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DESIGN AND RELIABILITY ASSESSMENT OF LOW-RISE BUILDINGS

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SUMMARY

This paper examines the structural integrity of low-rise buildings located in the New York City area and designed to resist the lateral forces generated by wind and earthquake. In the design and analysis of these buildings, appropriate provisions of ANSI A58.1-1982 (Ref. 1), ACI Code 318-83 (Ref. 2) and ATC 3-06 (Ref. 3) are used. The structural integrity is measured in terms of the limit state probability. The limit state probability values are computed under various seismic and wind design conditions. The results indicate that seismic hazard can by no means be disregarded for the type of building structure considered in this study.

INTRODUCTION

Conventional structures, in particular, low-rise buildings are usually designed according to provisions specified in building codes and standards such as the Uniform Building Code (UBC) (Ref. 4), Standard Building Code (SBC) (Ref. 5) and American National Standard ANSI A58.1 (Ref. 1). The code provisions are intended to achieve the satisfactory performance of buildings under loads imposed by the users or nature such as wind or earthquake. However, building codes usually employ simplified formulas in the provisions in order to facilitate the design process. For example, the equivalent static design forces are stipulated in building codes to represent the wind or seismic forces which are dynamic and random in nature. The United States are divided into several seismic zones to represent differing degrees of seismic hazards in these zones, and a typical peak ground acceleration (PGA) value is assigned to each zone. Furthermore, some building codes, e.g., New York City building laws, have provisions only for wind design without any provisions for aseismic design. Concern has been raised as to whether or not a building designed only for a wind load is safe under potential seismic hazards. In this paper, the design and reliability assessment of a five-story office building was carried out to examine this issue.

DESIGN OF SHEAR WALL STRUCTURE

The building selected for this study is a five-story office building supposedly located in New York City. Figure 1 shows a typical floor plan and section of the building. A reinforced concrete frame system is used to resist vertical loads, i.e., dead and live loads. The two reinforced concrete shear walls in the north-south direction as shown in Fig. 1 are used to resist all the lateral forces due to wind or earthquake loads in that direction. This paper focuses on the design and reliability assessment of these two shear walls. Four types of loads, i.e., dead, live, wind and earthquake loads are considered to act on the building. The values of these loads used in the design, i.e., the design loads, are specified according to the American National Standard, Minimum Design Loads for Buildings and Other Structures (Ref. 1). The shear wall is designed according to ACI Code 318-83 (Ref. 2). The details of the design of the five-story office building are shown in (Ref. 6) and summarized in Table 1 where E-2-S_1, for example, means a Zone 2 design under soil condition R_1.

PROBABILISTIC CHARACTERISTICS OF STRUCTURAL CAPACITY AND LOADS

The nominal structural capacity (resistance) and design loads are specified in building codes by simplified formulas. In reality, both structural capacity and loads are random in nature and also involve modeling as well as parameter uncertainty. In the present study, it is assumed that key parameters of the design, structural capacity and loads can be treated as lognormally-distributed random variables whose variability represents a combination of randomness and uncertainty. While other limit states such as deformation ductility and/or absorbed energy may have to be considered, the limit state considered here is that related to the base shear, primarily for simplicity of analysis.

A log-normal variable X can be described by its median value $\tilde{X}$ and standard deviation β_X of ℓn X. If the coefficient of variation (COV) is not very large, say, less than about 0.4, β_X is approximately equal to its COV value.

The structural capacity is affected by the variations of material strength, structural geometry and workmanship. Ellingwood and Hwang (Ref. 7) estimate that the median shear capacity of a shear wall $\tilde{Q}^*_R$ is about 1.70 times the nominal capacity V_n and the COV is 0.18. On the basis of these findings, the capacity of the shear wall is summarized in Table 2.

The probabilistic model for the wind pressure P* is assumed as:

$$P^* = 0.00256\ C^*_p K^*_z G^*_h (V^*)^2 \qquad (1)$$

where V* is the wind speed at the reference height of 10 m. From an analysis of the observation data (1947-1977) at LaGuardia Airport in New York City, Simiu et al. (Ref. 8) estimate that the annual extreme mean wind speed follows a Type I extreme-value distribution with expected value equal to 50.25 mph and standard deviation equal to 7.23 mph (COV = 0.14). In this study, it is assumed that the median value $\tilde{V}^*$ is the same as the mean, i.e., 50.25 mph and β_V = 0.14.

The statistics of C^*_p, K^*_z and G^*_h are described by Ellingwood et al. (Ref. 9). The median values of these factors, $\tilde{C}^*_p$, $\tilde{K}^*_z$ and $\tilde{G}^*_h$ are taken to be 1.0 times the design values. Thus, $\tilde{C}^*_p$ = 1.3, $\tilde{G}^*_h$ = 1.36 and $\tilde{K}^*_z$ varies with height. In addition, β_{cp} = 0.12, β_{kz} = 0.16 and β_{Gh} = 0.11 are adopted for the study following Ellingwood et al. (Ref. 9).

The base shear Q^*_W due to wind is a product of the wind pressure and the exposed area of the building. The dimensions of the building are assumed to be deterministic. Thus, the variation of the base shear is the same as that of the wind pressure. The median value and COV of the base shear are, respectively, $\tilde{Q}^*_W$ = 28.7 kips and β_{QW} = 0.36 according to the computations made by Hwang et al. (Ref. 6).

The total seismic base shear Q^*_{ET} is determined by the following expression in ATC 3-06 (Ref. 3).

$$Q^*_{ET} = \frac{1.2\ S^*W^*}{R^*(T^*)^{2/3}} A^* \tag{2}$$

In Eq. 2, A^* is the annual extreme peak ground acceleration (PGA). The annual extreme peak ground acceleration is asumed to distribute in accordance with the Type II extreme-value distribution (Ref. 9):

$$F_{A^*}(a) = \exp[-(\frac{a}{\mu})^{-\alpha}] \tag{3}$$

The parameters μ and α are assumed to be $\mu = 0.0135g$ and $\alpha = 3.14$ for the New York area on the basis of a study by Hwang et al. (Ref. 10). Equation 3 gives a COV of A^* equal to 0.626 and $\tilde{A}^* = 0.0152$. In this study, A^* is assumed to be log-normally distributed with the same median $\tilde{A}^* = 0.0152$ and $\beta_A = 0.575$ corresponding to COV = 0.626. The seismic hazard curve is obtained by plotting $1 - F_{A^*}(a)$ as a function of a. W^* is the weight of the structure. Ellingwood et al. (Ref. 9) recommended that β_W be 0.10 and the median of $\tilde{W}^*$ be 1.05 times the design value. $1.2/T^{*2/3}$ is a factor for linear dynamic response amplification. Based on the data collected by Haviland (Ref. 11), the median of the period $\tilde{T}^*$ is taken to be 0.91 times the computed value and β_T is 0.34. R^* is the response modification factor. The median value $\tilde{R}^*$ is assumed to be 7.0 and β_R is 0.4. Finally, the median $\tilde{S}^*$ of the soil factor S^* is taken to be the same as the design value, which depends on the soil type. β_S is assumed to be 0.3 for all soil conditions.

From Eq. 2 and the property of the lognormal variable, the median of the total seismic base shear $\tilde{Q}^*_{ET}$ is

$$\tilde{Q}^*_{ET} = \frac{1.2\ \tilde{S}^*\tilde{W}^*}{\tilde{R}^*(\tilde{T}^*)^{2/3}} \tilde{A}^* \tag{4}$$

For each shear wall, the median of the seismic base shear, $\tilde{Q}^*_E$, is equal to one-half of $\tilde{Q}^*_{ET}$. For soil types S_1, S_2 and S_3, $\tilde{Q}^*_E$ is 20.2 kips, 24.2 kips and 30.3 kips, respectively. Furthermore, β_{QE} and β_{QET} are the same and, under the assumed independence of the random variables involved, can be determined by:

$$\beta_{QE} = \beta_{QET} = [\beta^2_S + \beta^2_W + \beta^2_R + (\frac{2}{3})^2\beta^2_T + \beta^2_A]^{1/2} \tag{5}$$

Thus, β_{QE} is equal to 0.80.

SAFETY EVALUATION

The limit state probability is used as a measure of the integrity of the shear wall. The limit state probability under earthquake load $P_{f,E}$ can be defined as

$$P_{f,E} = P_r(\frac{Q^*_R}{Q^*_E} \leq 1) = \Phi[\frac{-\ln(\tilde{Q}^*_R/\tilde{Q}^*_E)}{(\beta^2_{QR} + \beta^2_{QE})^{1/2}}] \tag{6}$$

where $\Phi(\cdot)$ is the standardized normal distribution function. The expression for the limit state probability under wind load, $P_{f,W}$ can be obtained by replacing the subscript E with W in Eq. 6. Furthermore, disregarding the joint occurrence probability of earthquake and severe wind, the total limit state probability P_f is approximated by $P_f = P_{f,E} + P_{f,W}$, and summarized in Table 3.

DISCUSSION AND CONCLUSIONS

The limit state probability values summarized in Table 3 may be used primarily for comparative purposes. For example, it is clear from Table 2 that for the type of building and limit state considered, the seismic hazard appears to be more serious than the hazard imposed by wind, even when a zone 2 design is implemented. This conclusion obviously depends on the accuracy of the various assumptions made in the present study. Some of the more important factors that influence the probability values and therefore require further study are discussed in (Ref. 6).

This work presents the results of a preliminary study where the integrity of a low-rise shear-wall type building is evaluated in terms of the limit state probability. At the same time, the paper demonstrates how knowledge of different scientific and engineering disciplines can improve the various underlying assumptions in order to arrive at a more reliable safety evaluation. Similar results were also obtained for flat-slab structures by Hwang et al. (Ref. 6).

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Table 1 Design of Shear Wall

Case	Loading	Wall Thickness (in)	Horizontal Reinforcement at bottom	V_c (kips)	V_s (kips)	$V_n = V_c + V_s$	ϕV_n	$1.43 Q_E$ or $1.3 Q_W$
1	$E - 2 - S_1$	5	$\#3@7in$	121.8	139.8	261.6	222.4	219.4
2	$E - 2 - S_2$	5	$\#3@5in$	121.8	195.7	317.5	269.9	263.4
3	$E - 2 - S_3$	5	$\#4@7in$	121.8	254.2	376.0	319.6	307.2
4	Wind	5	$\#3@8in^*$	121.8	122.3	244.1	207.5	105.4

* Minimum reinforcement required by ACI 318-83.

Table 2 Distributionn of Shear Wall Resistance

Case	Wall Thickness (in)	Horizontal Reinforcement at bottom	V_n (kips)	$\overline{Q}_R = 1.7V_n$ (kips)	β_{Q_R}	Distribution
1	5	$\#3@7in$	261.6	444.7	0.18	Lognormal
2	5	$\#3@5in$	317.5	539.8		
3	5	$\#4@7in$	376.0	639.2		
4	5	$\#3@8in$	244.1	415.0		

Table 3 Annual Limit State Probability

Case	$P_{f,E}$	$P_{f,W}$	P_f
1	8.2×10^{-5}	3.9×10^{-12}	8.2×10^{-5}
2	7.5×10^{-5}	1.4×10^{-13}	7.5×10^{-5}
3	1.0×10^{-4}	5.0×10^{-15}	1.0×10^{-4}
4		1.1×10^{-11}	

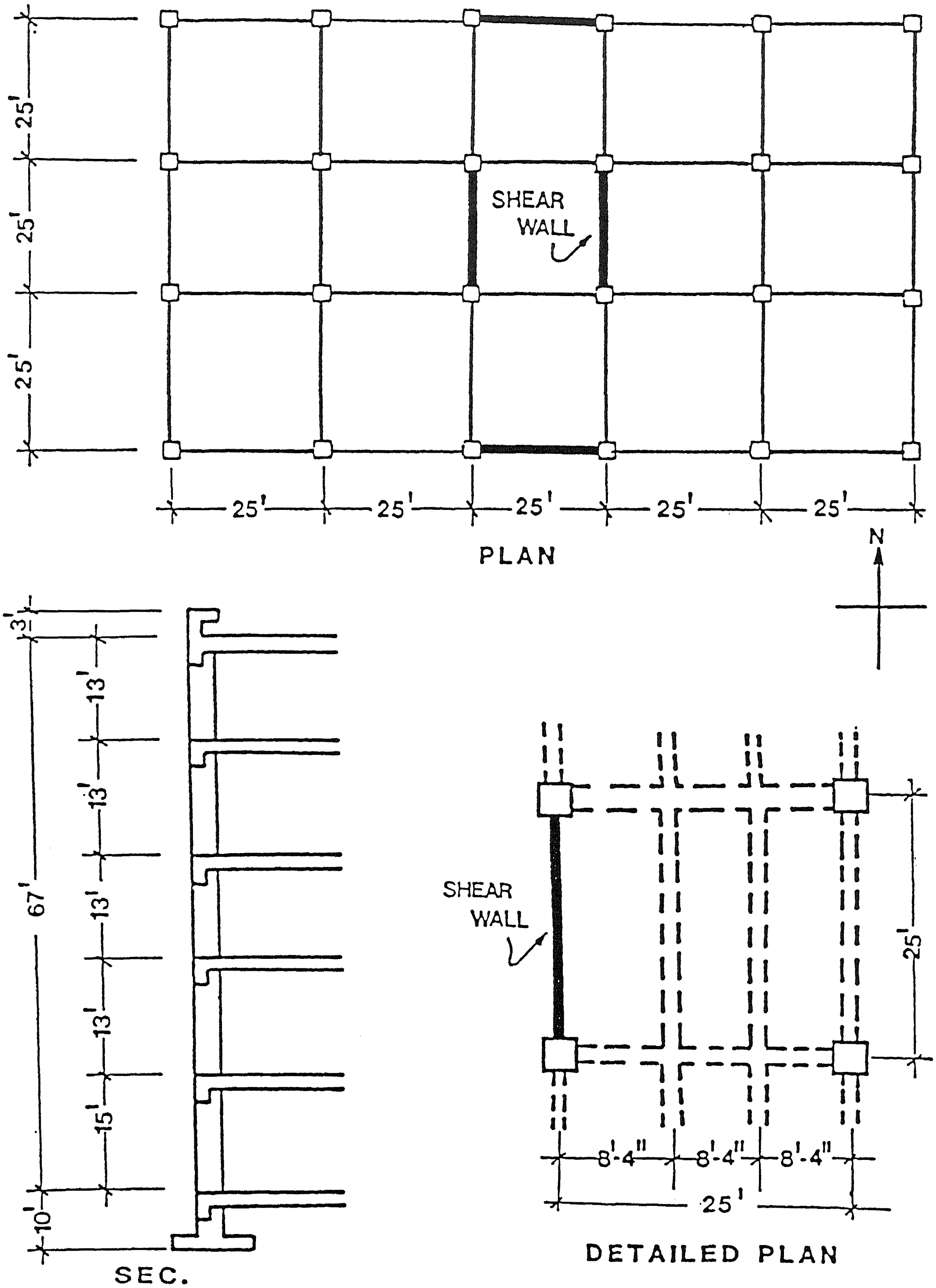

Fig. 1 Plan and Cross-Section of Building