EXTREME VALUE ANALYSIS OF STRUCTURAL LOADS IN THE KINGDOM OF SAUDI ARABIA

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ABSTRACT

This paper presents a probabilistic analysis of structural loads in Saudi Arabia. The main objective is to develop load models and estimate their statistical characteristics for reliability and risk analysis associated with code development and computation of safety factors. These loads include: dead, live, wind and earthquake loads. The results indicate that the dead load is normally distributed with mean-to-nominal ratio of 1.15 and coefficient of variation of 10 percent. Extreme wind load is well presented by Type I extreme distribution with mean-to-nominal ratio of 0.75 and coefficient of variation of 60 percent. Extreme Earthquake load is well presented by Type II extreme distribution with mean-to-nominal ratio of 0.70 and coefficient of variation of 137 percent. Statistics for sustained and extreme live loads are adopted from previous international studies. Results are compared with those obtained in similar international studies.

Keywords: Extreme value analysis, design loads, dead loads, wind loads, earthquake loads, reliability, safety factors, probabilistic approaches, and code calibration.

1. INTRODUCTION

Modern structural design codes have adopted reliability-based approaches for code calibration and determination of design safety factors. This new approach provides the necessary tools to compute load and resistance factors such that structural systems attain specified target reliability levels for different limit states. Structural reliability is defined as the probability of the structure not attaining any of its ultimate limit states during its service lifetime. Employing reliability tools, the values of safety factors can be computed such that consistent levels of structural reliability are obtained under various loading conditions.

In Appendix C of the ACI 318M-95 code [1] , a set of alternative load and strength reduction factors are adopted as an alternative safety checking approach. The set of load combinations and associated load factors are as follows,

$$\begin{aligned} &\mathbf{1.4\,D} \\ &\mathbf{1.2\,D + 1.6\,L} \\ &\mathbf{1.2\,D + 0.5\,L + 1.3\,W} \\ &\mathbf{1.2\,D + 0.5\,L + 1.5\,E} \end{aligned} \qquad \mathbf{(1)}$$

where **D, L, W,** and **E** are the effects of dead, live, wind and earthquake loads, respectively. The factor 0.5 of live load shall be equal to 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 4.8 kN/m^2. The corresponding strength reduction factors for flexural, shear and compression limit states are 0.8, 0.75 and 0.65, respectively. These factors were based on a comprehensive reliability-based study [2] and provide more consistent levels of structural reliability than the current load factors.

Loads on structures are the main source of uncertainty in the design process. Load studies, in general, serve two main objectives: (1) estimation of their statistical characteristics and stochastic models and (2) estimation of their nominal design values which are usually computed at an acceptable level of risk during the service lifetime (typically = 50 years) of the structure. The level of acceptable risk is not the same for the different loads. For example, the nominal wind load is computed such that the annual probability of being exceeded is 2 percent, i.e. the wind load return period is 50 years, while the nominal earthquake load is computed such that the probability of being exceeded is 10 percent during the service lifetime of the structure, i.e. the return period is 475 years.

The main step towards code calibration and determination of load and resistance safety factors is the development of the representative probabilistic models for individual load and strength variables. Load stochastic models and their characteristics depend on several factors. For example, variations in dead load depend on material properties and degree of precision in the construction process.

Variation of live loads depends on type of occupation and other parameters related to the structure such as floor size and area usage. Wind and earthquake loads are environmental loads which are associated with high degrees of variability. Wind load depends on wind speed, type of terrain surrounding the structure, wind fluctuation, height above ground, and exposed area. Earthquake load depends on magnitude of ground acceleration, distance from earthquake epicenter, and structural characteristics.

The variability of the load effect, **Q**, depends on three factors: the natural variability of structural loads, **A**, the uncertainties of load models which transform the actual spatially and temporally varying load into a statistically equivalent uniformly distributed load, **B**, and the uncertainties arising from the analysis which transform the load to a load effect, **C**. Since these factors are mutually independent random variables, the relation between the load effect, **Q**, and the structural load can be expressed in a multiplication form as [2],

$$\mathbf{Q = A * B * C} \qquad \mathbf{(2)}$$

where **A**, **B** and **C** are random variables. Thus the mean value, $\overline{\mathbf{Q}}$, and coefficient of variation, $\mathbf{V_Q}$, of the load effect are evaluated as follows [2]:

$$\overline{\mathbf{Q}} = \overline{\mathbf{A}} * \overline{\mathbf{B}} * \overline{\mathbf{C}} \qquad \mathbf{(3)}$$

$$\mathbf{V_Q} = \sqrt{\mathbf{V^{2,A} + V^{2,B} + V^{2,C}}} \qquad \mathbf{(4)}$$

where $\overline{\mathbf{A}}$, $\overline{\mathbf{B}}$, and $\overline{\mathbf{C}}$ are the mean values of variables **A**, **B** and **C** and $\mathbf{V_A}$, $\mathbf{V_B}$ and $\mathbf{V_C}$ are their respective coefficients of variation.

Several studies were conducted towards the development of a national design code for reinforced concrete structures in Saudi Arabia and determination of its appropriate safety factors. The framework, work plan for code development and the statistical characteristics of basic strength variables were presented in references [3,4,5]. The statistical characteristics of reinforced concrete members at the various limit states (flexure, shear and compression) were presented in reference [6].

This paper presents a probabilistic analysis of structural loads in Saudi Arabia. The main objective of this study is to develop loads models and estimate their statistical characteristics for reliability and risk analysis associated with code development and computation of load and resistance safety factors for the proposed Saudi design code. Results are compared with those obtained in similar international studies.

2. DEAD LOAD

The dead load comprises the weights of the structural elements and non structural items such as permanent equipment, partitions, installations, roofing and floor covering. The uncertainty in the weights of non-structural items is the main source of the dead load variability [2].

2.1 Weights of Non-Structural Items

This study included a survey on the variability in dead load components. Data was collected from a total of thirty three locations. The main purpose was to estimate the basic statistics of the dead weight of non-structural items such as blocks, tiles, marbles, and ceramic. The survey also included measurements of thicknesses of cement plaster, cement mortar under tiles, and the sand layer under tiles and marble. Table 1 presents basic statistics of weights of the surveyed non-structural items.

2.2 Weights of Structural Members

The variability in weights of structural members arises from the variations of sectional dimensions and concrete unit weight, γ_C. Results from site measurements on structural elements indicated that the variability in sectional dimensions is very small [2,3]. Variability in concrete density is investigated. Fig. 1 shows that variability in concrete density is well presented by normal distribution with mean of 22.6 kN/m^3 and coefficient of variation of 2.32 percent. This is comparable to the mean value of 22.7 kN/m^3 with coefficient of variation of 3 percent reported in Ref. [2] .

2.3 Variability of Total Dead Load

Typical design problems are carried out taking into account various structural and non-structural elements to estimate the dead load mean-to-nominal, λ_D, and coefficient of variation, V_D . Based of extensive site measurements and field testing on weights of non structural elements, see Table 1, the mean-to-nominal ratio of parameter **A**, in Eq. (2), is found to be 1.15 with a coefficient of variation of 6.2 percent. The other two variables **B** and **C** are usually assumed to have mean-to-nominal ratios = 1.0 and coefficient of variations = 5 percent. From Eqs. (3) and (4) , one can obtain that approximately λ_D = 1.15 and V_D = 10 percent which means that the average value of field testing and site measurements of dead loads are 15% more than the design value. Since dead load is a summation of many random structural and non-structural weights and according to the central limit theory [7] the total dead load can be assumed to be normally distributed.

For comparison, Ellingwood, et. al.[2] assumed normal distribution for dead load with estimated λ_D = 1.05 and V_D = 10 percent. The value of λ_D in the Kingdom is higher than that reported in the literature. This means that, in general, dead load is under estimated during the design process and contractors, during construction, do not closely follow the design requirements specially in case of non-structural items.

3. LIVE LOADS

The total live load on floor area consists of a sustained component which results from daily use of the structure and remains relatively constant with a particular occupancy, L_S , and an extraordinary component, L_E , which arises from infrequent clustering of people or activities such as remodeling. World wide, limited number of load surveys were conducted to estimate the statistics of the live load components [2,8,9]. In the Kingdom, it is expected that the daily and extraordinary uses of various types of structures are similar to those in other countries, therefore, the statistical characteristics of sustained and total live loads are assumed to be similar to those reported in the literature.

3.1 Sustained Live Load

The sustained live load intensity decreases with increasing the influence area of the structural member. The statistical characteristics of sustained live load are also affected by the influence area. For an influence area = 20 m^2 , the mean-to-nominal ratio of L_S is between 0.15 and 0.24 [2]. The coefficient of variation is between 60 and 90 percent which decreases by increasing the influence area [2]. The sustained live load proved to be fitted by a Gamma probability distribution [8]. The coefficients of variation of factors **B** and **C** in case of sustained live load are usually assumed to be 10 and 5 percents, respectively [2].

3.2 Extraordinary Live Load

Several live load surveys were conducted to estimate the statistics of extraordinary live loads . An extensive study was conducted by Choi [9]. The study covered 1989 extraordinary events including crowding and furniture-stacking events. Results of the study were analyzed in relation to other basic data on buildings such as floor size, area usage, and occupancy type [9] .

3.3 Total Live Load

The maximum lifetime live load is usually modeled by the Type I extreme function. For an influence area = 20 m^2, the mean-to-nominal ratio is between 1.10 and 1.40 with a coefficient of variation between 15 and 25 percent [2]. The coefficients of variation of factors **B** and **C** were assumed to be 20 and 5 percents [2]. The typical value of λ_L for the 50-year live load = 1.00 with $\mathbf{V_L}$ = 25 percent [2] . The corresponding values of u and α for the Type I extreme distribution are 0.98 and 0.21 respectively.

4. WIND LOADS

Wind loads are crucial for the design of some structures such as tall buildings, towers, radar and communication antennas. The data of the largest annual wind speeds, from the Saudi Meteorological and Environmental Protection Agency (MEPA), include records continuing over periods of ten to forty years measured at thirty stations distributed all over the country. Twenty of these stations have records over a continuous duration of fifteen years or more which is desirable for the probabilistic analysis.

4.1 Wind Speed Models

The commonly used model in extreme value analysis of maximum annual wind speeds is the Type I extreme distribution, $\mathbf{F_V(\upsilon)}$, which is described as [7,10],

$\mathbf{F_V(\upsilon) = exp [- exp (- (}$ خطأ! $\mathbf{)]}$ **(5)**

where α and u are the scale and location parameters to be estimated from the observed data at each station [10]. α is essentially a measure of scatter in the maximum annual wind speed and u represents its most probable (or model) value. The wind speed at a specified value of $\mathbf{F_V(\upsilon)}$ can be expressed as follows,

$$\upsilon(F) = u + \alpha \; x(F_V(\upsilon)) \qquad (6)$$

where $\mathbf{x\ (F_V(\upsilon)) = -\ ln\ (\ -\ ln\ (\ F_V(\upsilon)\ )\ )}$. This transformation is essential for performing the standard linear regression analysis and estimating the parameters of the distribution function. The mean and standard deviations can be estimated from u and α as follows,

$$\overline{\mathbf{V}} = \mathbf{u} + \mathbf{0.5772}\,\alpha \qquad \mathbf{(7)}$$

$$\sigma = \text{خطأ!}\alpha \qquad \mathbf{(8)}$$

Basic design wind (BDW) speed is defined as the wind speed measured at a standard height of 10 meters above ground in open country with 50 year mean recurrence interval i.e. with 2 percent annual probability of being exceeded [11]. The BDW speeds can be estimated by the extreme value analysis of wind speed records.

The extreme value analysis of wind speed was conducted for the Kingdom and wind speed models were developed for twenty cities [12]. The BDW speeds were also evaluated and the isotach map for design wind speeds in the Kingdom was developed in Ref. [12].

In this study, three cities distributed over the regions of the Kingdom were selected to perform the 50-year extreme wind speed and wind load analysis. The maximum annual wind speeds for these cities are best fitted by Type I extreme distributions with parameters **u** and α as listed in Table 2. The mean values and coefficients of variation of maximum annual wind speeds, $\overline{\mathbf{V}}$ and $\mathbf{V_\upsilon}$, are listed in Table 2.

Since the annual extreme distributions are Type I, the 50-year maximum wind speed also follows the Type I extreme distributions with mean, $\overline{\mathbf{V}}_{\mathbf{50}}$, and coefficient of variation, $\mathbf{V_{\upsilon 50}}$, which are related to the annual statistics as follows [2],

$$\overline{\mathbf{V}}_{\mathbf{50}} = \text{خطأ!} \qquad \mathbf{(9)}$$

$$\overline{\mathbf{V}}_{\mathbf{50}} \,.\, \mathbf{V_{\upsilon 50}} = \overline{\mathbf{V}} \,.\, \mathbf{V_\upsilon} \qquad \mathbf{(10)}$$

The values of $\overline{\mathbf{V}}_{\mathbf{50}}$ and $\mathbf{V_{\upsilon 50}}$ and their corresponding model parameters $\mathbf{u_{50}}$ and $\alpha_{\mathbf{50}}$ are also listed in Table 2. Eq. 10 means that the scatter of maximum annual wind speed is equal to that of 50 year extreme wind speed i.e. $\sigma_V = \sigma_{V50}$.

4.2 Wind Load Models

According to ASCE 7-93 [10], the velocity pressure, q_Z, at height z is computed as follows,

$$\mathbf{q_Z = 0.00256\, K_Z\, (I\, \upsilon_n)^2} \quad \text{(lb/in}^2\text{)} \qquad \textbf{(11)}$$

where 0.00256 reflects the air mass density for the standard atmosphere, $\mathbf{K_Z}$ accounts for exposure type and height variation of the pressure coefficient, **I** is the importance factor and υ_n is the BDW speed (MPH). The design wind pressure, **P**, can be computed as follows,

$$\mathbf{P = q_z\, G_h\, C_p} \quad \text{(lb/in}^2\text{)} \qquad \textbf{(12)}$$

where the gust factor, $\mathbf{G_h}$, incorporates the dynamical characteristics of the structure, like natural period and damping, and the turbulence characteristics of the wind. External pressure coefficient, $\mathbf{C_p}$, reflects the geometry of the structure and fluid mechanics of wind flow around the structure[11].

The pressure coefficient, $\mathbf{C_p}$, gust factor, $\mathbf{G_h}$, and exposure coefficient, $\mathbf{K_Z}$, are assumed to be normally distributed with mean-to-nominal ratios, λ = 1 and **V** = 12, 11 and 16 percents, respectively [2]. A reduction factor of 0.85 is included in the simulation process to account for the reduced probability that the maximum wind speed will occur in a direction most unfavorable to the response of building. The wind load W is computed by multiplying the wind pressure (P) by the tributary area (A) at the structural joint. The distribution functions of normalized wind load **W/Wn** in the selected three cities were developed by employing Monte-Carlo simulation. Results are plotted on Type I extreme distribution scale as shown in Fig. 2. Regression analysis was performed between normalized wind load and the parameter $\mathbf{x = -\ln(-\ln(F_V(\upsilon)))}$ over the range of the distribution above its 90th percentile, i.e. $\mathbf{x} \geq 2.3$, as shown in Fig. 2.

Results, listed in Table 3, indicate that distribution functions of normalized wind loads **W/Wn** in the three cities are very similar. It is found that the best fitted model is the Type I extreme distribution. The representative values of **u** and α are 0.55 and 0.35, respectively. The representative values of λ_w and $\mathbf{V_w}$ are 0.75 and 60 percent, respectively. For comparison, Ellingwood, et. al.[2] reported that wind load has Type I extreme distribution, with **u** = 0.65 and α = 0.22,with λ_w and $\mathbf{V_w}$ equal to 0.78 and 37 percent.

5. EARTHQUAKE LOADS

The hazard of earthquake loads is described in terms of the 50-year maximum peak ground acceleration. Recently, a comprehensive study to evaluate the seismic hazard in Saudi Arabia was conducted under the KACST Project No. AR-9-31 [13]. The study included the following steps: (1) collection of data on the seismicity and seismotectonics of the Arabian peninsula and its surrounding regions, (2) identification of seismic sources, (3) treatment of data at each of these sources, (4) identification of source characteristic including: source recurrence relationship, and source maximum magnitude, (5) evaluation of at sites peak ground accelerations, **A**, using appropriate attenuation functions and drawing the at site hazard curves, (6) drawing of the iso-acceleration map for 10 percent probability of being exceeded in 50 years, and finally, (7) drawing of the seismic zonation map. The study results were published in Ref. [14].

Based on iso-acceleration map for 10 percent probability of being exceeded in 50 years, four zones were identified. Following the Uniform Building Code (UBC 1991) [15] the seismic zones were designated as 0, 1, 2A and 2B. The corresponding seismic zone numbers **Z** are 0.05, 0.075, 0.15 and 0.2 which represent the nominal values of the normalized peak ground accelerations.

5.1 Distribution Function of Peak Ground Acceleration

At site hazard curves provide the probability of exceedence, $(1 - F_A(a))$, of a specified value of the peak ground acceleration, **A** , over a specified duration period. These probabilities are calculated for zones 2A and 2B in the Kingdom, and the results are listed in Table 4. Regression analysis is performed between the natural logarithm of **A** and the parameter $x = -\ln(-\ln(F_A(a)))$ over the range of the distribution above its 90th percentile, i.e. $x \geq 2.3$, as shown in Fig. 3. Results indicate that extreme peak ground accelerations for zones 2A and 2B in the Kingdom are best fitted by Type II extreme distribution function which is expressed as,

$$F_A\,(a) = \exp\left[- \left(\text{خطأ!} \right) \text{خطأ!} \right] \qquad (13)$$

where the parameters ω and γ are the model constants and can be estimated from the observed seismic data. These parameters are related to those of Type I extreme model as follows,

$$\omega = e^{u} \quad \text{and} \quad \gamma = \text{خطأ!} \qquad (14)$$

The mean value is related to the distribution parameters as follows [7],

$$\overline{A} = \omega\ \Gamma\,(1 - \text{خطأ!} \qquad \gamma > 1 \qquad (15)$$

and

$$V_A = \text{خطأ!} \qquad \gamma > 2 \qquad (16)$$

where Γ is the Gamma function [7]. The model parameters and the mean-to-nominal ratios for zones 2A and 2B, in the Kingdom, are listed in Table 5.

5.2 Earthquake Load Models

Load effects due to seismic ground shaking are usually determined by static analysis method based on the base shear, **S**, which is calculated as follows [15],

$$S = \text{خطأ!}W \qquad (17)$$

where **Z** is the seismic zone number, **C** is a factor accounts for the system spectral amplification factor and the soil factor, R_W is the system quality factor, and W is the weight of structure.

The variability of earthquake load, **S**, is mainly attributed to the variability of **A** which is represented by **Z** in Eq. (17) . The variability of other parameters can be ignored if compared with that of **A** [2]. The mean-to-nominal ratio of the 50-year normalized earthquake load, λ_S , in the Kingdom, is found to be 0.70 and 0.67 for zones 2A and 2B with Vs = 137 and 135 percent, respectively. These results are listed in Table 5.

The obtained load models and their statistical characteristics are compared with those obtained in a similar U.S. studies [2] as presented in Table 6.

7. CONCLUSION

In this paper, loads models were developed and their statistical characteristics were estimated. Results indicate that dead load is normally distibuted with mean-to-nominal ratio of 1.15 and coefficient of variation of 10 percent. Live loads statistics for sustained and extreme loads were adopted from previous international studies. These studies indicated that sustained live load is well presented by Gamma function with mean-to-nominal ratio of 0.2 and a coefficient of variation of 70 percent whereas extreme live load is well presented by Type I extreme distibution with mean-to-nominal ratio of 1.10 and coefficient of variation of 25 percent . Extreme wind load is well presented by Type I extreme distribution with mean-to-nominal ratio of 0.75 and coefficient of variation of 60 percent. Extreme Earthquake load is well presented by Type II extreme distribution with mean-to-nominal ratio of 0.70 and coefficient of variation of 137 percent. These results are essential for reliability and risk analysis associated with code development and computation of load and resistance safety factors.

ACKNOWLEDGEMENT

This paper is part of a study sponsored by King Abdul-Aziz City for Science and Technology under grant number AR-11-57. The authors would like to express their thanks and appreciation for this support.

REFERENCES

1. American Concrete Institute, ACI Committee 318, "Building Code Requirements for Reinforced Concrete Structures (ACI 318M-95) ", Farmigton Hill, 1995.
2. Ellingwood, B. et al., "Development of A Probabilistic Based Load Criterion for American National Standard A58, NBS Publication 577, Washington, D.C., June 1980.
3. Arafah, A., Al-Zaid, R., Al-Haddad, M., AL-Tayeb, A., Al-Sulimani, G., and Wafa, F.," Development of a Solid Foundation for a National Reinforced Concrete Design Building Code," Final Report, KACST project No. AT-9-34, Riyadh, 1991, 350 pp.
4. Arafah, A., M., Al-Zaid, R., Al-Haddad, M., El-Tayeb, A., Al-Sulimani, G. and Wafa, F. , "Basic Studies Towards Reliability-Based Design Criteria for R.C. Buildings in the Kingdom," *Proceedings of the Third Saudi Engineering Conference*, pp. 818-822, April 28- May 1, 1991, King Saud University, Riyadd, Saudi Arabia.

5. Arafah, A., "Statistics for Concrete and Steel Quality in Saudi Arabia," *Magazine of Concrete Research,* London, Vol. 49, No. 180, September 1997, pp. 185-194.
6. Arafah, A., "Basic Statistics of Strength Limit States of Reinforced Concrete Members in Saudi Arabia," accepted for publication in The Arabian Journal for Science and Engineering, King Fahd University of Petroleum and Minerals, Dhahran, Saudi Arabia,.
7. Benjamin, J. and Cornell, C., "Probability Statistics, and Decision for Civil Engineers", McGraw-Hill, 1970.
8. Corotis, R., and Doshi, V., "Probability Models for Live Load Survey Results," Journal of the Structural Division, ASCE, Vol. 103, No. ST6, June, 1977, pp1257-1274.
9. Choi, E., "Live load for Office Buildings:Effect of Occupancy and Code Comparison," Journal of Structural Engineering, Vol. 116, No. 11 November 1990.
10. Simiu, E., and Scanlan, R., Wind Effects on Structures, Second Edition, Wiley-Interscience Publication, New York, 1986.
11. " Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers, ASCE 7-93, New York, New York, 1993.
12. Arafah, A.M., Siddiqi, G.H. and Dakheelallah A., "Extreme Wind Speeds in the Kingdom of Saudi Arabia," The Arabian Journal for Science and Engineering, Vol. 17, No:3, King Fahd University of Petroleum and Minerals, Dhahran, Saudi Arabia, 1992.
13. Al-Haddad, M., Siddiqi G. H., Al-Zaid, R., Arafah, A., Necioglu, A., and Turkelli, N., "A Study leading to a Preliminary Seismic Design Criteria for the Kingdom," Final Report No. AR-09-31. King AbdulAziz City for Science and Technology KACST, Riyadh, 1992.
14. Al-Haddad, M. S. , Siddiqi, G.H., Al-Zaid, R. Z., Arafah, A. M. , Necioglu, A., and Turkelli, N., "A Basis for Evaluation of Seismic Hazard and Design Criteria for Saudi Arabia," *Journal of Earthquake Engineering Research Institute, EERI, Earthquake Spectra,* Volume 10, No. 2, pp. 231-258, May 1994, USA.
15. "The Uniform Building Code," International Conference of Building Officials, California, USA, 1994.

List of Notations

D:	effects of dead load
L:	effect of live load
W:	effect of wind load
E:	effect of earthquake load
Q:	load effects
A:	a factor accounts for the variability of structural load
B:	a factor account for the uncertainties of load model
C:	a factor account for analysis uncertainties
λ_D, V_D:	mean-to-nominal ratio and the coefficient of variation of (D)
λ_L, V_L:	mean-to-nominal ratio and the coefficient of variation of (L)
λ_W, V_W:	mean-to-nominal ratio and the coefficient of variation of (W)
υ:	maximum annual wind speed
$\overline{V}$,V_υ:	mean and coefficient of variation of (υ)
$F_v(\upsilon)$:	Distribution function of (υ)
α , u:	scale and location parameters of the Type I extreme distribution of (υ)
υ_{50}:	50-year maximum wind speed
$\overline{V}_{50}$, V_{v50}:	mean and coefficient of variation of (υ_{50})
α_{50}, u_{50}:	scale and location parameters of the Type I extreme distribution of (υ_{50})
ω , γ :	model constants for Type II extreme distribution function
Γ:	Gamma function
S:	base shear force
Z:	seismic zone number
R_W:	system quality factor

Table 1 Basic Statistics of Weights of Non-Structural Items

Items	Average (kN/m^2)	σ (kN/m^2)	V%
Concrete block, 200 mm thickness*	3.0	0.15	5.0
Concrete block, 150 mm thickness*	2.4	0.13	5.4
Concrete block, 100 mm thickness*	1.6	0.08	4.7
Red clay block, 200 mm thickness*	2.0	0.06	3.0
Red clay block, 150 mm thickness*	1.6	0.03	2.0
Red clay block, 100 mm thickness*	1.2	0.09	7.1
Terrazo tiles (with marbel fragments)	0.59	0.06	11.0
Terrazo tiles (with stone fragments)	0.56	0.04	9.0
Ceramic tiles	0.15	0.03	19.0
Marble tiles	0.51	0.03	5.2
Polished limestone tiles	0.83	0.12	14.9
Plaster	0.37	0.10	26.7
Mortar under tiles on floors	0.54	0.10	19.0
Mortar behind tiles on walls	0.68	0.17	25.1
Fill materials (sand) under tiles on floor	1.30	0.40	31.2

* includes weight of mortar joints

Table 2 Design Wind Speed Parameters at Selected Cities in the Kingdom

City	Annual Wind Speed				50-Year Maximum Wind Speed			
	u (mph)	α (mph)	$\overline{V}$ (mph)	V_{υ}	$\overline{V}_{50}$ (mph)	V_{v50}	u_{50} (mph)	α_{50} (mph)
Dhahran	45.90	4.90	48.73	0.13	67.90	0.09	65.07	4.90
Gassim	63.19	11.28	69.70	0.21	113.83	0.13	107.32	11.28
Jeddah	48.59	6.44	52.31	0.16	77.50	0.11	73.78	6.44

Table 3 Statistics of the Normalized 50-year Wind Load (W/W_n) for the Selected Cities in the Kingdom.

City	50-year W/W_n				
	CDF	u	α	λ	V
Dhahran	Type I	0.841	0.218	0.72	0.59
Gassim	Type I	0.853	0.273	0.77	0.61
Jeddah	Type I	0.847	0.239	0.73	0.63

Table 4 Probabilities of Exceeding Specified PGA's in 50-Years for Zones 2A and 2B in the Kingdom

	Probability of Exceedence	
PGA	Zone 2A	Zone 2B
0.02	0.934	0.984
0.03	0.753	0.891
0.04	0.583	0.757
0.05	0.453	0.629
0.06	0.359	0.522
0.07	0.291	0.436
0.08	0.242	0.368
0.09	0.205	0.314
0.10	0.177	0.270
0.11	0.155	0.236
0.12	0.137	0.207
0.13	0.123	0.183
0.14	0.112	0.164
0.15	0.102	0.147
0.16	0.094	0.133
0.17	0.086	0.120
0.18	0.080	0.110
0.19	0.074	0.100
0.20	0.069	0.092
0.25	0.050	0.062
0.30	0.038	0.044
0.35	0.029	0.032
0.40	0.022	0.024
0.45	0.017	0.018
0.50	0.013	0.014

Table 5 Statistics of Peak Ground Acceleration in Zones 2A and 2B in the Kingdom

Zone	u	α	$\omega = e^u$	$\gamma = 1/\alpha$	Z	$\overline{A}$	$\overline{A} / A_n$
2A	-2.60	0.432	0.074	2.31	0.15	0.106	0.70
2B	-2.37	0.381	0.093	2.62	0.20	0.134	0.67

Table 6 Load Models in the Kingdom as Compared with those in U.S. A.

		DL	L_S	LL	WL	EL
KSA	CDF	Normal	Gamma	Type I	Type I	Type II
	u, ω	--	--	0.98	0.55	0.074
	α, γ	--	--	0.21	0.35	2.31
	λ	1.15	0.20	1.10	0.75	0.70
	V%	10	70	25	60	137
USA	CDF	Normal	Gamma	Type I	Type I	Type II
	u, ω	--	--	0.98	0.65	0.034
	α, γ	--	--	0.21	0.22	2.3
	λ	1.05	0.20	1.10	0.78	0.64
	V%	10	70	25	37	138