walls. Then, the reliability analysis is performed for each wall. However, it is possible to extend the reliability analysis methodology so that the whole building can be taken into consideration. The reliability analysis of a whole building may be more expensive, but the three dimensional effect and stress redistribution in the non-linear range could be included.

In this study, three loads, i.e., dead load, live load and in-plane earthquake are considered in the reliability analysis. However, the shear walls in the nuclear power plants may be subjected to other loads such as tornado-borne missiles. Further work is needed to improve the reliability analysis methodology in order to include other load effects.

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Table 1. Probabilistic Models For Loads.

Load	Model
Dead Load (D)	Time Invariant Normal Distribution With $\overline{D} = 1.0\ D_n$ and CoV(D) = 0.07
Live Load (L)	Poisson Renewal Process With Rectangular Pluse Mean Duration 0.25 Year (3 Months) Occurrence Rate 0.5 Per Year Point in Time Intensity; Gamma Distribution With $\overline{L} = 0.36\ L_n$ and CoV(L) = 0.54
Earthquake (E)	Seismic Hazard Follows a Type II Distribution $1 - G_A(a) = \exp[-(a/\mu)^{-\alpha}]$ $\alpha = 2.7,\ \mu = 0.02757$ $a_{SSE} = 0.5$ g With $G_A(a_{sse}) = 4.0 \times 10^{-4}$ Per Year An earthquake is represented by the stationary random process with a Kanai-Tajimi spectrum $S_{gg}(\omega) = S_o \dfrac{1 + 4\zeta_g^2(\omega/\omega_g)^2}{[1 - (\omega/\omega_g)^2]^2 + 4\zeta_g^2(\omega/\omega_g)^2}$ where $\omega_g = 8\pi$ rad/sec, $\zeta_g = 0.6$ $a_o = 0.05$ g, $a_{max} = 3a_{SSE}$ Occurrence rate, $\lambda_E = 0.2004$ per year Mean duration, $\mu_{dE} = 20$ seconds

Table 2. Probabilistic Model For Material Strength.

Material Strength	Model
f'_c	Normal Distribution $\overline{f'_c} = 1219 + 1.02\ f'_{cn}$ $f'_{cn} = 4000$ psi, $\overline{f'_c} = 5299$ psi $CoV(f'_c) = 0.14$
f_y	Lognormal Distribution $\overline{f_y} = 71000$ psi ($f_{yn} = 60{,}000$ psi) $CoV(f_y) = 0.11$

Table 3. Distribution of f'_c, f_y, D and B.

Probability	f'_c	f_y	D	B
.050	.407875E+04	.569256E+05	.418096E+07	.720732E+00
.150	.453011E+04	.629916E+05	.438220E+07	.808228E+00
.250	.479862E+04	.655423E+05	.450191E+07	.865238E+00
.350	.501315E+04	.676541E+05	.459756E+07	.913662E+00
.450	.520578E+04	.696084E+05	.468344E+07	.959449E+00
.550	.539222E+04	.715536E+05	.476656E+07	.100595E+01
.650	.558485E+04	.736205E+05	.485244E+07	.105636E+01
.750	.579938E+04	.759927E+05	.494809E+07	.111548E+01
.850	.606789E+04	.790698E+05	.506780E+07	.119417E+01
.950	.651925E+04	.845258E+05	.526904E+07	.133913E+01

Table 4. Latin Hypercube Samples.

Sample Set	f'_c	f_y	D	B
1	.558485E+04	.790698E+05	.418096E+07	.105636E+00
2	.579938E+04	.845258E+05	.476656E+07	.100595E+00
3	.407875E+04	.715536E+05	.506780E+07	.720732E+00
4	.606789E+04	.736205E+05	.459756E+07	.133913E+00
5	.453011E+04	.696084E+05	.450191E+07	.913662E+00
6	.520578E+04	.629916E+05	.526904E+07	.808228E+01
7	.539222E+04	.676541E+05	.438220E+07	.111548E+01
8	.501315E+04	.759927E+05	.494809E+07	.865238E+01
9	.479862E+04	.589256E+05	.485244E+07	.119417E+01
10	.651925E+04	.655423E+05	.468344E+07	.959449E+01

Table 5. Limit State Probabilities With Live Load.

Load Combination	Flexure		Shear	
	p(D+E)	P_f	p(D+E)	P_f
D+E	4.204 -4	2.949 -3	2.144 -4	1.504 -3
D+L+E	6.339 -4	6.351 -4	3.854 -4	3.861 -4
Overall		3.584 -3		1.890 -3

Table 6. Reliability of Shear Walls.

Case			1	2	3
Shape			Rectangular	Rectangular	Flanged
Load Combination			D+E	D+L+E	D+E
Reinforcement Ratio	Flange	ρ_n	---	---	0.01919
	Web	ρ_n	0.00765	0.00765	0.00175
		ρ_h	0.0030	0.0030	0.00175
P_f		Flexure	3.370 -3	3.584 -3	4.96 -3
		Shear	1.718 -3	1.890 -3	2.617 -3

Table 7. Fragility of Shear Walls.

Thickness (in)	Range (g)	Median Value (g)
12	0.40 - 1.70	0.92
15	0.50 - 2.00	1.12
18	0.57 - 2.35	1.30
24	0.67 - 2.90	1.61
30	0.84 - 3.4	1.90
36	0.93 - 3.80	2.15

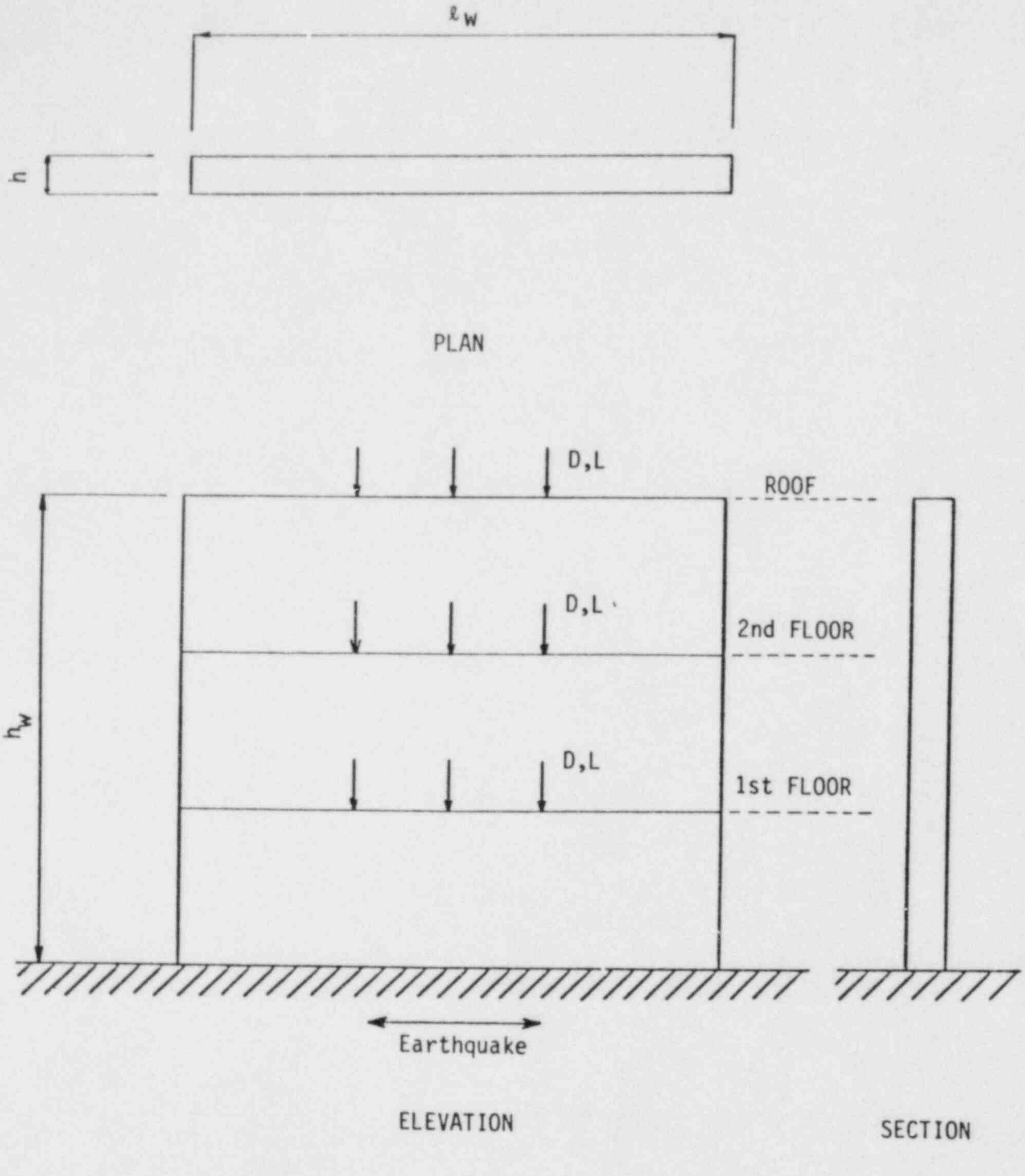


Fig. 1. Representative Shear Wall Structure.

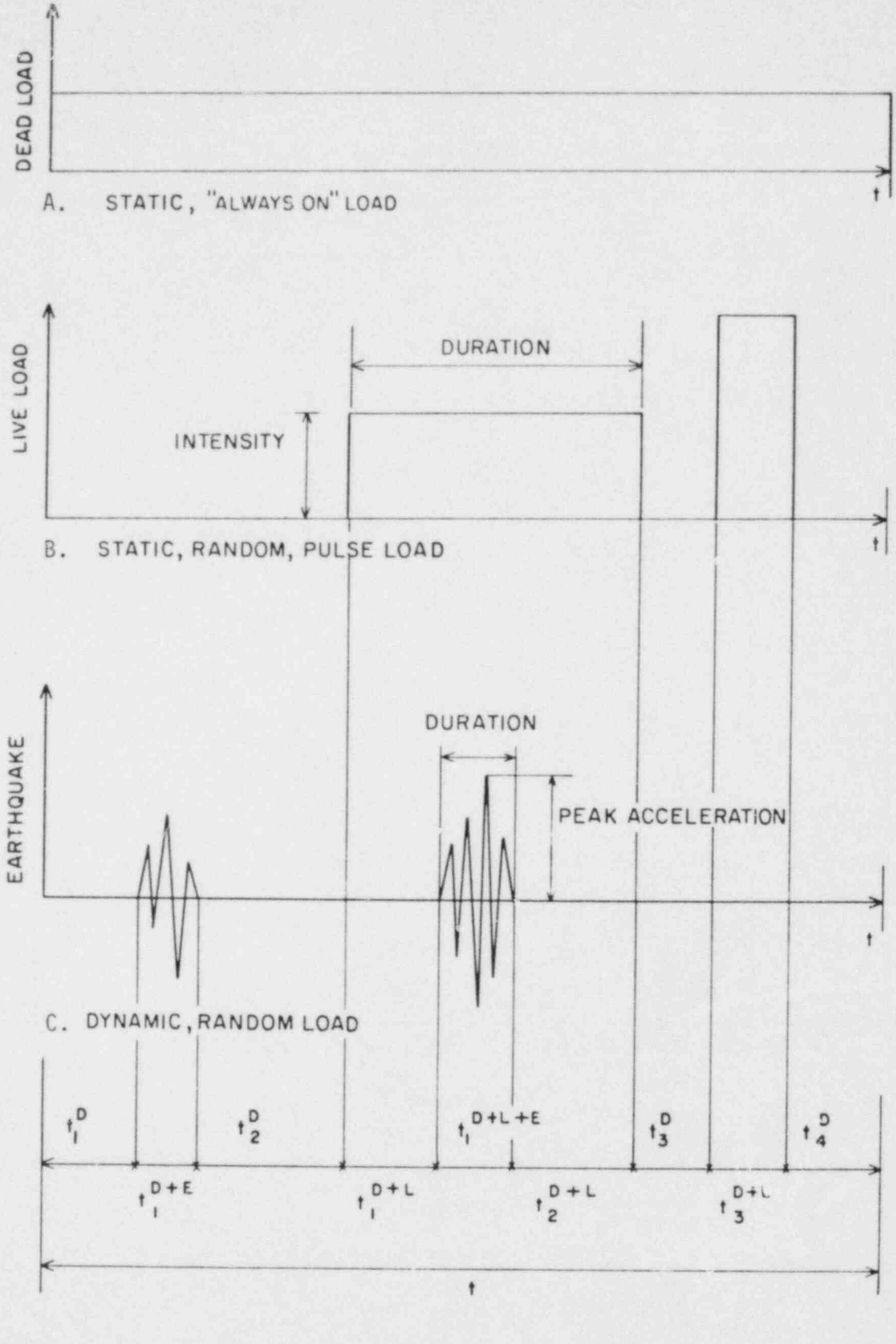


Fig. 2. Probabilistic Representation of Loads.

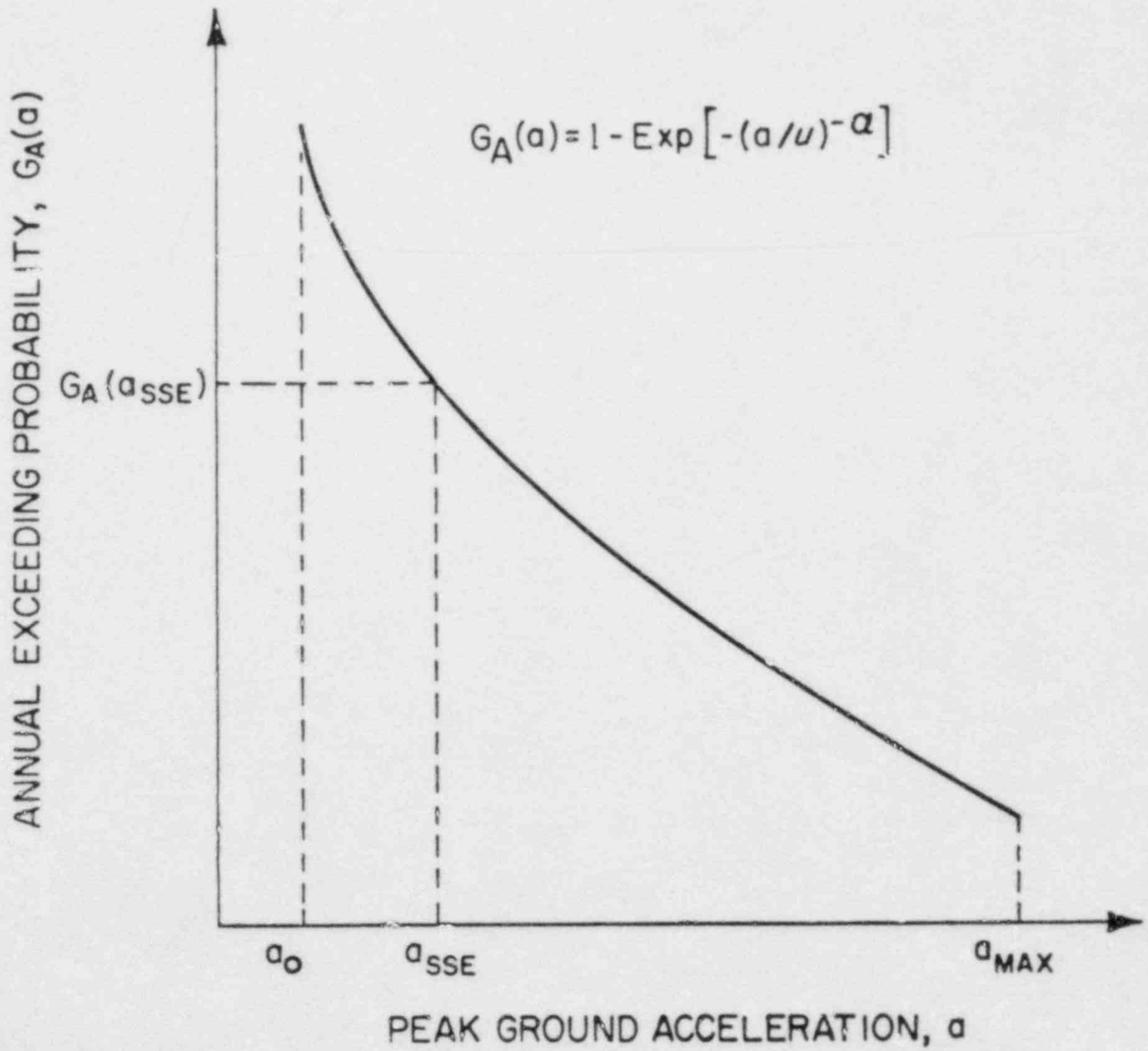
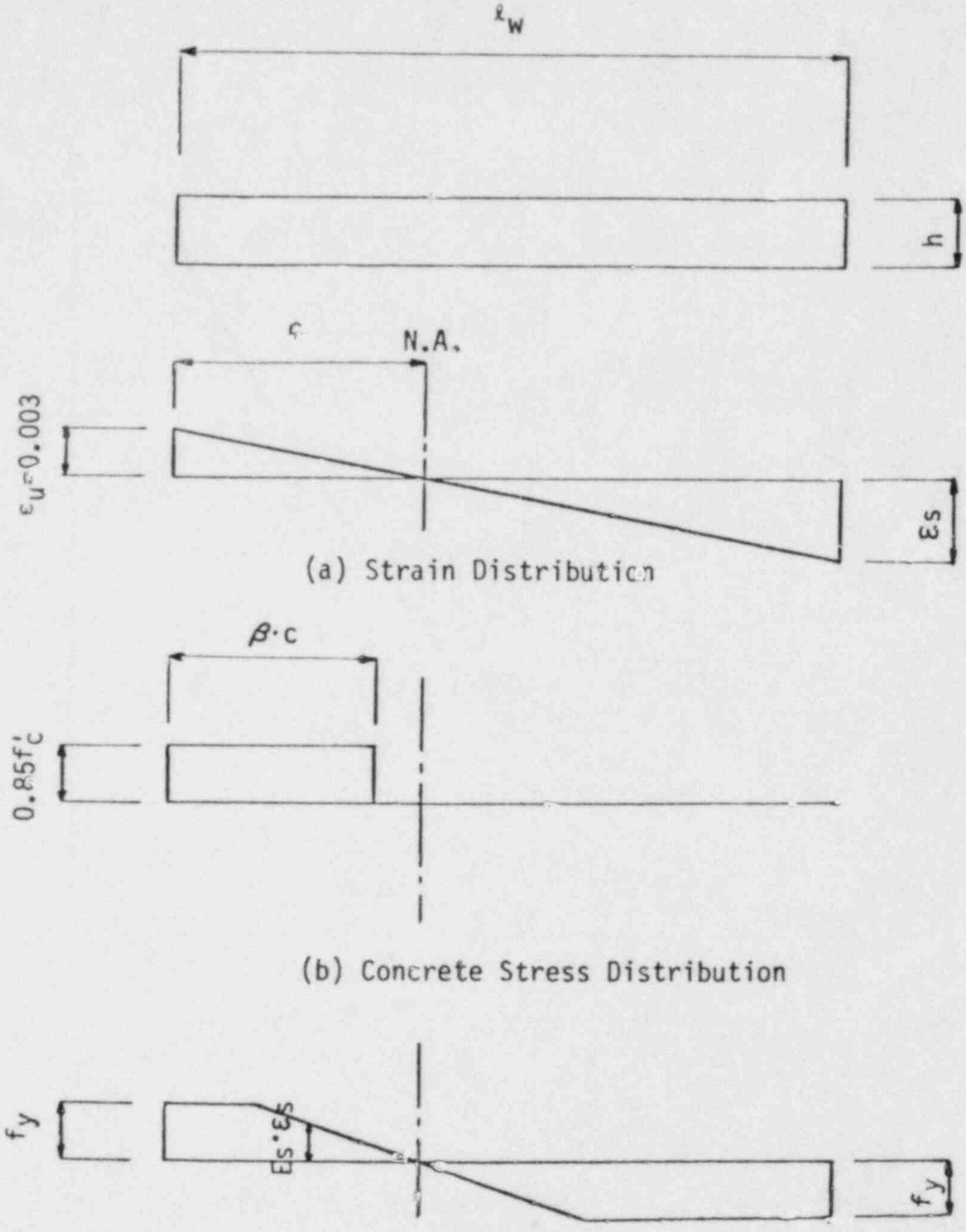


Fig. 3. Seismic Hazard Curve



(a) Strain Distribution

(b) Concrete Stress Distribution

(c) Rebar Stress Distribution

Fig. 4. Stress and Strain Distributions.

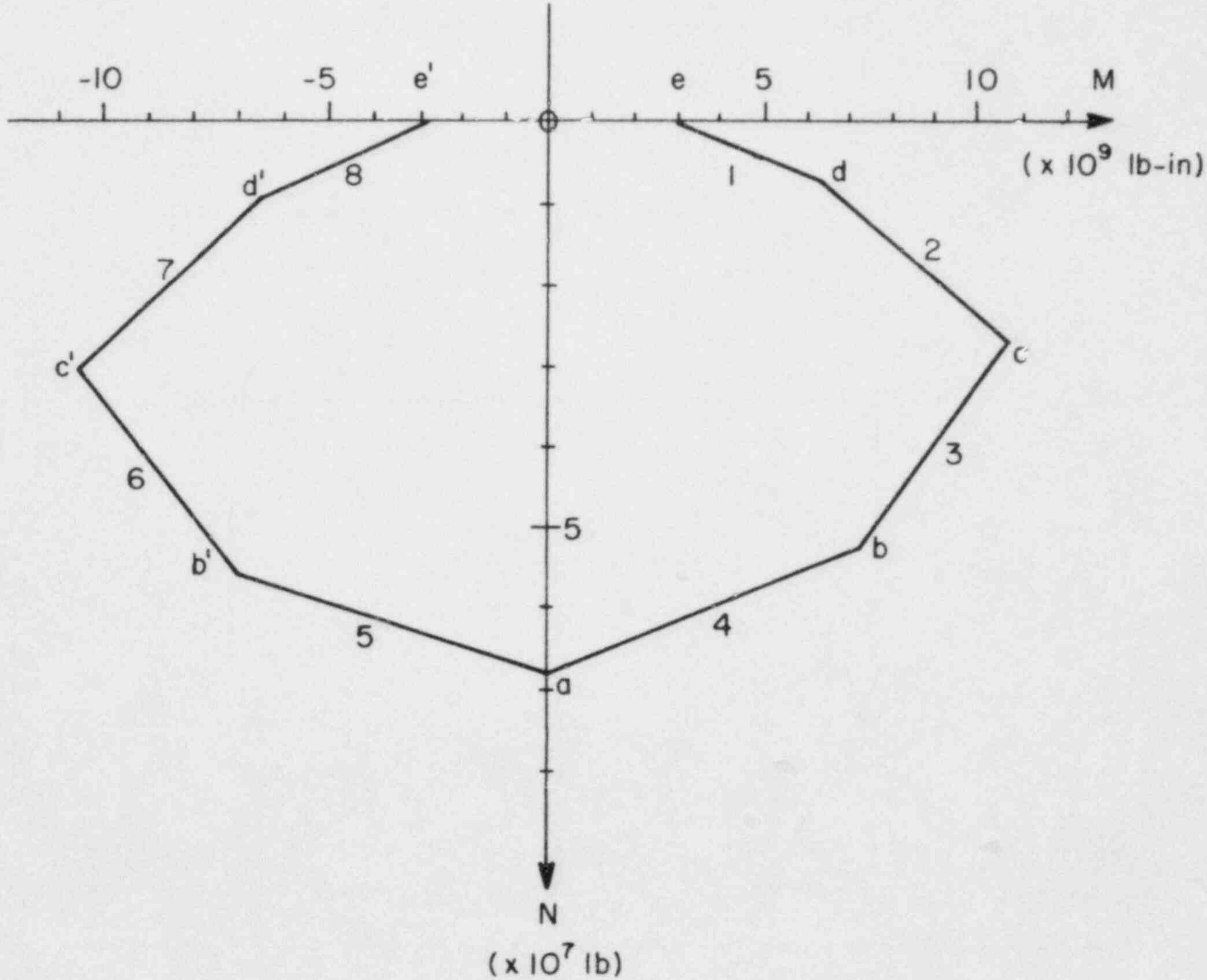


Fig. 5. Flexure Limit State Surface.

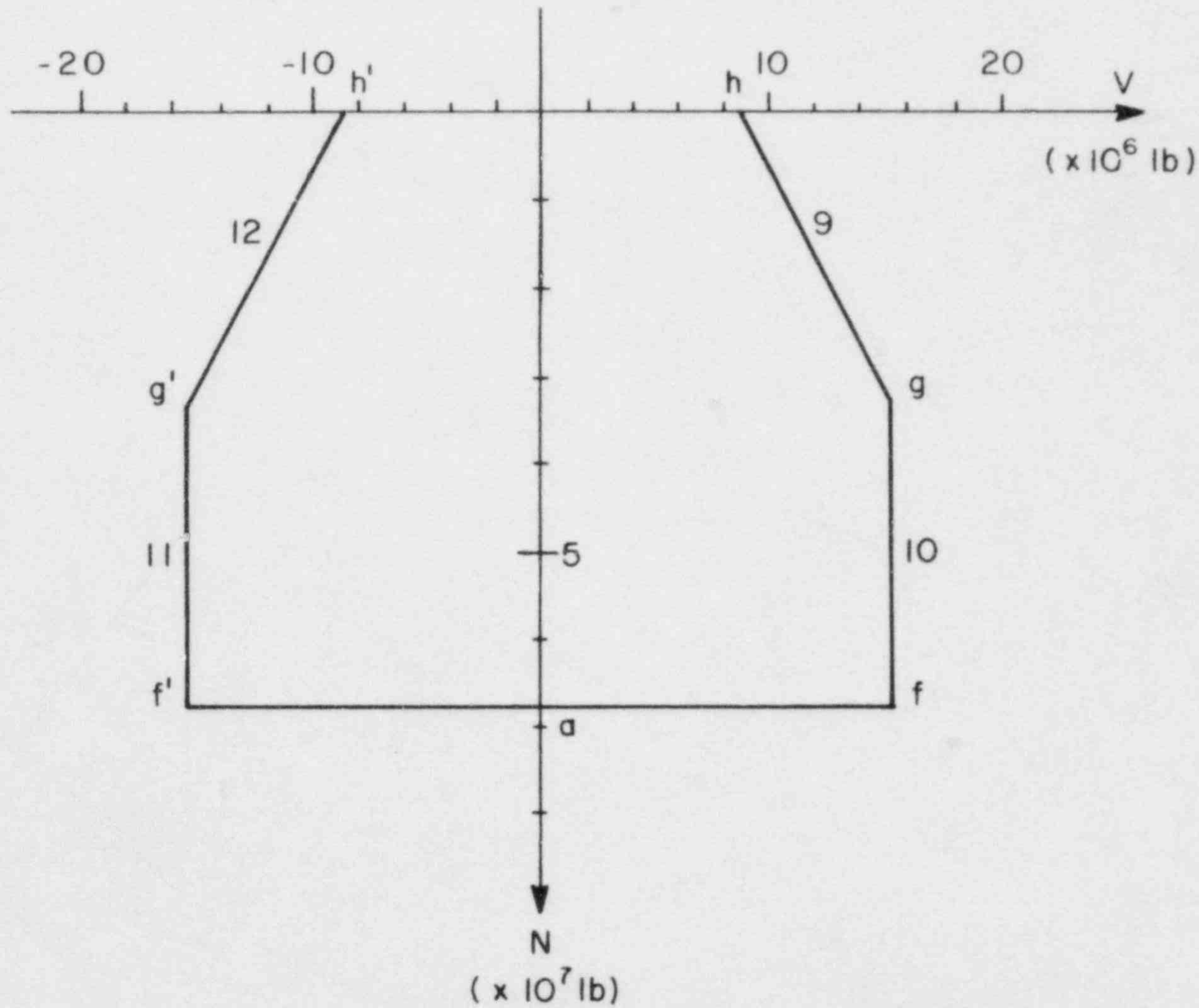


Fig. 6. Shear Limit State Surface.

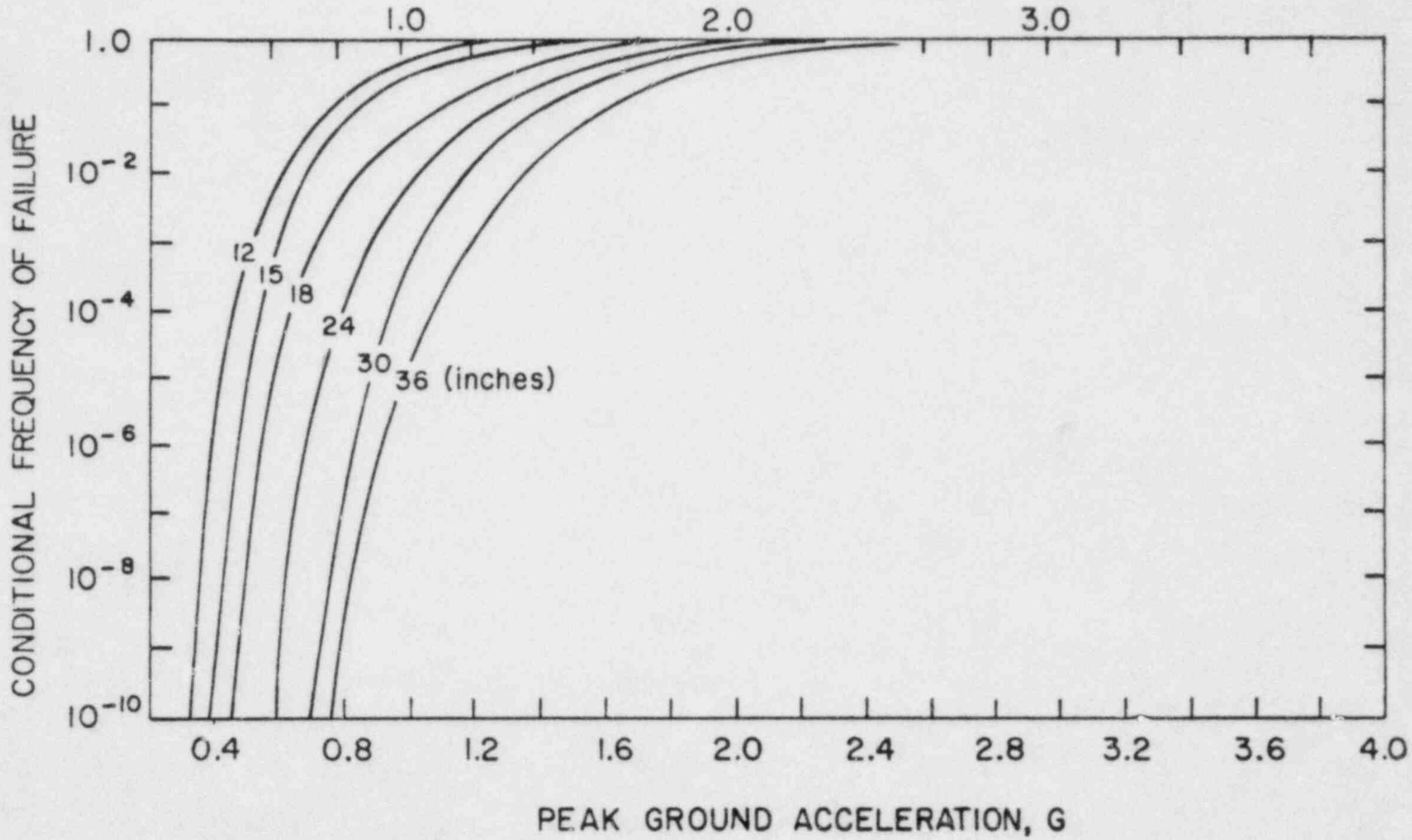


Fig. 7. Fragility Curves For Shear Walls.

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13. ABSTRACT (200 words or less)

This report describes a method for the assessment of the reliability of low-rise shear wall structures, which are often used in nuclear power plants. The shear walls are modeled by stick models with beam elements, and are subjected to dead load, live load and earthquake during their lifetimes. The earthquake load is assumed to be a segment of a stationary Gaussian process with a zero-mean and a Kanai-Tajimi power spectral density function. The seismic hazard at a site, represnted by a hazard curve, is also included in the reliability analysis.

Both shear and flexure limit states are analytically defined. The flexure limit state is defined according to the ACI strength design formula, while the shear limit state is established from test data. The reliability analysis methodology is described in detail. Illustrative examples are given to demonstrate the method and the applications. This reliability analysis method can also be used to generate the fragility curve of the shear wall structure.

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