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Structural Load Combinations

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Abstract

This paper presents the latest results of the program entitled, "Probability Based Load Combinations For Design of Category I Structures". In FY 85, a probability-based reliability analysis method has been developed to evaluate safety of shear wall structures. The shear walls are analyzed using stick models with beam elements and may be subjected to dead load, live load and in-plane earthquake. Both shear and flexure limit states are defined analytically. The limit state probabilities can be evaluated on the basis of these limit states.

Utilizing the reliability analysis method mentioned above, load combinations for the design of shear wall structures have been established. The proposed design criteria are in the load and resistance factor design (LRFD) format. In this study, the resistance factors for shear and flexure and load factors for dead and live loads are preassigned, while the load factor for SSE is determined for a specified target limit state probability of 1.0×10^{-6} or 1.0×10^{-5} during a lifetime of 40 years.

1. INTRODUCTION

The program entitled, "Probability Based Load Combinations for Design of Category I Structures", is currently being worked on for the Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission. The objective of this program is to develop a probabilistic approach for evaluating safety of reactor containments and other seismic category I structures subjected to multiple static and dynamic loadings. Furthermore, based on this probabilistic approach, load combination criteria for the design of Category I structures will also be established.

This paper presents the latest results of the program. Specifically, the reliability analysis method for shear wall structures, and the probability-based load combinations for the design of shear walls recently have been de-

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veloped. In the following sections, the shear wall structures and the limit states used in this study are described first. Then, the probabilistic models of loads and material strengths are presented. Next, the reliability analysis method for shear walls is discussed and an example is given to demonstrate the method. Finally, the probability-based design criteria are presented.

2. SHEAR WALL STRUCTURES AND LIMIT STATES

Shear walls are used in many Category I structures in nuclear power plants as the primary system for resisting lateral loads such as earthquakes. These shear walls usually have low height-to-length ratios and exist either as part of a rectangular box or as individual walls. In this study, a low-rise three-story rectangular shear wall, as shown in Fig. 1, is chosen as a representative shear wall structure. The shear wall is analyzed using a stick model with beam elements and it may be subjected to dead load, live load and in-plane horizontal earthquake during its lifetime.

The limit states of a low-rise shear wall include flexure, shear, sliding and buckling. A typical shear wall in a nuclear plant structure is massive and low. Thus, buckling failure would be very rare. Resistance to sliding 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, sliding failures would also be rare. In this study, therefore, sliding and buckling failures of shear walls are not considered. The shear and flexure limit states are defined below.

2.1 Flexure Limit State

The flexure limit state for shear walls is defined analytically according to ultimate strength analysis of reinforced concrete. It is described 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 yielding of rebars is permitted. Based on the above definition of the limit state, a limit state surface can be constructed for a cross-section with given geometry and rebar arrangement in terms of the axial force and bending moment on a cross-section. A typical flexure limit state surface, which is approximated by a polygon, is shown in Fig. 2. In this figure, point "a" is determined from a 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'" correspond to zero axial force. Lines abc and ab'c' in Fig. 2 represent compression failure and lines cde and c'd'e' represent tension failure.

The flexure limit state surface represents the flexural capacity of a shear wall. Since the flexural capacity is calculated using the ultimate strength analysis of reinforced concrete, the variability of the capacity is caused primarily by the variations of concrete compressive strength and rebar yield strength.

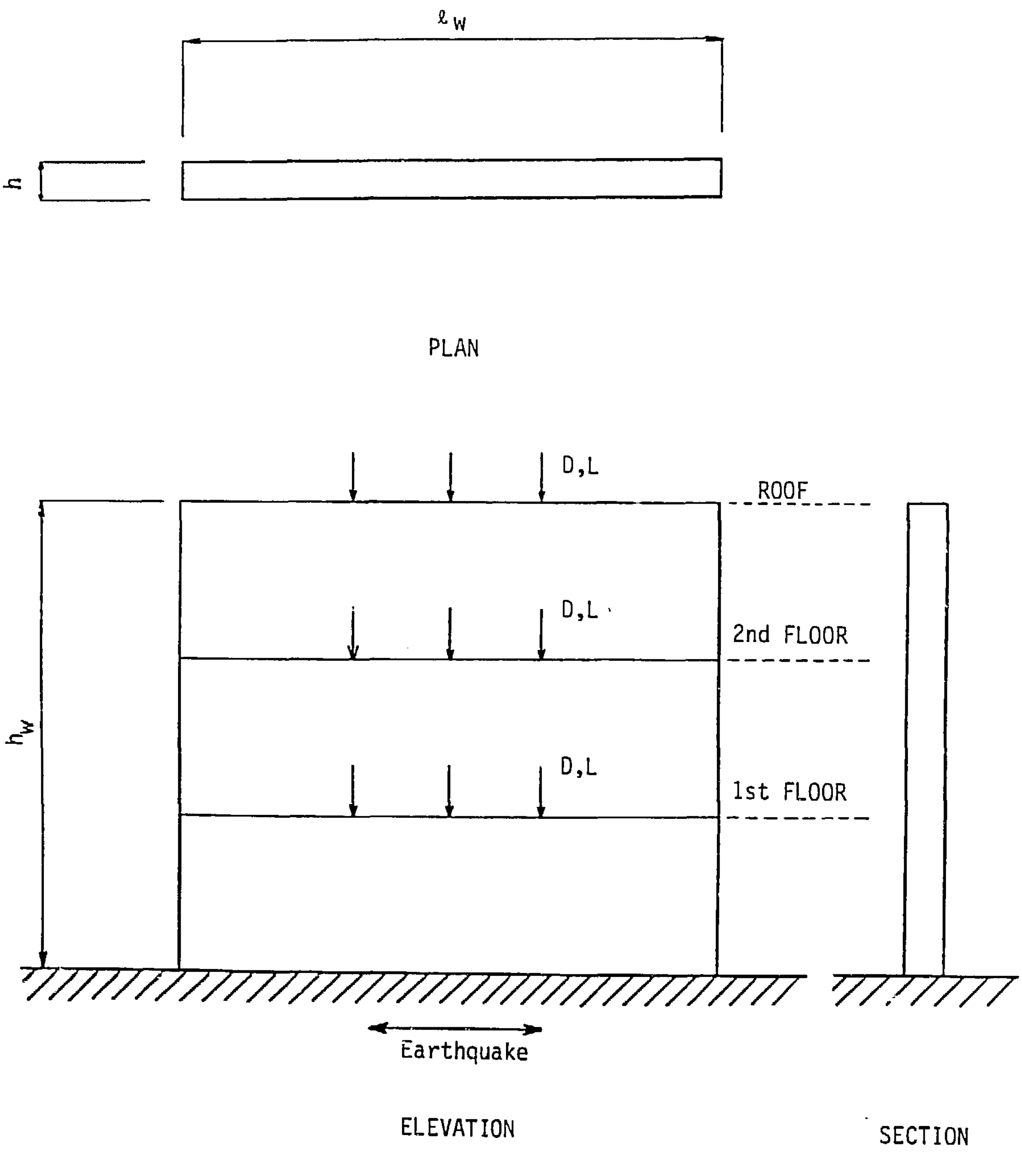

Fig. 1. Representative Shear Wall Structure.

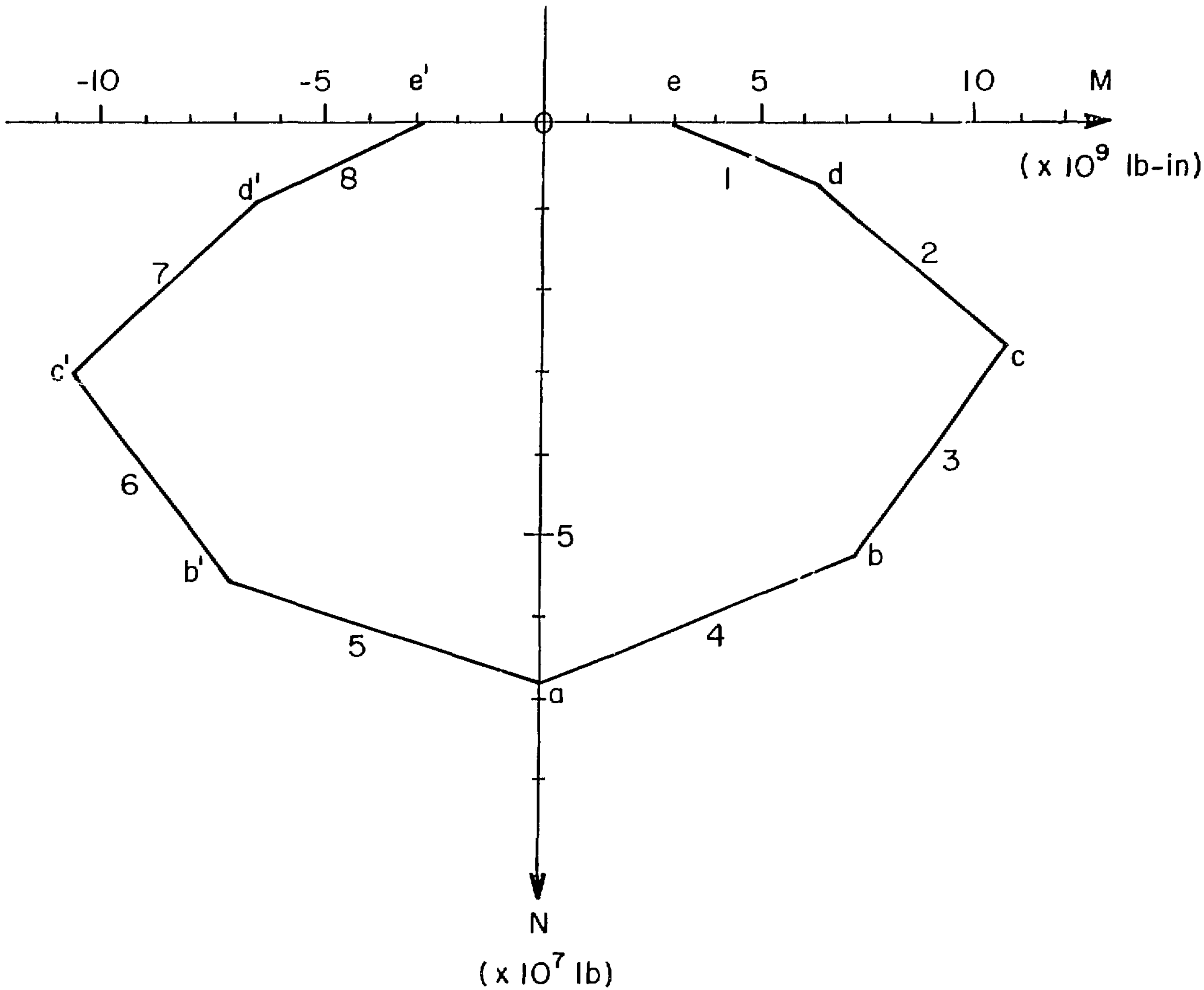

Fig. 2. Flexure Limit State Surface.

2.2 Shear Limit State

The shear limit state is reached when diagonal cracks form in two directions; following the formation of the diagonal cracks, either concrete crushes or rebars yield and fracture. The ultimate shear strength of a shear wall, v_u, expressed in units of force/area, is

$$v_u = v_c + v_s \tag{1}$$

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

Barda, et al.[2], 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 by,

$$v_c = 8.3 \sqrt{f'_c} - 3.4 \sqrt{f'_c}\left(\frac{h_w}{\ell_w} - \frac{1}{2}\right) + \frac{N_u}{4\ell_w h} \quad ; \frac{1}{4} \leqslant \frac{h_w}{\ell_w} \leqslant 1.0 \tag{2}$$

in which N_u is axial force taken as positive in compression and h is the wall thickness. 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 less. However, it was less effective as height-to-length ratio approached 1.

Since the effectiveness of the horizontal and vertical reinforcement varies for different height-to-length ratios, the following equation for v_s is recommended[22],

$$v_s = (a\ \rho_h + b\ \rho_n) f_y \tag{3}$$

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

$$b = \begin{cases} 1 & ; \quad \frac{h_w}{\ell_w} < 1/2 \\ 2 - 2\frac{h_w}{\ell_w} & ; \quad 1/2 \leqslant \frac{h_w}{\ell_w} \leqslant 1 \\ 0 & ; \quad \frac{h_w}{\ell_w} > 1 \end{cases} \tag{4}$$

and

$$a = 1 - b$$

Both horizontal and vertical rebars are partially effective outside the given limits, but Eq. 4 is not sensitive to these limits as long as horizontal and vertical rebars both are used.

Gergely[12] has suggested that a low-rise shear wall would fail by diagonal crushing of the concrete if the shear stress is larger than the following unit ultimate shear strength:

$$v_u = 0.25 \, f_c' \tag{5}$$

However, Eq. 5 does not account for the effects of wall slenderness and reinforcement. In this study, the unit ultimate shear strength is taken as the smaller of those determined from Eqs. 1-4 or Eq. 5. The total ultimate shear strength V_u is computed as

$$V_u = v_u \, h \, d \tag{6}$$

where h is the wall thickness and d is the effective depth, which is taken to be 0.8 ℓ_w for rectangular walls. From Eq. 6, a shear limit state surface can be constructed for any shear wall cross-section. A typical shear limit state surface is shown in Fig. 3. In this figure, lines 9 and 12 are governed by Eqs. 1-4 and lines 10 and 11 are governed by Eq. 5.

From simulation results, Ellingwood[10] suggested that the actual shear resistance can be treated as

$$V_u = B \, \overline{V}_u \tag{7}$$

where $\overline{V}_u$ is the mean value determined from Eq. 6 using mean values of f_c' and f_y. B is a lognormal random variable with unit mean value and coefficient of variation of 0.19. In this study, the shear strength obtained from Eq. 7 is used for the reliability assessment of the shear wall.

3. PROBABILISTIC CHARACTERISTICS OF LOADS AND MATERIAL STRENGTHS

Since the loads involve random and other uncertainties, an appropriate probabilistic model for each load must be established in order to perform the reliability analysis.

3.1 Dead Load

Dead load is a static load and acts permanently on a structure. It is derived mainly from the weights of the structural system, the permanent equipment and attachments such as pipings, HVAC ducts and cable trays. Except for the attachments, the variations associated with the weights of structure or equipment are small.[11,13] Dead load is assumed to be normally distributed.[11] The mean value is equal to the design value and the coefficient of variation (CoV) is estimated to be 0.07.[11] Permanent equipment loads are treated separately in the proposed probability-based load combinations.[11,14]

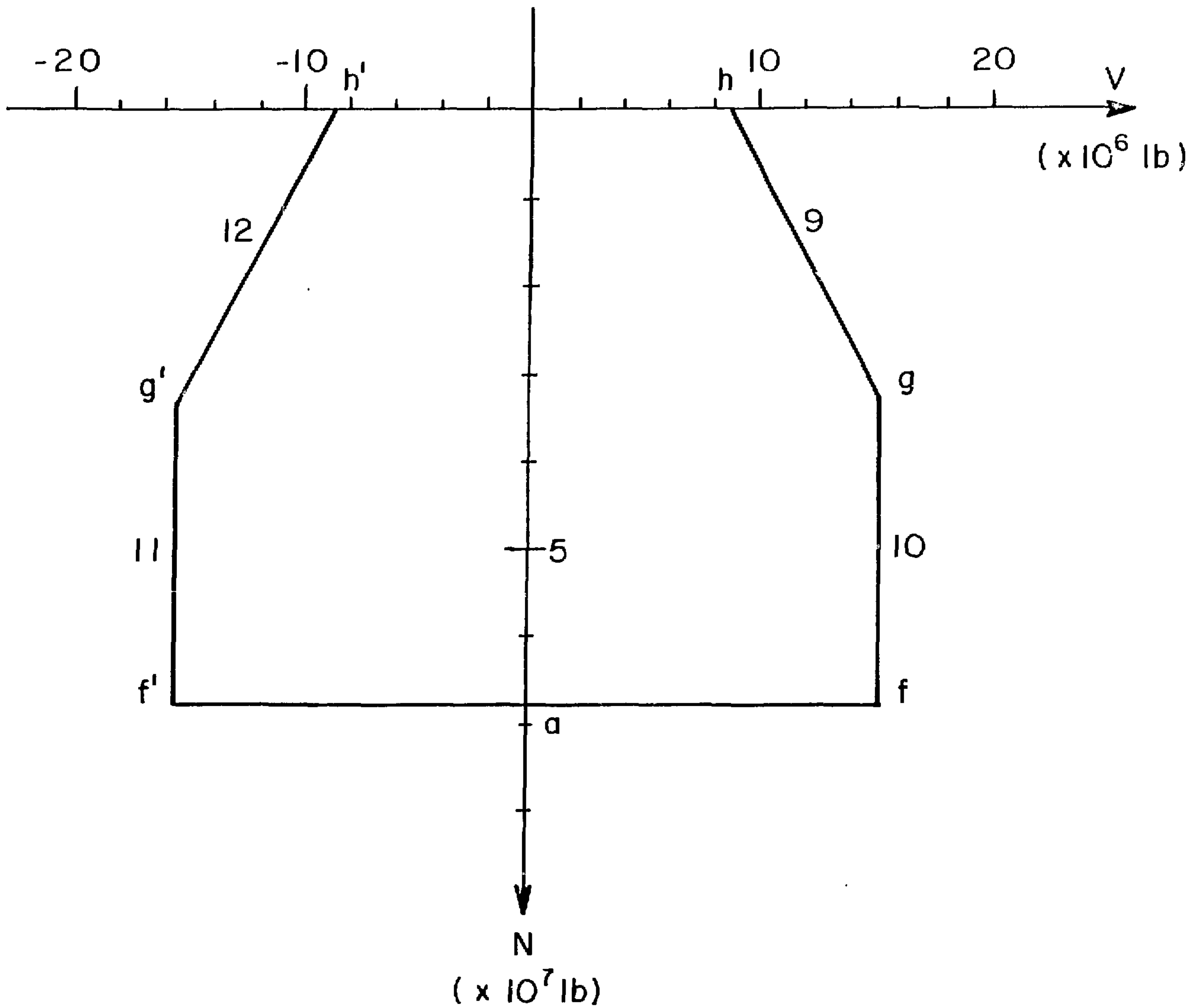

Fig. 3. Shear Limit State Surface.

3.2 Live Load

Live load in nuclear power plants denotes any temporary load resulting from human occupancy, movable equipment and other operational or maintenance conditions. Significant live load might arise from temporary equipment or materials during maintenance or repair within the plant. Thus, live load is modeled as a Poisson renewal rectangular pulse process which is defined by the occurrence rate, mean duration, and the probability distribution of the point-in-time intensity.

Measurements of live loads in nuclear power plants were unavailable. 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.[13] The live load data from the consensus estimation survey were analyzed in Appendix A of Ref. 11. 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 value and its coefficient of variation is 0.37. With a mean duration of three months, several statistics for the point-in-time live load corresponding to different occurrence rates can be obtained.[11] In this study, the occurrence rate is taken to be 0.5 per year; thus, the mean value of the point-in-time live load intensity is 0.36 times the nominal design value and the coefficient of variation is 0.54. The point-in-time live load is assumed to have a gamma distribution.

3.3 Earthquake

The seismic hazard at the site of a nuclear power plant is described by a seismic hazard curve. A seismic hazard curve, is a plot of annual exceedance probability $G_A(a)$ vs. the peak ground acceleration. In this study, the probability distribution $F_A(a)$ of the annual peak ground acceleration A is assumed to be the Type II extreme value distribution[9],

$$1 - G_A(a) = F_A(a) = \exp\left[-(a/\mu)^{-\alpha}\right] \qquad (8)$$

where α and μ are two parameters to be determined. The value of α for the U.S. is estimated to be 2.7.[14] The parameter μ is computed based on this α-value and the assumption that the annual probability of exceeding the safe shutdown earthquake at the site is 4×10^{-4} per year.[19] The hazard curve used in this study compares well with six out of the eight curves with 50 percent confidence for eight specific plant sites in the Eastern United States.[3,14]

In addition to the mean duration of an earthquake, 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 the 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 effects of different values of a_{max} on the load factors are reported in Ref. 14. In this study, a_{max} is chosen to be $2a_{SSE}$.

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],

$$S_{gg}(\omega) = S_0 \frac{1 + 4\zeta_g^2(\omega/\omega_g)^2}{[1 - (\omega/\omega_g)^2]^2 + 4\zeta_g^2(\omega/\omega_g)^2} \tag{9}$$

where the parameter S_0 is a random variable which represents the intensity of an earthquake. The distribution of S_0 can be determined as shown in Ref. 20. Parameters ω_g and ζ_g are the dominant ground frequency and the critical damping, respectively, which depend on the site soil conditions. For rock and deep cohesionless soil conditions, ω_g is taken to be 8π rad/sec and 5π rad/sec, respectively. ζ_g is taken to be 0.6 for both soil conditions.[9]

3.4 Material Properties

In order to perform a reliability analysis of a shear wall structure, it is necessary to determine the actual material properties. In this study, the material strengths are random, while other properties are assumed to be deterministic.

A. Concrete

The density of concrete is taken to be 150 lb/ft^3. Young's modulus is computed according to ACI code[5] and Poisson's ratio for concrete is 0.2. The concrete compressive strength, f'_c, is assumed to be normally distributed with CoV of 0.14 and a mean value at 1 year, $\bar{f}'_c$[10],

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

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

B. Reinforcing Bars

The yield strength f_y of ASTM A 615 Grade 60 deformed bars is assumed to have a lognormal distribution with a mean value of 71.0 ksi and CoV of 0.11.[10,17] Young's modulus and Poisson's ratio are taken to be 29.0 x 10^6 psi and 0.3, respectively.

4. RELIABILITY ANALYSIS METHODOLOGY

The reliability analysis methodology for shear walls is presented in Ref. 21. It follows the same approach as described in Ref. 20. The limit state probability, $P_{f,s}$ is defined as the probability that the structural response will reach the limit state "s" 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 at least 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 and D+L is zero, the limit state probability $P_{f,s}$ can be expressed as

$$P_{f,s} = P_{f,s}^{(D+E)} + P_{f,s}^{(D+L+E)} \tag{11}$$

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

$$P_{f,s}^{(q)} = T\ \lambda(q)\ P_{c,s}^{(q)} \tag{12}$$

in which T is the lifetime of the structure, taken as 40 years. $\lambda(q)$ is the occurrence rate of the load combination (q) and is determined by formulas in Ref. 20. The conditional limit state probability given the occurrence of the load combination (q), i.e., $P_{c,s}^{(q)}$, is the probability that the combined load effects exceed the structural resistance. The technique to compute $P_{c,s}^{(q)}$ is shown in Ref. 21.

The fragility, P_s, is defined as the conditional limit state probability with respect to a limit state "s", given a peak ground acceleration. The evaluation of the fragility is also shown in Ref. 21.

4.1 Illustrative Application

A rectangular shear wall, as shown in Fig. 1, is subjected to dead load and earthquake during its lifetime. The height of the shear wall is 75 feet, the width is 125 feet and the thickness is 15 inches. 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 16 Kip/ft and the safe shutdown earthquake (SSE) for design of the wall is taken to be 0.32 g. The specified concrete compressive strength is 5000 psi and yield strength of the reinforcing bar is 60,000 psi. The wall is designed according to the proposed Load combination criteria as shown in Section 5.5. The required horizontal and vertical reinforcement ratios are determined to be 0.00236 and 0.00523, respectively.

The probabilistic characteristics of loads and material strengths described in Section 3 are summarized in Tables 1 and 2, respectively. The limit states for flexure and shear as defined in Section 2 are reached at the base of the shear wall. In this study, the variations of structural resistance and dead load are included in the analysis using a Latin hypercube sampling technique.[14] The sample size is chosen to be ten; thus, ten values of f'_c, f_y, D and B, are chosen according to their distributions, and each value has equal probability. Table 3 gives the ten sets of the Latin hypercube samples and the corresponding conditional limit state probabilities for flexure and shear limit states i.e., $P_{c,m}^{(D+E)}$ and $P_{c,v}^{(D+E)}$. The average values of these ten conditional limit state probabilities are 2.52×10^{-11} and 4.10×10^{-10}. For a lifetime of 40 years, the flexure and shear limit state probabilities are 6.06×10^{-11} and 9.86×10^{-10}, respectively.

For the shear limit state, the fragility of the shear wall, which is defined as the conditional limit state probability given a peak ground acceleration, is also evaluated. The fragility data are tabulated in Table 4 and plotted in Fig. 4.

Table 1. Probabilistic Models for Loads.

Load	Model
Dead Load (D)	Time Invariant Normal Distribution With $\overline{D} = 1.0\ D_n$ and $CoV(D) = 0.07$, $D_n = 16$ Kip/ft per each floor
Earthquake (E)	Seismic Hazard Follows a Type II Distribution $1 - G_A(a) = \exp[-(a/\mu)^{-\alpha}]$; $\alpha = 2.7$, $\mu = 0.01765$ $S_{gg}(\omega) = S_o \dfrac{1 + 4\zeta_g^2(\omega/\omega_g)^2}{[1 - (\omega/\omega_g)^2]^2 + 4\zeta_g^2(\omega/\omega_g)^2}$ where $\omega_g = 5\pi$ rad/sec, $\zeta_g = 0.6$ $a_o = 0.05g$, $a_{max} = 0.64g$ Occurrence rate, $\lambda_E = 0.0601$ per year Mean duration, $\mu_{dE} = 20$ seconds

Table 2. Probabilistic Model for Material Strength.

Material Strength	Model
f'_c	Normal Distribution $\overline{f'_c} = 1.219 + 1.02\ f'_{cn}$ $f'_{cn} = 5000$ psi, $\overline{f'_c} = 6319$ 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. Conditional Limit State Probabilities With Latin Hypercube Samples.

Samples	f_c'(psi)	f_y(psi)	D(lb)	B	$P_{c,m}^{(D+E)}$	$P_{c,v}^{(D+E)}$
1	6.659 +3	7.362 +4	8.122 +6	0.808	6.016 -13	5.459 -11
2	5.978 +3	6.765 +4	7.547 +6	0.865	3.119 -11	3.602 -12
3	4.863 +3	8.452 +4	8.319 +6	1.339	7.196 -14	0
4	7.235 +3	6.297 +4	7.824 +6	0.913	3.086 -11	2.153 -13
5	6.207 +3	7.906 +4	7.965 +6	0.720	2.171 -13	4.041 -9
6	7.774 +3	7.599 +4	6.863 +6	1.115	3.670 -12	1.414 -18
7	6.915 +3	6.960 +4	7.193 +6	1.194	2.118 -11	0
8	5.722 +3	6.554 +4	7.390 +6	0.959	1.063 -10	1.961 -14
9	5.402 +3	5.892 +4	8.649 +6	1.005	5.379 -11	5.217 -16
10	6.430 +3	7.155 +4	7.688 +6	1.056	4.458 -12	4.082 -17
Average					2.52 -11	4.10 -10

NOTE: 6.659 + 3 = 6.659 x 10^3.

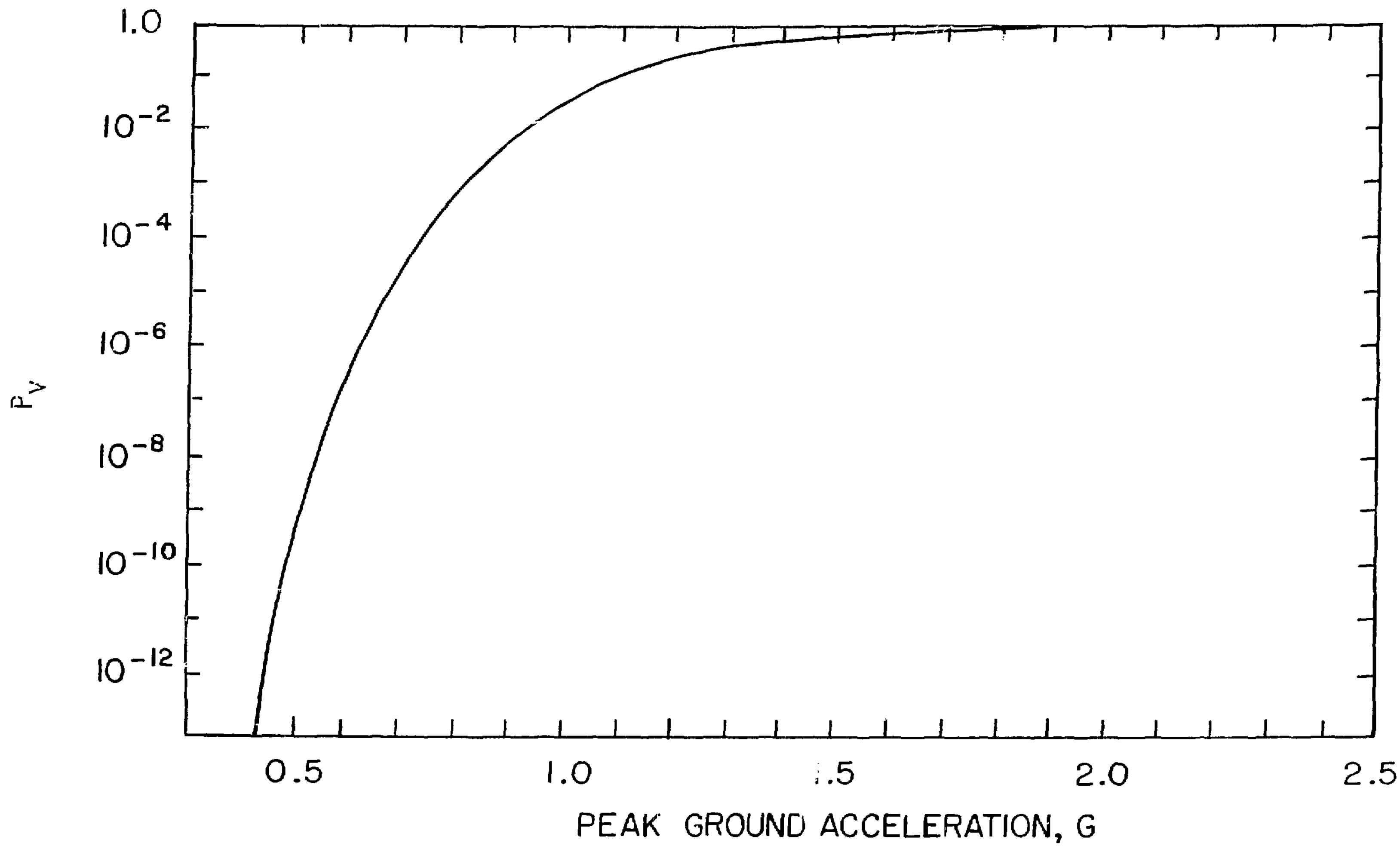

Fig. 4. Fragility Curve (Shear).

Table 4. Fragility Data (Shear).

PGA(g)	P_V
0.45	5.713 -13
0.50	2.234 -10
0.55	1.853 -08
0.60	5.359 -07
0.65	7.392 -06
0.70	5.975 -05
0.75	3.251 -04
0.80	1.310 -03
0.85	4.169 -03
0.90	1.095 -02
0.95	2.441 -02
1.00	4.714 -02
1.05	8.022 -02
1.10	0.123
1.15	0.173
1.20	0.229
1.25	0.291
1.30	0.357
1.40	0.493
1.50	0.622
1.60	0.733
1.80	0.884
2.00	0.959
2.20	0.992
2.40	0.999
2.60	1.000

5. LOAD COMBINATION CRITERIA FOR DESIGN OF SHEAR WALL STRUCTURES

A procedure for developing probability-based load combinations for the design of category I structures has been established.[11,14] Using this procedure, load factors for design of shear walls were determined.[15] The procedure is summarized as follows:

1. Select an appropriate load combination format.
2. Establish representative structures.
3. Define limit states and select a target limit state probability.
4. Assign initial values for all parameters (e.g., load and resistance factors) associated with the selected load combination format.
5. Design each representative structure.
6. Determine the limit state probability of each representative structure.
7. Compute the objective function measuring the difference between the target limit state probability and the computed limit state probability.
8. Determine a new set of parameters along the direction of maximum descent with respect to the objective function.
9. Repeat steps 5 to 8 until a set of parameters that minimizes the objective function is found.

5.1 Load Combination Format

The load and resistance factor design (LRFD) format[18] has been selected for this study. This format has been adopted in several specifications[1,4,5] and the Standard Review Plan, Section 3.8.4.[23] The LRFD format is simple enough to be used in routine design while offering sufficient flexibility to achieve consistent reliabilities in various design situations. If three loads, i.e., dead load, live load and earthquake are considered to act on the shear walls during a reference period, the load combinations in the LRFD format are:

$$1.2\ D + 1.0\ L + \gamma_{ES} E_{SS} \leqslant \phi_i R_i \tag{13}$$

$$0.9\ D - \gamma_{ES} E_{SS} \leqslant \phi_i R_i \tag{14}$$

where

- D = load effect due to design dead load
- L = load effect due to design live load
- E_{SS} = load effect due to safe shutdown earthquake (SSE)
- γ_{ES} = load factor for safe shutdown earthquake
- ϕ_i = resistance factor for the i-th limit state under consideration
- R_i = nominal structural resistance for the i-th limit state under consideration

It is assumed that design loads and nominal structural resistance are defined as in current standards. The load and resistance factors are determined so as to achieve the desired reliability. However, in this study the dead load factor, live load factor and resistance factors are preset to simplify the optimization. The mean value of the dead load is approximately equal to its nominal value and its variability is quite small. A dead load factor of 1.2 (or 0.9 when the dead load has a stablizing effect) has been found to be more than adequate to account for uncertainty in dead load.[1,8] Furthermore, experience with the treatment of live load as a companion load in conventional structures has shown that it is reasonable to preassign the live load factor of 1.0 (or zero if live load has a stabilizing effect).[8,11] The dead and live load factors in Eqs. 13 and 14 are the same as those appearing in the A58 load requirements.[1] With a few trials, it was found that if the resistance factor for shear, ϕ_v, is set to be 0.85 and the resistance factor for compression or compression with flexure, ϕ_m, is set to be 0.65, they will produce approximately the same optimum values of the load factor γ_{ES}. Hence, in this study, these resistance factors, which are similar to those specified in ACI Standard 349, are adopted.

5.2 Representative Shear Wall Structures

An important requirement for codified structural design is that all the structures designed according to a code should meet the code performance objectives which are expressed in probabilistic terms. In order to test if this requirement is satisfied, four representative (sample) structures are selected for evaluating the design criteria. In this study, representative shear wall structures are determined from examining existing shear walls in U.S. nuclear power plants. A low-rise three-story rectangular shear wall, as shown in Fig. 1, is chosen as a representative shear wall structure. The shear wall may be subjected to dead load, live load and in-plane earthquake forces. The ranges of the design parameters such as height-to-length ratio, material strengths, and design loads are determined and one, two or four representative values are selected to represent the range of each design parameter. Then the Latin hypercube sampling technique is used to identify sample shear walls using these representative design values. Four sample shear walls thus identified are shown in Table 5. With the design parameters in Table 5 specified, the remaining design parameters, which still need to be determined, are the wall thickness and the reinforcement.

Table 5. Representative Shear Wall Structures.

Design Parameters	Sample 1	Sample 2	Sample 3	Sample 4
Height (ft)	75	75	75	75
Width (ft)	75	125	100	150
Concrete Compressive Strength (psi)	4000	5000	5000	4000
Rebar Yield Strength (psi)	60,000	60,000	60,000	60,000
Superimposed Dead Load (Kip/ft)	16	16	16	16
Live Load (Kip/ft)	12	8	12	8
SSE (g)	0.17	0.32	0.25	0.50
Soil	Rock	Deep Cohesionless	Deep Cohesionless	Rock
Earthquake Duration (sec)	10	20	10	20

5.3 Design of Shear Walls

Each representative shear wall shown in Table 5 has to be designed according to the proposed load combinations with trial load and resistance factors, specified design loads, and nominal resistance. The shear strength determmned from Eq. 6 is proportional to the wall thickness. It is known that the shear limit state probability of a shear wall with larger wall thickness is less than that of a shear wall with smaller thickness, even through both shear walls are designed according to the same criteria. Thus, for the design of shear wall structures, the wall thickness cannot be assigned arbitrarily. Utilizing the nominal shear strength expression for shear walls in the ACI code and a horizontal wall reinforcement ratio of 0.0025, the following expression is used in this study to determine the appropriate wall thickness.

$$h \geq \frac{\frac{V_u}{\phi_v d} - \frac{N_u}{4 \ell_w}}{3.3\sqrt{f'_{cn}} + 0.0025 f_{yn}} \qquad (15)$$

where

h = thickness of a shear wall
V_u = factored shear force at a cross-section
N_u = factored axial force at a cross-section
ϕ_v = resistance factor for shear
ℓ_w = total length of a shear wall
d = effective length of a shear wall, $d = 0.8\ \ell_w$ for rectangular wall
f'_{cn} = nominal concrete compressive strength
f_{yn} = nominal yield strength of reinforcement

Once the wall thickness is determined, the remaining design parameter, which needs to be determined, is the required wall reinforcement. For the structural analysis of the shear wall, a beam element model is used. In this study, 3 beam elements are used to model each story; thus, a shear wall is represented by a beam model with 10 nodes. The mass used in the model is calculated from the mean values of dead and live loads, as specified in Section 3. The axial force, which results from dead load with or without live load, is obtained from static analysis. The shear and moment due to earthquake are obtained from response spectrum analysis. The horizontal response spectrum used in this study is the design spectrum specified in the Regulatory Guide (R.G.) 1.60.[6] The damping ratio is taken to be 7 percent of critical for the SSE, as specified in the R.G. 1.61.[7] The axial force, shear and moment thus obtained are combined using the proposed load combinations, i.e., Eqs. 13 and 14, with the trial load factors.

The nominal resistance of the shear wall is computed using the formula specified in the current ACI code. The minimum wall reinforcement can be determined such that the factored nominal resistance will be larger than the factored load effect. In practice, the designers usually provide reinforcement larger than the minimum requirement. In this study, however, the minimum rebar area will be used in design and reliability assessments. The representative shear walls designed by the procedure described above are shown in Table 6.

Table 6. Required Wall Thickness and Reinforcement Ratios (D+L+E_{SS}).

Sample	γ_{ES}	h (in)	ρ_m	ρ_h	ρ_n
1	1.1	8	0.00623	0.00148	0.00148
	1.2	8	0.00793	0.00213	0.00213
	1.3	8	0.00957	0.00278	0.00271
	1.4	9	0.00947	0.00266	0.00262
	1.5	10	0.00926	0.00257	0.00256
2	1.1	13	0.00265	0.00256	0.00256
	1.2	15	0.00284	0.00235	0.00235
	1.3	16	0.00315	0.00257	0.00256
	1.4	18	0.00331	0.00241	0.00241
	1.5	20	0.00334	0.00230	0.00230
3	1.1	10	0.00480	0.00278	0.00275
	1.2	12	0.00459	0.00232	0.00232
	1.3	13	0.00508	0.00245	0.00245
	1.4	14	0.00534	0.00256	0.00256
	1.5	15	0.00564	0.00267	0.00265
4	1.1	25	0.00230	0.00256	0.00256
	1.2	28	0.00255	0.00260	0.00260
	1.3	32	0.00270	0.00250	0.00250
	1.4	36	0.00277	0.00245	0.00245
	1.5	40	0.00284	0.00243	0.00243

NOTE: 1. ρ_m is vertical reinforcement ratio required by flexure.

2. ρ_h and ρ_n are horizontal and vertical reinforcement ratios, respectively required by shear.

5.4 Determination of Load Factors

The load and resistance factors are determined to be consistent with a specified target limit state probability for each limit state. The selection of a target limit state probability should consider many factors, e.g., the characteristics of the limit states, the consequence of failure, and the risk evaluation and damage cost. Hence, the target reliability may not necessarily be the same for different limit states. It is anticipated that the target limit state probability will be set by the regulatory authority and/or the code committee.

Once a target limit state probability $P_{f,T}$ is specified, the load and resistance factors are determined such that the limit state probabilities of the sample shear walls are sufficiently close to the target limit state probability. The closeness is measured by an objective function defined as follows:

$$\Omega(\gamma,\phi) = \sum_{i=1}^{N} w_i \, (\log P_{f,i} - \log P_{f,T})^2 \qquad (16)$$

where N is the total number of representative shear wall structures, $P_{f,i}$ is the limit state probability computed for the i-th sample structure, w_i represents a weight factor for the i-th sample structure. In the Latin hypercube sampling technique, it is assumed that each sample in Table 5 is equally representative, and thus, $w_i = 1.0$. The optimum values of the load and resistance factors are then derived by minimizing the objective function Ω.

The limit state probabilities of the shear walls shown in Table 6 under the three loads in 40 years, are shown in Table 7. It is to be noticed that the limit state probability for shear is calculated on the basis of the required shear reinforcement without including the reinforcement required for flexure. Similarly, the limit state probability for flexure is computed without considering the shear reinforcement. Using these limit state probabilities, the objective function Ω can be computed for several values of γ_{ES} and $P_{f,T}$. Figure 5 shows parabolic curves plotted through these values of the objective function. For $P_{f,T} = 1.0 \times 10^{-6}$ per 40 years, the optimum values of γ_{ES} are 1.366 and 1.411 for shear and flexure limit states, respectively. For $P_{f,T} = 1.0 \times 10^{-5}$ per 40 years, the optimum values of γ_{ES} are 1.214 and 1.267 for shear and flexure limit states, respectively. Hence, γ_{ES} is recommended as 1.4 or 1.2 corresponding to the specific target limit state probability mentioned above.

Table 7. Limit State Probabilities ($D+L+E_{SS}$).

Sample	Limit State	$\gamma_{ES}=1.1$	$\gamma_{ES}=1.2$	$\gamma_{ES}=1.3$	$\gamma_{ES}=1.4$	$\gamma_{ES}=1.5$
1	Flexure	3.349 -4	1.240 -4	4.670 -5	1.315 -5	3.930 -6
	Shear	3.312 -4	1.829 -4	9.847 -5	4.249 -5	1.681 -5
2	Flexure	5.452 -5	9.453 -6	2.041 -6	2.586 -7	4.507 -8
	Shear	2.002 -5	3.087 -6	7.162 -7	9.165 -8	1.076 -8
3	Flexure	3.483 -5	6.607 -6	9.835 -7	1.862 -7	2.779 -8
	Shear	4.302 -5	6.414 -6	1.507 -6	3.327 -7	6.842 -8
4	Flexure	7.968 -4	2.195 -4	4.635 -5	1.105 -5	2.511 -6
	Shear	1.021 -4	2.736 -5	5.466 -6	1.028 -6	1.870 -7

5.5 Proposed Load Combination Design Criteria

If the target limit state probability is selected as 1.0×10^{-6} per 40 years (equivalent to 2.5×10^{-8} per year), the proposed load combinations for design of the shear walls subjected to dead load, live load and earthquake during the service life are as follows:

$$\left.\begin{array}{l} 1.2D + 1.0L + 1.4\,E_{SS} \\ 0.9D \qquad\quad - 1.4\,E_{SS} \end{array}\right\} \leq \phi_i R_i \qquad (17)$$

The resistance factor for shear, ϕ_v, is 0.85 and the resistance factor for compression or compression with flexure, ϕ_m, is 0.65. The determination of the nominal design values for loads and nominal resistance follows current practice.

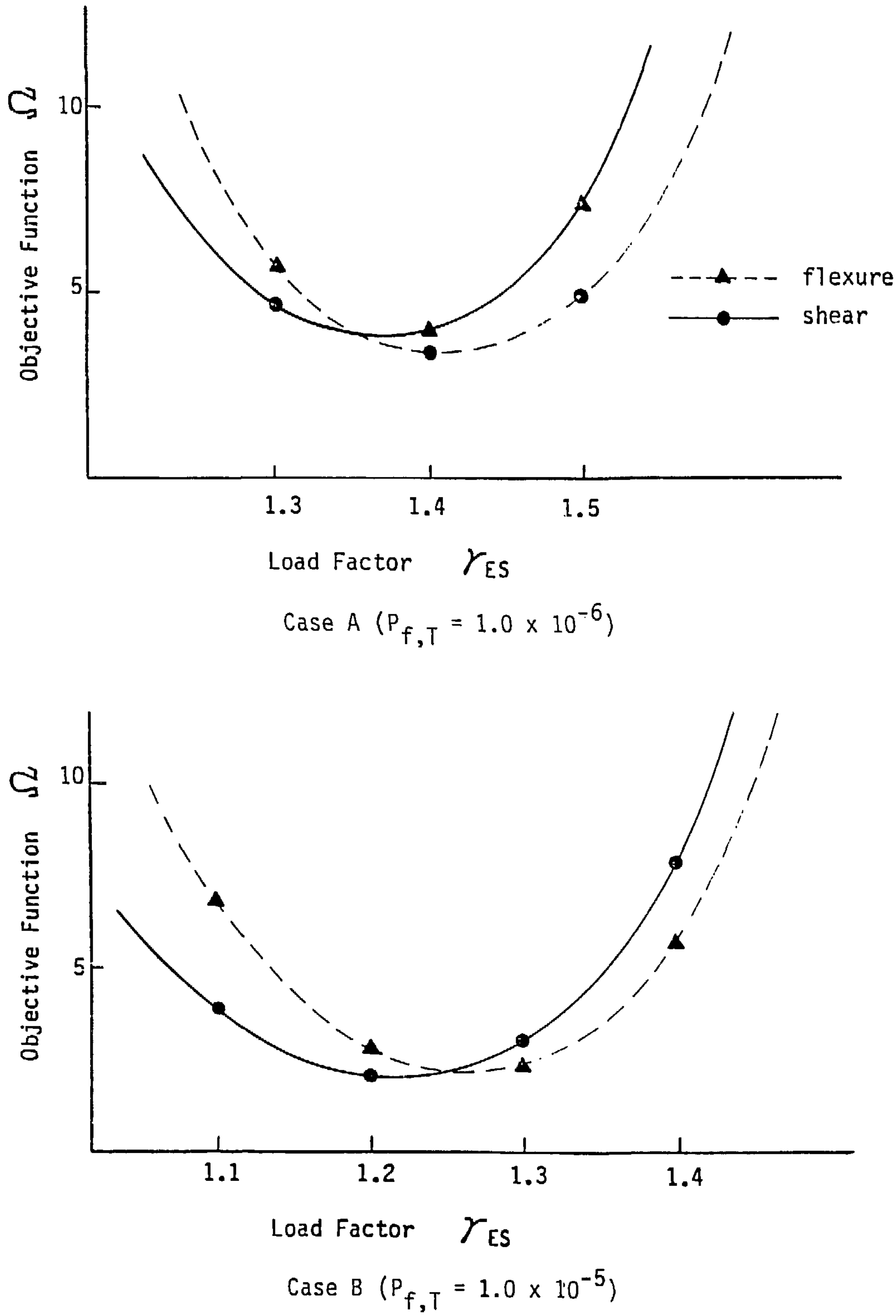

Fig. 5. Objective Function vs. Load Factor ($D+L+E_{SS}$).

The proposed load combinations are similar to those specified in ANSI Standard A58.1-1982.[1] The proposed load factor for earthquake in this study is 1.4 instead of 1.5 in the A58 standard. However, the definition of earthquake is quite different from the design earthquake in the A58 Standard. In general, the safe shutdown earthquake specified for nuclear structures is much stronger than that specified for conventional structures. Another difference appears in the resistance factor for shear. In this study, the resistance factor for shear is recommended to be 0.85, while 0.70 was recommended for use with the A58 load criteria.[16] In this connection, however, it should be noted that the mean shear capacity of low-rise walls, as described by Eqs. 1-4, is much higher with respect to the nominal shear capacity specified by ACI[4,5] than is the mean shear capacity of slender walls and beams.[8,10]

Reference 15 compared two shear wall structures designed using the proposed design criteria and the current ACI-349 code. The results with respect to shear limit state are shown in Tables 8 and 9. This comparison revealed that the proposed design criteria, based on the target limit state probability of 1.0×10^{-6} per 40 years, are more stringent than those specified in ACI-349.

Table 8. Shear Walls Designed With ACI and Proposed Criteria.

Sample	Design Criteria	Thickness (in)	ρ_n	ρ_h
2	ACI	9	0.00263	0.00264
	Proposed	15	0.00236	0.00236
4	ACI	18	0.00271	0.00271
	Proposed	30	0.00245	0.00245

Table 9. Reliability Assessments of Shear Walls.

Design Criteria	Limit State	Sample 2	Sample 4
ACI	Shear	1.644 -4	3.614 -4
Proposed	Shear	1.453 -7	1.385 -6

6. CONCLUDING REMARKS

A reliability analysis method for shear walls has been developed. In this method, the shear wall is modelled by beam elements. The limit state for flexure is defined according to ultimate strength analysis for combined axial forces and bending moments. The shear limit state is established from test results. At present, three loads, i.e., dead load, live load and in-plane earthquake, are considered in the reliability analysis. The randomness and other uncertainties of the structural resistance are included in the reliability analysis using a Latin hypercube sampling technique. Based on the above information, the limit state probabilities of a shear wall can be computed for flexure and shear limit states. This reliability analysis method can be used to evaluate the reliability level of existing shear walls and to derive fragility curves of shear walls for PRA studies.

Utilizing the reliability analysis method described above, load combination criteria for the design of shear wall structures have also been established. The proposed design criteria are in the load and resistance factor design (LRFD) format. The load factor for SSE is determined for a target limit state probabilities of 1.0×10^{-6} or 1.0×10^{-5} during a lifetime of 40 years. The proposed load combinations according to $P_{f,T} = 1.0 \times 10^{-6}$ per 40 years are summarized in Section 5.5. It is clear that the use of such criteria would entail no major change in the way that routine structural design calculations are performed. However, in contrast to existing design procedures, the proposed criteria are risk-consistent and have a well-established rationale.

On the basis of the data used in this study, shear walls designed by current ACI-349 for earthquake loading , but without tornado loads, may not be adequate for the reliability level specified. This may be because the target limit state probability is too small or because of other assumptions made in our analysis. However, it may be due to the fact that the code committee does not consider the whole range of seismic hazard. If the a_{max} is larger than two times the SSE value, the difference will be even greater. We believe that this problem should be given proper attention. However, this does not necessarily imply that the current shear walls used in the nuclear plants are unsafe. Since shear walls are designed to resist tornado-borne missiles, they are more massive than would be required to resist only earthquake loadings.

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