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PROBABILITY BASED LOAD COMBINATION CRITERIA FOR DESIGN OF SHEAR WALL STRUCTURES

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ABSTRACT

This report describes the development of probability-based load combination criteria for the design of reinforced concrete shear wall structures subjected to dead load, live load and earthquake. The proposed design criteria are in the load and resistance factor design (LRFD) format. The load and resistance factors are determined for flexure and shear limit states and target limit state probabilities. The flexure limit state is defined according to the ACI ultimate strength formula. The shear limit state is established from experimental results.

In order to test whether the proposed criteria meet the reliability-based performance objectives, four representative structures are selected using a Latin hypercube sampling technique. These representative structures are designed using trial load and resistance factors. Then, a reliability analysis method is employed to assess their reliabilities. An objective function is defined and a minimization technique is developed to find the optimum load factors. In this study, the resistance factors for shear and flexure, and load factors for dead and live loads are preassigned to simplify the minimization. The load factor for SSE is determined for the target limit state probability of 1.0×10^{-6} or 1.0×10^{-5} with a lifetime of 40 years.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

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EXECUTIVE SUMMARY

Shear walls are used in many category I structures as the primary structural system for resisting lateral loads such as earthquakes. These shear walls usually have a low height-to-length ratio and exist either as part of a rectangular box or as individual walls. The current load combination criteria for design of shear wall structures are specified in ACI Standard 349 and the NRC Standard Review Plan (SRP), Section 3.8.4. The load and resistance factors in these specifications are based on collective judgement and experience and thus, their application in structural designs may result in unknown and non-uniform reliability. By utilizing structural probabilistic methods, it is possible to modify the load and resistance factors so that consistent safety margins for shear wall structures will be attained under various loading conditions.

This report details the development of probability-based criteria for the design of shear wall structures. Proposed design criteria are specified in the load and resistance factor design (LRFD) format. This format is similar to that used in the current standards mentioned above. These load and resistance factors in LRFD format were determined on the basis of limit states and a target limit state probability. Thus, while the format of the proposed design criteria resembles that of the currently used standards, it nevertheless fully reflects the probabilistic nature of the design parameters.

For this study, two limit states were considered. The flexure limit state which is defined according to conventional ultimate strength analysis and the shear limit state which is established on the basis of experimental data performed on low-rise walls. In this work, three loads, i.e., dead load, live load and in-plane earthquake were considered to act on the shear walls. The proposed load combinations were derived for target limit state probabilities of 1.0×10^{-5} or 1.0×10^{-6} per 40 years of plant life. The proposed criteria are a valuable asset for decision-making bodies such as NRC who can utilize them for improving current provisions of the Standard Review Plan.

1. INTRODUCTION

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 a low height-to-length ratio and exist either as part of a rectangular box or as individual walls. The current load combination criteria for design of shear wall structures are specified in ACI Standard 349[5] and the NRC Standard Review Plan (SRP), Section 3.8.4.[23] The load and resistance factors in these specifications are determined based on collective judgement and experience. Modifications to these load and resistance factors can be made using probabilistic methods so that consistent safety margins for shear wall structures can be attained under various conditions.

A procedure for developing probability-based load combination criteria for the design of category I structures has been established.[11,15] Using this procedure, load factors for the design of concrete containments 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.

This report describes the development of probability-based load combinations for the design of low-rise shear wall structures. The shear wall structures are subjected to in-plane earthquake forces, and dead loads with or without live loads. The shear and flexure limit states for the shear walls are established. Using the procedure summarized above, load and resistance factors for the design of shear walls are determined for the selected target limit state probabilities.

2. 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 NRC 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.

The general expression of the LRFD format is given in Ref. 11. In this format, the factored nominal structural resistance is required to be larger than or equal to the sum of factored design load effects. In the code, it would actually be a set of design equations. For example, if three loads, i.e., dead load, live load and earthquake are considered, the load combinations in the LRFD format are:

$$1.2\ D + 1.0\ L + \gamma_{ES} E_{SS} \leqslant \phi_i R_i \qquad (1)$$

$$0.9\ D - \gamma_{ES} E_{SS} \leqslant \phi_i R_i \qquad (2)$$

where

- D = Dead loads or their related internal moments and forces
- L = live loads or their internal moments and forces including movable equipment loads
- 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

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 stabilizing 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 a value equal to 1.0 (or zero if live load has a stabilizing effect).[8,11] The dead and live load factors in Eqs. 1 and 2 are the same as those appearing in the A58 load requirements.[1] The determination of resistance factors for flexure and shear is described in Section 7; they are similar to those specified in ACI Standard 349.

3. 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 the existing shear walls in the 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[15] is used to identify sample shear walls using these representative design values. Four sample shear walls thus identified are shown in Table 1. With the design parameters in Table 1 specified, the remaining design parameters, which still need to be determined, are the wall thickness and the reinforcement.

Table 1. Representative Shear Wall Structures.

Design Parameters	Sample 1	Sample 2	Sample 3	Sample 4
Height (ft)	75	75	75	75
Length (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

4. 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. Similarly, the probabilistic model for structural resistance must be establisled.

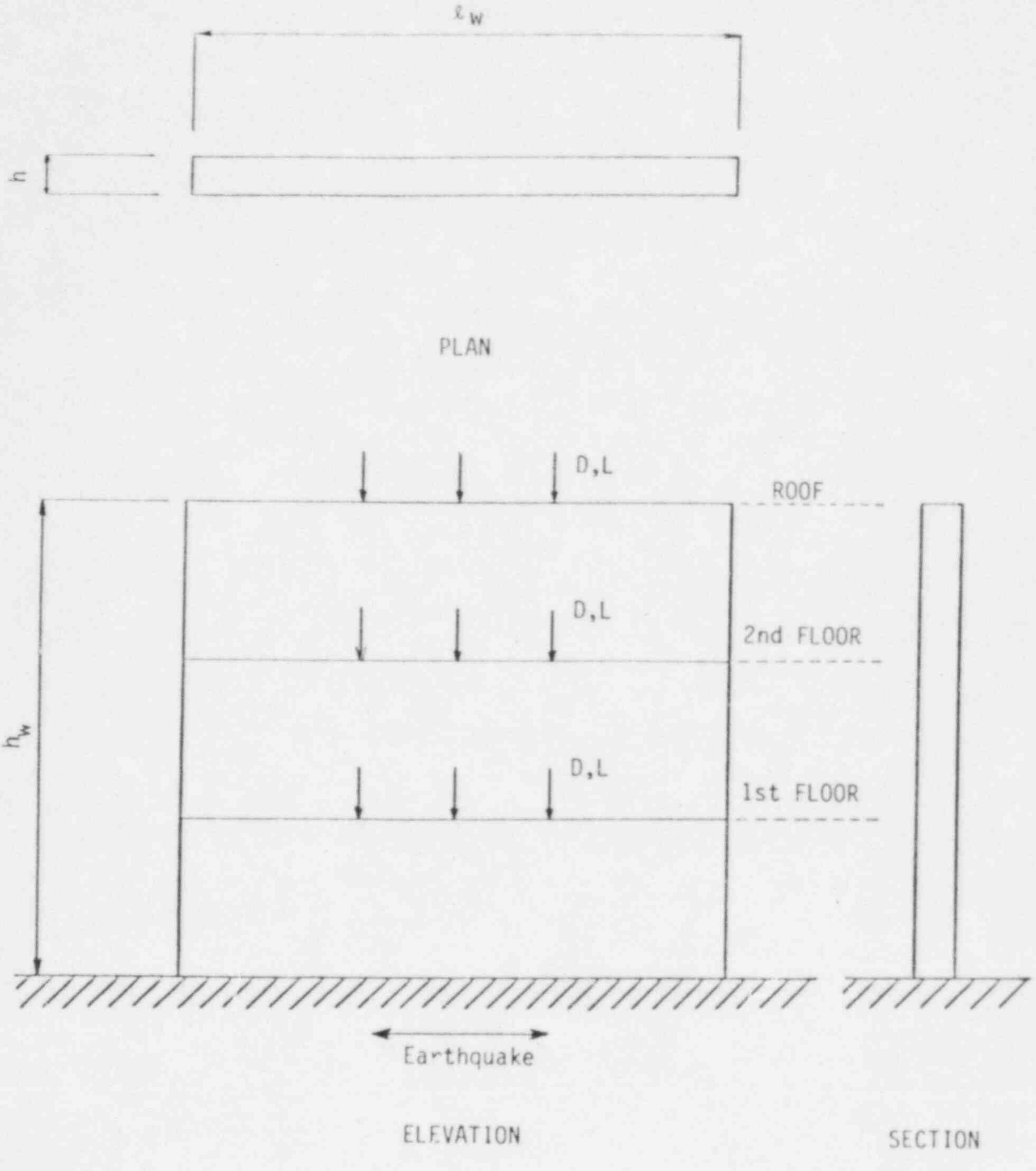


Fig. 1. Representative Shear Wall Structure.

4.1 Dead Load

Dead load is a static load and acts permanently on structures. 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 structural weights are small.[11,13,14] Since the structural weights contribute the major portion of the dead load, their statistics will dominate the statistics for dead load. Dead load is assumed to be normally distributed.[11,14] 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.[11,15]

4.2 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 loads data from the consensus estimation survey were anelyzed as shown 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.[15] 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.

4.3 Earthquake

The seismic hazard at the site of a nuclear power plant is described by a seismic hazard curve. A seismic hazard curve, as shown in Fig. 2, 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],

$$1 - G_A(a) = F_A(a) = \exp\left[-(a/\mu)^{-\alpha}\right] \qquad (3)$$

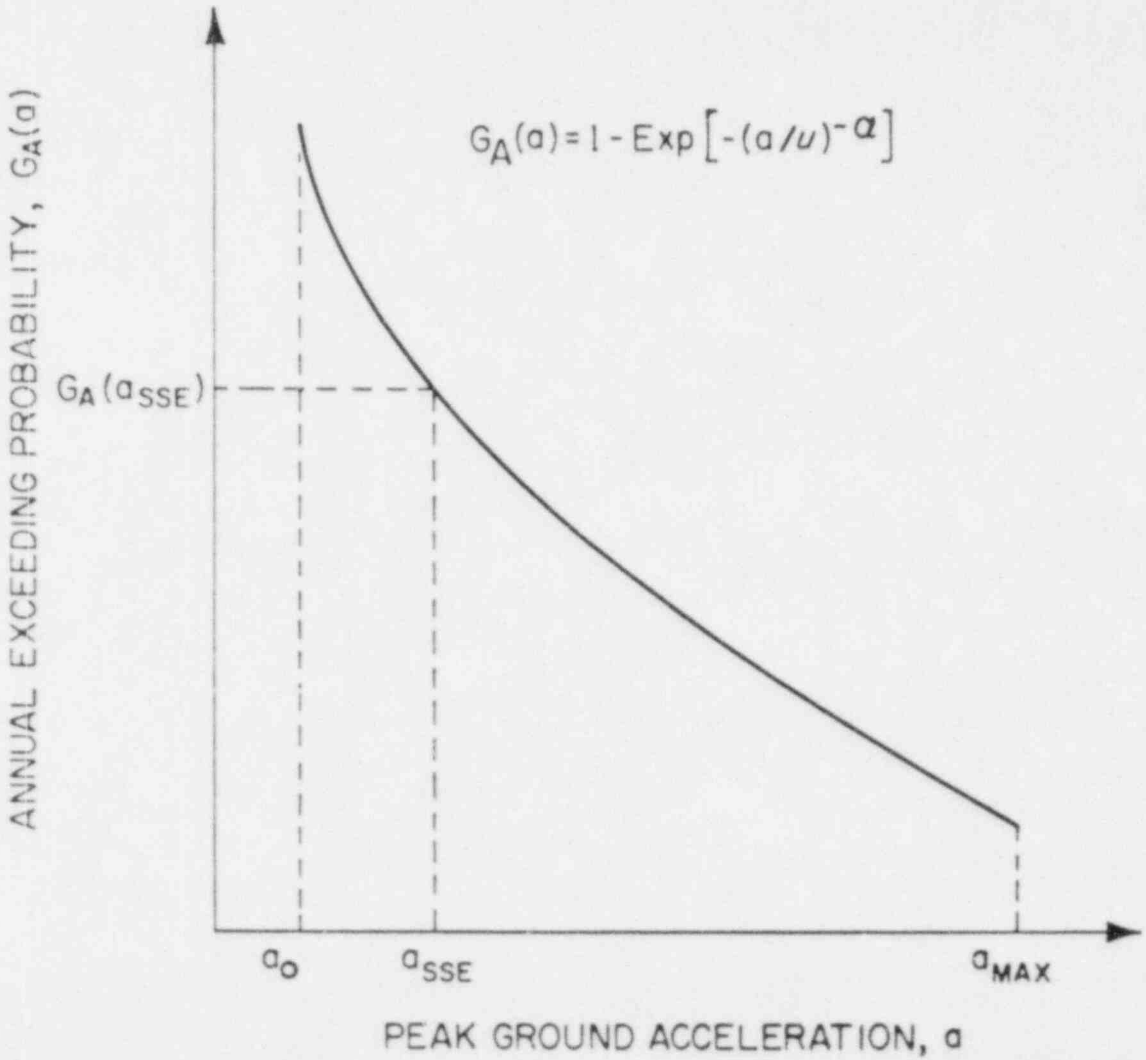


Fig. 2. Seismic Hazard Curve.

where α and μ are two parameters to be determined. The value of α for the U.S. iw estimated to be 2.7.[15] 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] This assumption implies that the operating basis earthquake (OBE), which is usually one-half the SSE, has a mean recurrence interval of only 385 years. Figure 3 shows the comparison of the hazard curve used in this study and the hazard curves with 50 percent confidence for eight specific plant sites in the Eastern United States.[3] From this figure, it can be seen that the hazard curve used in this study compares well with six out of the eight curves.

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. The effects of different values of a_{max} on the load factors are reported in Ref. 15. 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],

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

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] The mean duration of the stationary phase of the earthquake acceleration is assumed to be 10 or 20 seconds in this study.

4.4 Material Properties

In order to perform a reliability analysis of e 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.

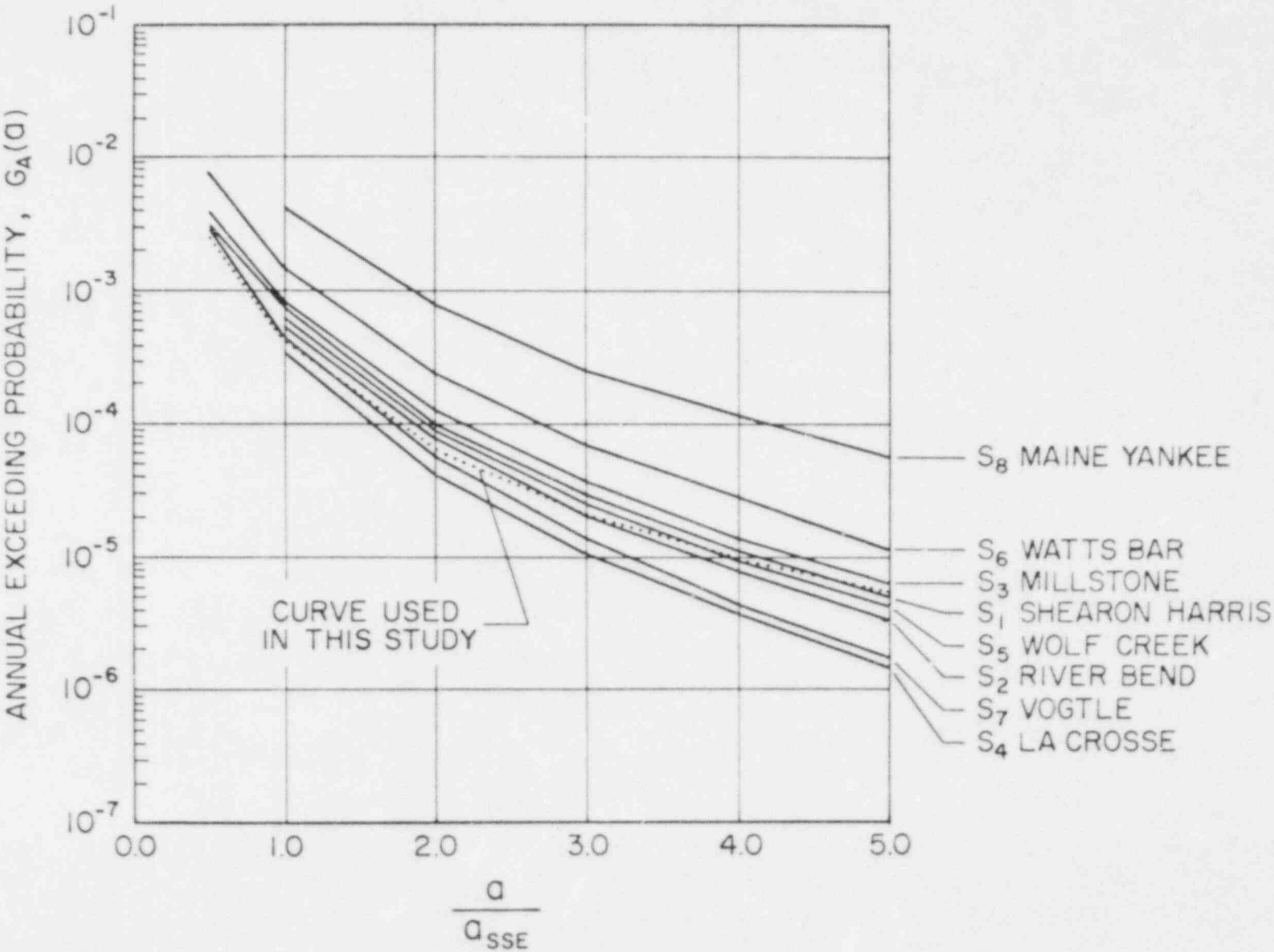


Fig. 3. Comparison of Seismic Hazard Curves.

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, $\bar{f}'_c$, equal to,[10]

$$\bar{f}'_c = 1219 + 1.02\, f'_{cn} \qquad \text{(psi)} \qquad (5)$$

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.

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 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.

5.1 Flexure Limit State

The flexure limit state for shear walls is defined analytically according to ultimate strength analysis of reinforced concrete. 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

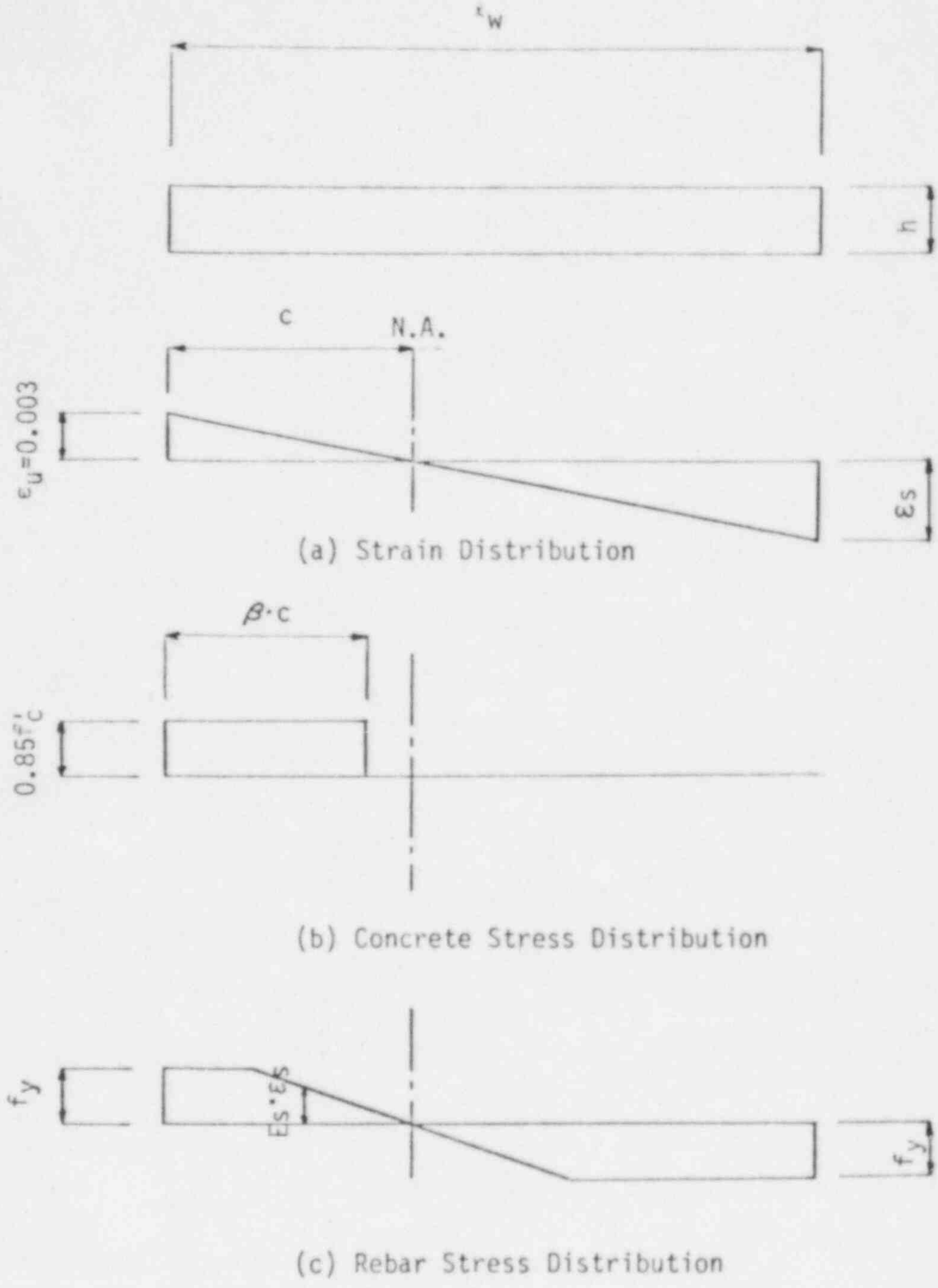


Fig. 4. Stress and Strain Distributions.

0.003; while the yielding of rebars is permitted. Based on the above definition of the limit state, a limit state surface can be constructed for given geometry and rebar arrangement in terms of the axial force and bending moment on 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 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'" are determined from zero axial force. 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 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[10], as described in Section 4.4.

5.2 Shear Limit State

The 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. The ultimate shear strength of a shear wall expressed in units of force/area, v_u, is

$$v_u = v_c + v_s \qquad (6)$$

in which v_c and v_s are the contributions of concrete and reinforcement to the 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,

$$v_c = 8.3\sqrt{f'_c} - 3.4\sqrt{f'_c}\left(\frac{h_w}{\ell_w} - \frac{1}{2}\right) + \frac{N_u}{4\ell_w h} \;;\; \frac{1}{4} \leq \frac{h_w}{\ell_w} \leq 1.0 \qquad (7)$$

in which N_u is 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 less. However, it was less effective if height-to-length ratio is equal to 1.

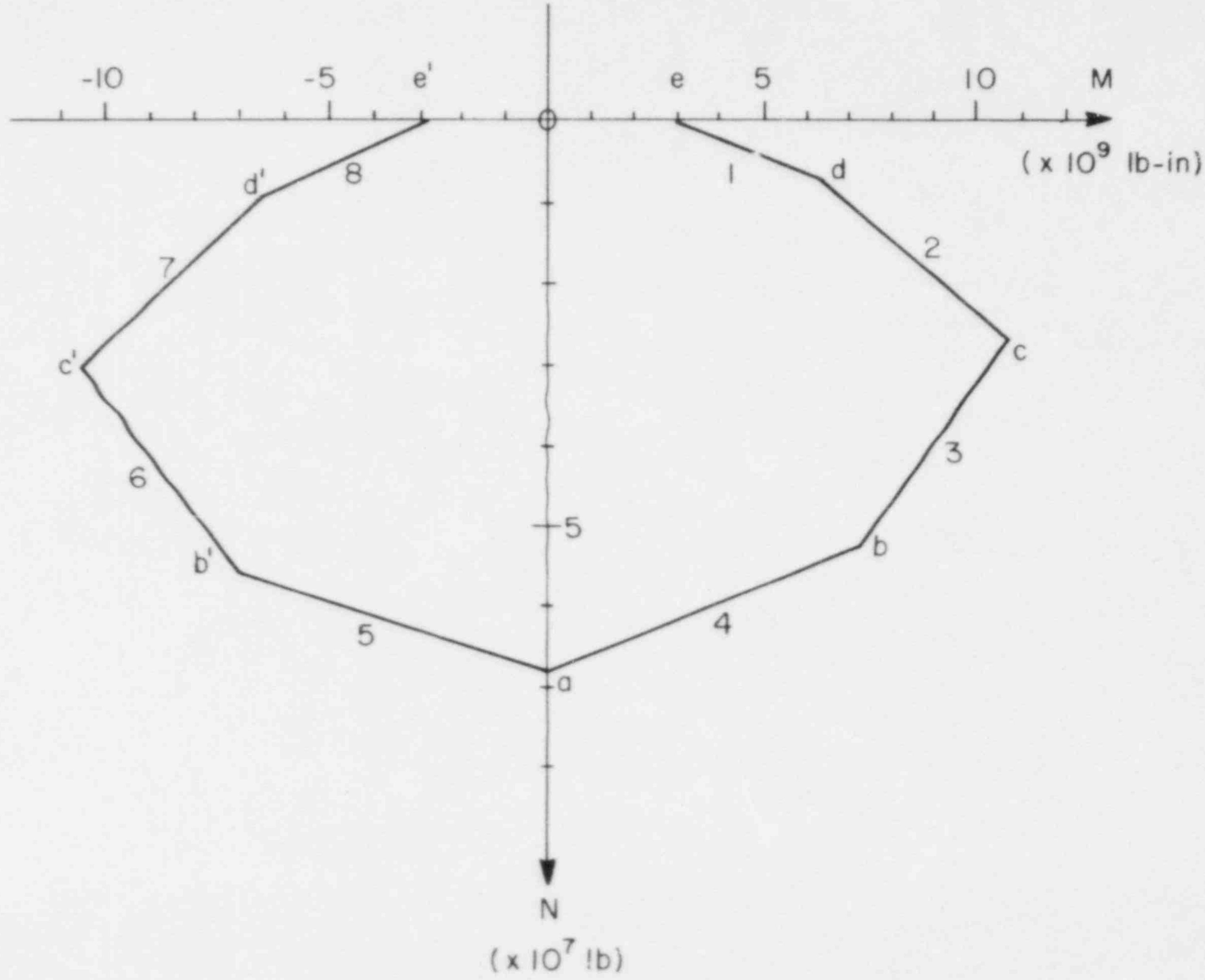


Fig. 5. Flexure Limit State Surface.

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],

$$v_s = (a\ \rho_h + b\ \rho_n)f_y \tag{8}$$

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

$$b = \begin{cases} 1 & ; \quad \dfrac{h_w}{\ell_w} < 1/2 \\ 2-2\,\dfrac{h_w}{\ell_w} & ; \quad 1/2 \leq \dfrac{h_w}{\ell_w} \leq 1 \\ 0 & ; \quad \dfrac{h_w}{\ell_w} > 1 \end{cases} \tag{9}$$

and

$$a = 1 - b$$

Both horizontal and vertical rebars actually are partially effective outside the given limits, but Eq. 9 is not sensitive to these limits as long as horizontal and vertical rebars both are used.

Gergely[12] 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:

$$v_u = 0.25\ f'_c \tag{10}$$

However, Eq. 10 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. 6-9 or Eq. 10. The total ultimate shear strength V_u is computed as

$$V_u = v_u\ h\ d \tag{11}$$

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. 11, a shear limit state surface can be constructed for the 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. 6-9 and lines 10 and 11 are governed by Eq. 10.

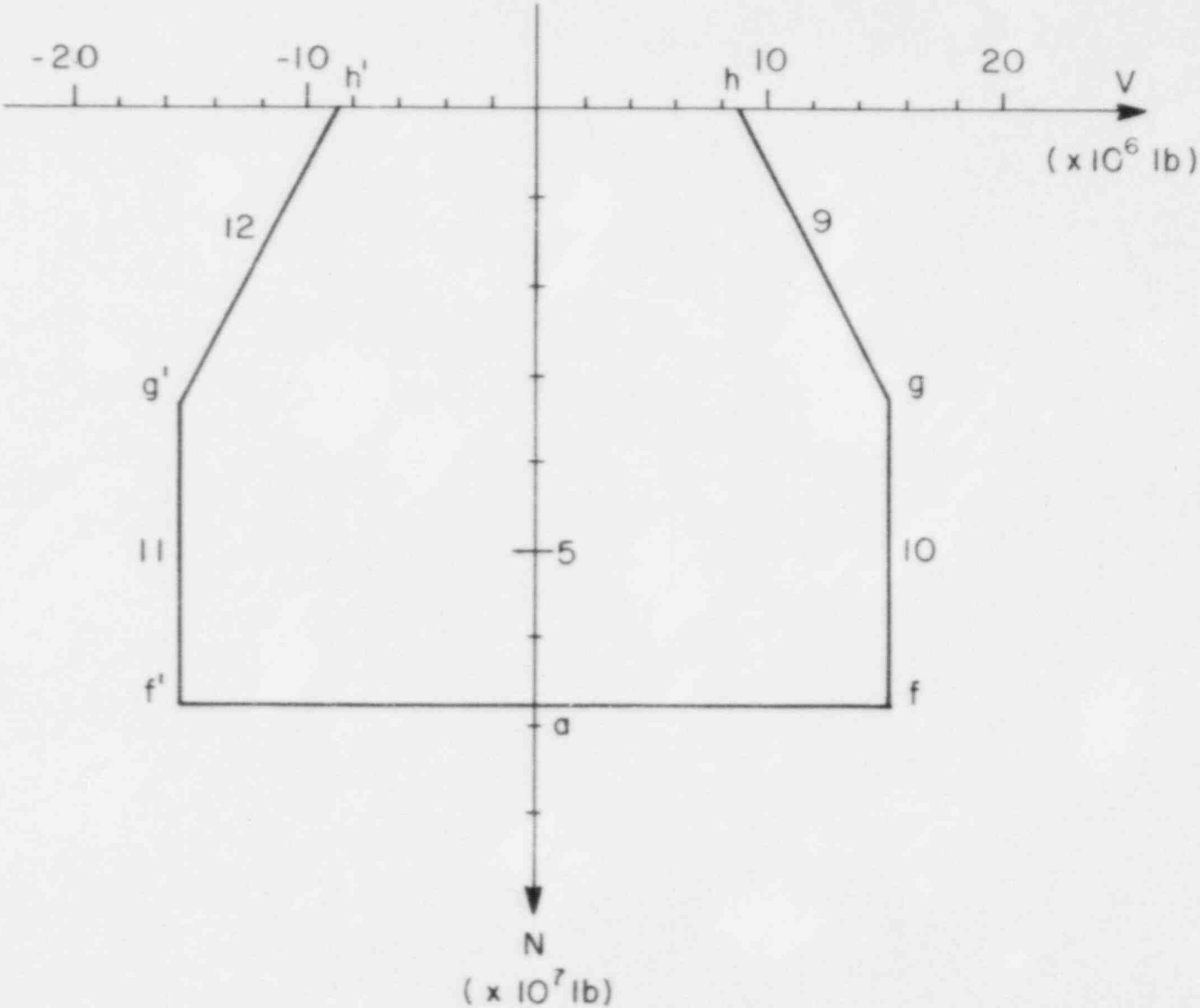


Fig. 6. Shear Limit State Surface.

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

$$V_u = B\,\overline{V}_u \tag{12}$$

where $\overline{V}_u$ is the mean value determined from Eq. 11 with mean values of f'_c and f_y. B is a lognormal random variable describing inherent randomness and modeling error with unit mean value and coefficient of variation of 0.19. In this study, the shear strength obtained from Eq. 12 is used for the reliability assessment of the shear wall.

6. DESIGN OF SHEAR WALLS

Each representative shear wall shown in Table 1 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 determined from Eq. 11 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 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.

$$h \geq \frac{\dfrac{V_u}{\phi_v d} - \dfrac{N_u}{4\ell_w}}{3.3\sqrt{f'_{cn}} + 0.0025 f_{yn}} \tag{13}$$

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 as shown in Fig. 7. The mass used in the model is calculated from the mean values of dead and live loads, as specified in Section 4.

Fig. 7. Beam Element Model For Shear Wall.

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. 1 and 2, 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, 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.

7. DETERMINATION OF LOAD FACTORS

The load and resistance factors are determined according to 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:

$$\Omega(\gamma,\phi) = \sum_{i=1}^{N} w_i \, (\log P_{f,i} - \log P_{f,T})^2 \tag{14}$$

where N is the total number of representative shear wall structures and $P_{f,i}$ 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 1 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.

7.1 Dead Load and Safe Shutdown Earthquake

In this section, we assume that the shear wall structures are subjected only to dead load and earthquake without live load during their lifetimes. The proposed load combinations are Eqs. 1 and 2, in which L is set equal to zero. 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_c, 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 will be adopted. Using the design parameters specified in Table 1 and a trial value of γ_{ES}, the thickness of each representative shear wall can be determined by Eq. 13. Once the thickness of the shear wall is determined, the design procedure described in Section 6 is utilized to determine the required reinforcement ratios for shear and flexure separately. The required thickness and reinforcement ratios of four sample shear walls are shown in Table 2.

For reliability assessment of each representative shear wall, the reliability analysis method described in Ref. 21 is used. The probabilistic characteristics of loads and material strength are delineated in Section 4. The random resistance for shear and flexure are described in Section 5. The Latin hypercube samplmng technique is used to include these variations in the reliability assessment and the sample size is chosen to be ten. As an example, ten values of f_c', f_y, D and B which are selected according to their distribution are shown in Table 3. Then, following the Latin hypercube sampling technique, ten sample sets are obtained and shown in Table 4. Each of the ten samples in Table 4 is used to compute the limit state probability of the shear wall during a lifetime of 40 years. The average values of these ten limit state probabilities are shown in Table 5 for several assigned values of load factor γ_{ES}. The limit state probability for shear is calculated on the basis of the required shear reinforcement without including of the reinforcement required for flexure. Similarly, the limit state probability for flexure is computed without considering the shear reinforcement.

The objective function defined by Eq. 14 is then used to determine the optimum load factor, γ_{ES}. In this study, the dead load factor and resistance factors for flexure and shear are preset; only the load factor for SSE needs to be determined. The target limit state probability, $P_{f,T}$, is assumed to be 1.0×10^{-5} per 40 years (2.5×10^{-7} per year) or 1.0×10^{-6} per 40 years (2.5×10^{-8} per year). Using the limit state probabilities of the sample shear walls in Table 5, the objective function Ω is computed at the selected values of γ_{ES} and tabulated in Table 6. Parabolic curves are plotted through these values, as shown in Fig. 8. From this figure, for $P_{f,T} = 1.0 \times 10^{-6}$ per 40 years, the optimum load factors for SSE, obtained from considering shear and flexure, are 1.365 and 1.413, respectively. Similarly, for $P_{f,T} = 1.0 \times 10^{-5}$ per 40 years, the optimum values of γ_{ES} are 1.207 and 1.269 for shear and flexure limit states, respectively.

Table 2. Required Wall Thickness and Reinforcement Ratios($D+E_{SS}$).

Sample	γ_{ES}	h (in)	ρ_m	ρ_h	ρ_n
1	1.1	8	0.00243	0.00002	0.00002
	1.2	8	0.00384	0.00054	0.00054
	1.3	8	0.00508	0.00105	0.00105
	1.4	8	0.00628	0.00157	0.00157
	1.5	8	0.00775	0.00208	0.00208
2	1.1	11	0.00186	0.00236	0.00236
	1.2	12	0.00230	0.00252	0.00252
	1.3	14	0.00241	0.00221	0.00221
	1.4	15	0.00277	0.00236	0.00236
	1.5	16	0.00315	0.00250	0.00250
3	1.1	8	0.00314	0.00228	0.00228
	1.2	9	0.00371	0.00227	0.00227
	1.3	10	0.00412	0.00227	0.00227
	1.4	11	0.00422	0.00229	0.00229
	1.5	12	0.00454	0.00230	0.00230
4	1.1	21	0.00197	0.00250	0.00250
	1.2	24	0.00219	0.00245	0.00245
	1.3	27	0.00230	0.00243	0.00243
	1.4	30	0.00249	0.00245	0.00245
	1.5	33	0.00270	0.00248	0.00248

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.

Table 3. Distributions of f_c', f_y, D and B.

Probability	f_c'	f_y	D	B
.050	.407875E4	.589256E5	.368243E7	.720732E0
.150	.453011E4	.629916E5	.385967E7	.808228E0
.250	.479862E4$	.655423E5	.396511E7	.865238E0
.350	.501315E4	.676541E5	.404935E7	.913662E0
.450	.520578E4	.696084E5	.412499E7	.959449E0
.550	.539222E4	.715536E5	.419821E7	.100595E1
.650	.558485E4	.736205E5	.427385E7	.105636E1
.750	.579928E4	.759927E5	.435809E7	.111548E1
.850	.606789E4	.790698E5	.446353E7	.119417E1
.950	.651925E4	.845258E5	.464077E7	.133913E1

Table 4. Latin Hypercube Samples.

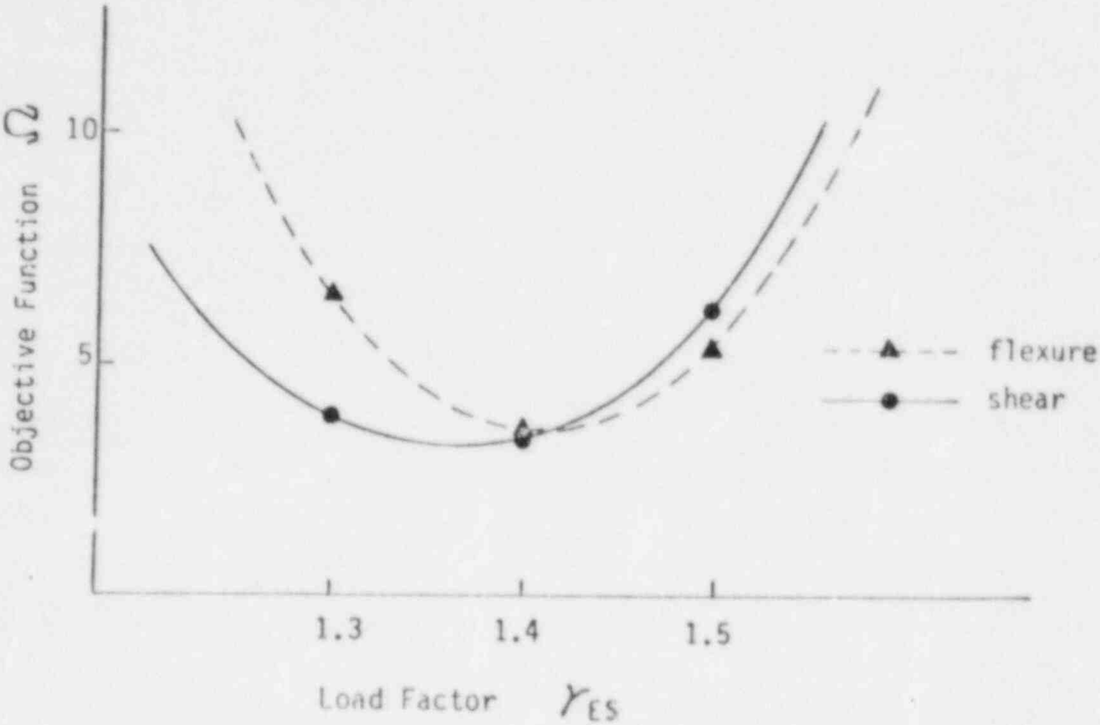
Sample Set	f_c'	f_y	D	B
1	.558485E4	.790698E5	.368243E7	.105636E1
2	.579938E4	.845258E5	.419821E7	.100595E1
3	.407875E4	.715536E5	.446353E7	.720732E0
4	.506789E4	.736205E5	.404935E7	.133913E1
5	.453011E4	.696084E5	.396511E7	.913662E0
6	.520578E4	.629916E5	.464077E7	.808228E0
7	.539222E4	.676541E5	.385967E7	.111548E1
8	.501315E4	.759927E5	.435809E7	.865238E0
9	.479862E4	.589256E5	.427385E7	.119417E1
10	.651925E4	.655432E5	.412499E7	.959449E0

Table 5. Limit State Probabilities ($D+E_{SS}$).

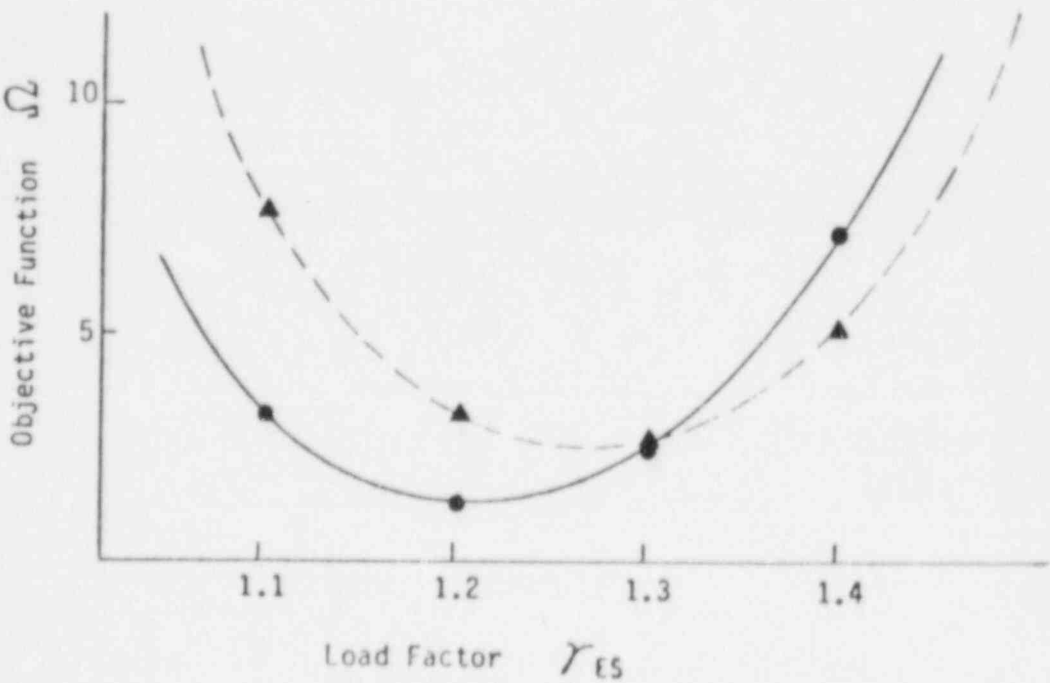
Sample	Limit State	$\gamma_{ES}=1.1$	$\gamma_{ES}=1.2$	$\gamma_{ES}=1.3$	$\gamma_{ES}=1.4$	$\gamma_{ES}=1.5$
1	Flexure	5.316 -4	1.571 -4	5.314 -5	1.902 -5	5.410 -6
	Shear	2.129 -4	1.167 -4	6.243 -5	3.262 -5	1.667 -5
2	Flexure	6.309 -5	1.284 -5	2.182 -6	3.325 -7	3.589 -8
	Shear	2.069 -5	4.800 -6	5.625 -7	1.453 -7	1.990 -8
3	Flexure	4.018 -5	6.133 -6	9.073 -7	2.093 -7	2.550 -8
	Shear	4.050 -5	9.125 -6	1.871 -6	3.516 -7	6.069 -8
4	Flexure	7.992 -4	2.243 -4	6.832 -5	1.427 -5	2.349 -6
	Shear	1.078 -4	2.611 -5	5.758 -6	1.385 -6	2.355 -7

Table 6. Values of Objective Fucntion ($D+E_{SS}$).

$P_{f,T}$	Limit State	$\gamma_{ES}=1.1$	$\gamma_{ES}=1.2$	$\gamma_{ES}=1.3$	$\gamma_{ES}=1.4$	$\gamma_{ES}=1.5$
1.0×10^{-6}	Flexure	---	---	6.459	3.659	5.302
	Shear	---	---	3.938	3.425	6.262
1.0×10^{-5}	Flexure	7.603	3.312	2.746	5.107	---
	Shear	3.299	1.416	2.782	7.073	---



Case A ($P_{f,T}$ = 1.0 x 10^{-6})



Case B ($P_{f,T}$ = 1.0 x 10^{-5})

Fig. 8. Objective Function vs. Load Factor ($D+E_{SS}$).

7.2 Dead Load, Live Load and Safe Shutdown Earthquake

If we assume that the shear wall structures are subjected to dead load, live load and earthquake during their lifetime, then the proposed load combinations for design are Eqs. 1 and 2. The design and reliability assessment of the representative shear wall structures follows the same approach as desc·ibed in Sections 6 and 7.1. The required wall thickness and reinforcement ratios are shown in Table 7. In addition, the limit state probabilities of the shear walls under these three loads in 40 years are shown in Table 8. Using these limit state probabilities, the objective function Ω can be computed for several values of γ_{ES} and $P_{f,T}$ as shown in Table 9. Figure 9 shows parabolic curves plotted through these values of the objective function. For $P_{f,T} = 1.0 \times 10^{-6}$, the optimum values of γ_{ES} are 1.366 and 1.411 for shear and flexure limit states, respectively. For $P_{f,T} = 1.0 \times 10^{-5}$, the optimum values of γ_{ES} are 1.214 and 1.267 for shear and flexure limit states, respectively.

Table 7. Required Wall Thickness and Reinforcement Ratios ($D+L+E_{SS}$).

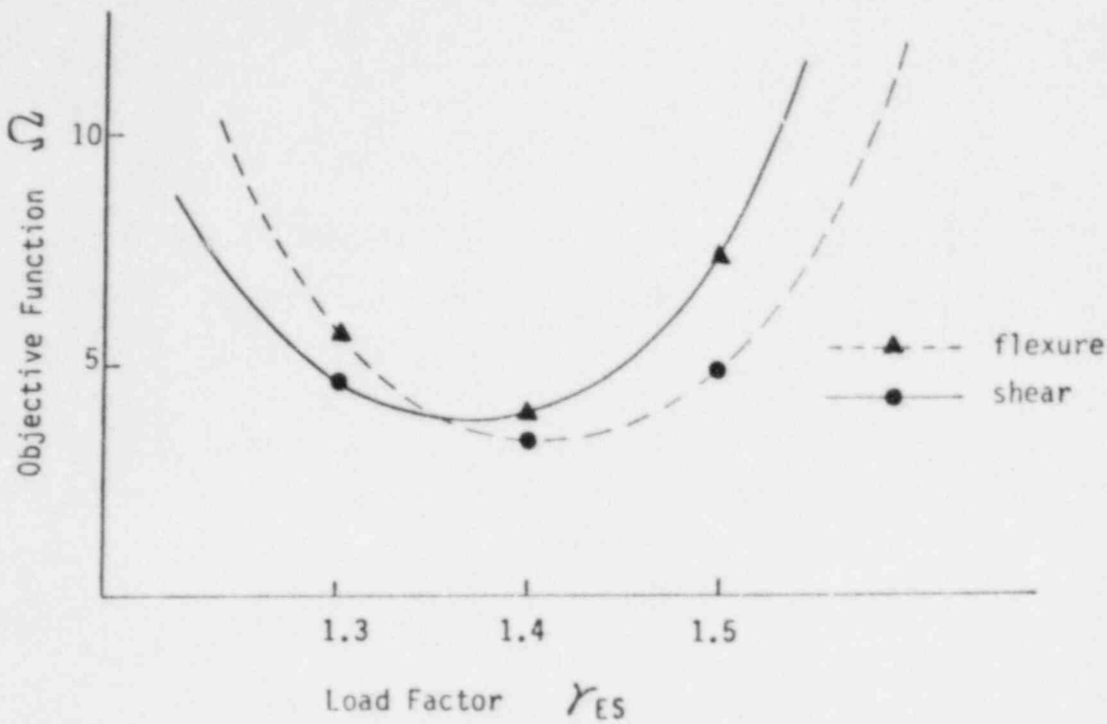
Sample	γ_{ES}	h (in)	ρ_m	ρ_h	ρ_n
1	1.1	8	0.00623	0.00148	0.00148
	1.2	8	0.00793	0.00213	0.00213
	1.3	8	0.00957	0.00278	0.00271
	1.4	9	0.00947	0.00266	0.00262
	1.5	10	0.00926	0.00257	0.00256
2	1.1	13	0.00265	0.00256	0.00256
	1.2	15	0.00284	0.00235	0.00235
	1.3	16	0.00315	0.00257	0.00256
	1.4	18	0.00331	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

Table 8. Limit State Probabilities ($D+L+E_{SS}$).

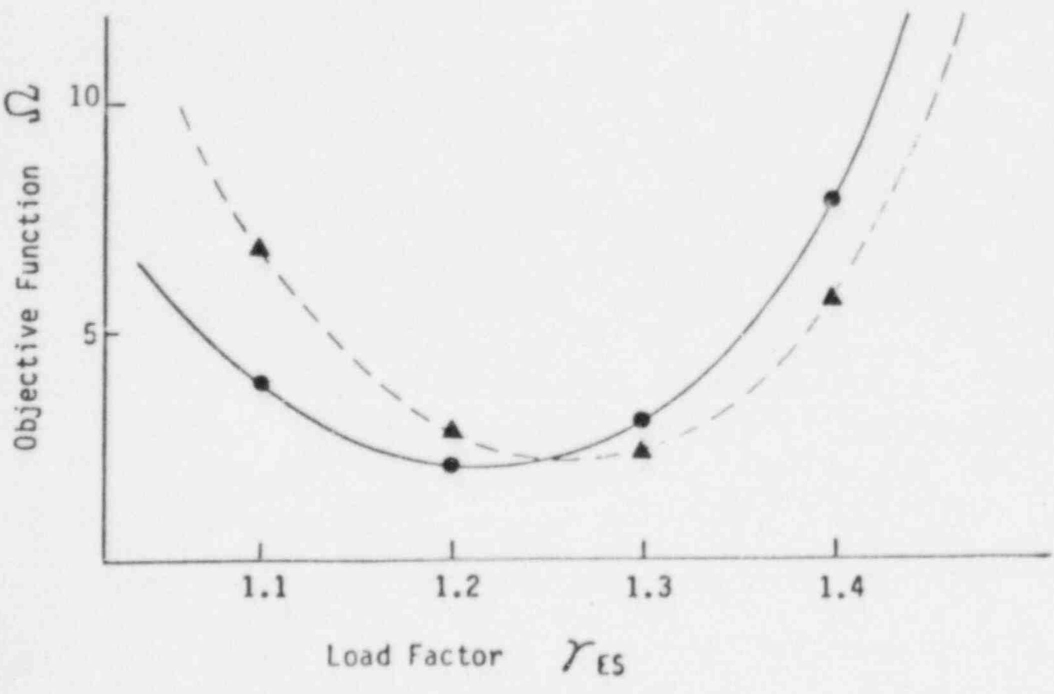
Sample	Limit State	$\gamma_{ES}=1.1$	$\gamma_{ES}=1.2$	$\gamma_{ES}=1.3$	$\gamma_{ES}=1.4$	$\gamma_{ES}=1.5$
1	Flexure	3.349 -4	1.240 -4	4.670 -5	1.315 -5	3.930 -6
	Shear	3.312 -4	1.829 -4	9.847 -5	4.249 -5	1.681 -5
2	Flexure	5.452 -5	9.453 -6	2.041 -6	2.586 -7	4.507 -8
	Shear	2.002 -5	3.087 -6	7.162 -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

Table 9. Values of the Objective Function ($D+L+E_{SS}$).

$P_{f,T}$	Limit State	$\gamma_{ES}=1.1$	$\gamma_{ES}=1.2$	$\gamma_{ES}=1.3$	$\gamma_{ES}=1.4$	$\gamma_{ES}=1.5$
1.0×10^{-6}	Flexure	---	---	5.658	3.219	4.747
	Shear	---	---	4.570	3.957	7.263
1.0×10^{-5}	Flexure	6.777	3.028	2.382	5.529	---
	Shear	3.821	2.082	3.042	7.708	---



Case A ($P_{f,T} = 1.0 \times 10^{-6}$)



Case B ($P_{f,T} = 1.0 \times 10^{-5}$)

Fig. 9. Objective Function vs. Load Factor (D+L+E_{SS}).

The optimum load factors obtained from Sections 7.1 and 7.2 are very similar and are summarized in Table 10. For $P_{f,T} = 1.0 \times 10^{-6}$ per 40 years, γ_{ES} is recommended to be 1.4, and for $P_{f,T} = 1.0 \times 10^{-5}$ per 40 years, γ_{ES} is recommended to be 1.2 . These recommended values of γ_{ES} are also shown in Table 10.

Table 10. Optimum Value of γ_{ES}.

$P_{f,T}$	Limit State	D+E	D+L+E	Recommendation
1.0×10^{-6}	Shear	1.365	1.366	1.4
	Flexure	1.413	1.411	
1.0×10^{-5}	Shear	1.207	1.214	1.2
	Flexure	1.269	1.267	

8. TENTATIVE DESIGN CRITERIA FOR SHEAR WALLS

As described in Section 7, the proposed design criteria for shear wall structures are in the LRFD format. In this study, the resistance factors and load factors for dead and live loads were preassigned to simplify the optimization. The proposed design criteria for sheer walls subjected to dead load, live load and earthquake during the service life are as follows:

$$\left.\begin{array}{l} 1.2D + 1.0L + \gamma_{ES} E_{SS} \\ 0.9D \qquad\quad - \gamma_{ES} E_{SS} \end{array}\right\} \leq \phi_i R_i \qquad (15)$$

The load factor for SSE, γ_{ES}, is 1.2, if the target limit state probability is selected as 1.0×10^{-5} per 40 years (2.5×10^{-7} per year); γ_{ES} will increase to 1.4 if $P_{f,T}$ is selected as 1.0×10^{-6} per 40 years (2.5×10^{-8} per year). The resistance factor for shear, ϕ_v, is 0.85 and the factor for compression or compression with flexure, ϕ_c, is 0.65. The determination of the nominal design values for loads and nominal resistance follows current practice.[23]

The proposed load combinations are similar in general appearance to those specified in ANSI Standard A58.1-1982.[1] The proposed load factor for earthquake in this study is 1.2 or 1.4 instead of the value of 1.5 appearing in the A58 Standard.[1] However, the definition of earthquake for designing nuclear plant structures is quite different from the design earthquake in the A58 Standard. In general, the safe shutdown earthquake specified for nuclear plant structures is much stronger than that specified for conventional structures. The design earthquake in the proposed load combination is represented by only one level of earthquake (e.g., SSE). The reason for selecting only one design earthquake in LRFD is explained in the Refs. 11 and 15.

Another difference appears in the resistance factor for shear. In this study, the recommended resistance factor for shear is 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. 6-9, 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] Thus, ϕ_v is higher than one might otherwise expect from previous probability-based design studies.[8]

9. COMPARISON OF PROPOSED CRITERIA WITH ACI-349 AND SRP 3.8.4

For this comparative study, two representative shear walls (i.e., samples 2 and 4) in Table 1 are utilized. These shear walls are assumed to be subjected to dead load and earthquake during their lifetime of 40 years.

Design by ACI Criteria

The load combinations for design of shear walls specified in the current ACI-349 code are as follows:

$$\left.\begin{aligned} 1.0\ D + 1.0\ E_{SS} \\ 1.4\ D + 1.7\ E_{O} \end{aligned}\right\} \leqslant \phi_i R_i \qquad (16)$$

where E_O is the load effect due to the operating basis earthquake (OBE). For this comparative study, it is assumed that $E_O = 1/2\ E_{SS}$. The resistance factors for shear and for compression, with or without flexure, are 0.85 and 0.70, respectively. ACI 349 specifies minimum reinforcement ratios (both horizontal and vertical) while no specification is given for wall thickness. Wall thickness of 9 and 18 inches are selected so that the required reinforcement ratios for shear are approximately the same as the minimum values specified in ACI 349. Shear walls in nuclear power plants usually have larger thicknesses due to the consideration of radiation or effects of tornado-borne missiles.

The axial force due to dead load is obtained from static analysis. For seismic analysis, the response spectrum analysis method is employed. The horizontal response spectrum is specified in the Regulatory Guide 1.60.[6] The damping ratio is taken to be 7 percent of critical for SSE and 4 percent of critical for OBE, as specified in the Regulatory Guide 1.61.[7] The force resultants due to dead load and SSE (or OBE) are combined according to the load combinations specified in Eq. 16. The required reinforcement ratios then are determined. The reinforcement ratio required for flexure is less than the minimum specified in ACI 349. Hence the minimum value (0.0015) is used. The wall thickness$and required reinforcement ratios are tabulated in Table 11.

Design by SRP 3.8.4

For shear walls subjected to dead load and earthquaoe, the load combinations specified in the NRC Standard Reveiw Plan (SRP), Section 3.8.4 are as follows:

$$\left.\begin{array}{l} 1.0\ D + 1.0\ E_{ss} \quad (17a) \\ 1.2\ D + 1.9\ E_o \quad (17b) \end{array}\right\} \leq \phi_i R_i$$

Design requirements are the same as in ACI 349 except the load factors in Eq. 17b. The two representative shear walls designed according to SRP 3.8.4 are also listed in Table 11.

Table 11. Shear Walls Designed With ACI and Proposed Criteria.

Sample	Design Criteria		Thickness (in)	ρ_m	ρ_n	ρ_h
2	ACI		9	0.00150	0.00263	0.00264
	SRP 3.8.4		9	0.00150	0.00404	0.00404
	Proposed	$\gamma_{ES} = 1.2$	12	0.00230	0.00252	0.00252
		$\gamma_{ES} = 1.4$	15	0.00277	0.00236	0.00236
4	ACI		18	0.00150	0.00271	0.00271
	SRP 3.8.4		18	0.00150	0.00388	0.00388
	Proposed	$\gamma_{ES} = 1.2$	24	0.00219	0.00245	0.00245
		$\gamma_{ES} = 1.4$	30	0.00249	0.00245	0.00245

Design by Proposed Criteria

The proposed criteria for the design of sheer wall structures are summarized in Section 8. Using Eq. 15 and the design procedure described in Section 6, the design of the two sample shear walls is performed. The required wall thickness and reinforcement ratios are also tabulated in Table 11. The wall thickness of the shear walls is determined by Eq. 13. From Table 11, it can be seen that the wall thickness and the reinforcement ratio for flexure required by the proposed criteria are larger than those required by ACI 349 or SRP 3.8.4, while the required reinforcement ratios for shear are almost the same. This implies that the proposed design criteria are more stringent than those specified in the ACI 349 or SRP 3.8.4.

Comparison of Reliability Analysis Results

The reliability assessments of the shear walls shown in Table 11 are carried out using the probabilistic descriptions of loads and material strengths described in Section 4. Using only the required shear reinforcement, the limit state probabilities are evaluated and shown in Table 12 for the shear limit state. On the basis of the data used in this study, the limit state probabilities of the shear walls designed according to ACI-349 or SRP 3.8.4 are approximately 1.0×10^{-4} per 40 years (2.5×10^{-6} per year). Since the proposed criteria are based on $P_{f,T} = 1.0 \times 10^{-5}$ per 40 years, or 1.0×10^{-6} per 40 years, the proposed criteria are more stringent.

Table 12. Reliability Assessments of Shear Walls.

Design Criteria		Limit State	Sample 2	Sample 4
ACI		Shear	1.644 -4	3.614 -4
SRP 3.8.4		Shear	4.943 -5	1.311 -4
Proposed	$\gamma_{ES} = 1.2$	Shear	4.800 -6	2.611 -5
	$\gamma_{ES} = 1.4$	Shear	1.453 -6	1.385 -6

10. CONCLUDING REMARKS

This report describes the development of probability-based criteria for the design of shear wall structures. The proposed design criteria are in the load and resistance factor design (LRFD) format. The load and resistance factors are generally determined on the basis of limit states and a target limit state probability. For this study, two limit states are considered. The flexure limit state is defined according to conventional ultimate strength analysis and the shear limit state is established from experimental data on low-rise walls. At present, three loads, i.e., dead load, live load and in-plane earthquake are considered to act on the shear walls. The proposed load combinations are summarized in Section 8 for the target limit state probability of 1.0×10^{-5} or 1.0×10^{-6} per 40 years. The proposed criteria are risk-consistent and have a well-established rationale. Of course, the regulatory authority and/or code committee must make a decision on the reliability level to be specified in the design criteria.

The proposed criteria are for the design of shear walls. The criteria should be verified with regard to their applicability to other types of structures. Furthermore, the shear wall structures may be subjected to tornado-borne missiles and other loading conditions. The proposed design criteria need to be reviewed before being implemented for such loading conditions.

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13. ABSTRACT (200 words or less)

This report describes the development of probability-based load combination criteria for the design of reinforced concrete shear wall structures subjected to dead load, live load and earthquake. The proposed design criteria are in the load and resistance factor design (LRFD) format. The load and resistance factors are determined for flexure and shear limit states and target limit state probabilities. The flexure limit state is defined according to the ACI ultimate strength formula. The shear limit state is established from experimental results.

In order to test whether the proposed cirteria meet the reliability-based performance objectives, four representative structures are selected using a Latin hypercube sampling technique. These representative structures are designed using trial load and resistance factors. Then, a reliability analysis method is employed to assess their reliabilities. An objective function is defined and a minimization technique is developed to find theoptimum load factors. In this study, the resistance factors for shear and flexure, and load factors for dead and live loads are preassigned to simplify the minimization. The load factor for SSE is determined for the target limit state probability of 1.0×10^{-6} or 1.0×10^{-5} with a lifetime of 40 years.

14. DOCUMENT ANALYSIS – a. KEYWORDS/DESCRIPTORS: Concrete; Design Criteria; Earthquake; limit states; Loads; Load Factors; Probability theory; Reliability analysis; Resistance Factors; Shear Wall Structures

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