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Wind Effects Statistics of Diagrid High-rise Building

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Abstract: *Building codes typically provides a "basic wind speed (V)" corresponding to a given Mean Recurrence Interval (MRI) as a design value based on extreme value analyses. From the wind design standpoint, the use of a single wind velocity as a design load input assumes that systematic differences in the input necessarily cause systematic differences in the response of a structure. This assumption is based mostly on the results of studies on low-rise buildings, for which the assumption seems to hold. However, for high rise buildings it may not be true that the structural response hazard level will be proportional to the wind loading hazard level. In this paper, aerodynamic pressure time series from wind tunnel test are used to run 3D time history analyses on a numerical model of a diagrid high rise building. For the thirty years recorded wind speed data, thirty full scale time history analyses are performed on the FE model. For a particular response quantity, thirty response values are obtained and fit to an extreme value distribution to obtain a design value for a particular MRI. This value represents the "response hazard" for a particular hazard level (50-yr MRI for this case). Similarly, "response hazard" values are calculated for other response quantities such as base shear, base moment, base torsion, roof acceleration, and roof displacement. Using a single design wind speed, these response quantities are calculated and compared with the "response hazard" values. The result shows that design quantities based on "design wind speed" always overestimate the response except for maximum roof acceleration. If a diagrid high rise building is designed using a "design wind speed", it will underestimate the acceleration at roof level compared to the acceleration value calculated based on "response statistics".*
Keywords: *Diagrid Building, Hazard level, High-rise, Wind*

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I. INTRODUCTION

Building codes typically provide a "basic wind speed (V)" corresponding to a 50-yr Mean Recurrence Interval (MRI) as a design value. This value implies a 64% probability that the wind speed V will be exceeded in 50 year and is based on extreme value analyses. From the wind design standpoint, the use of a single wind velocity as a design load input assumes that systematic differences in the input necessarily cause systematic differences in the response of a structure.

In this paper a statistical analysis of the response of a 64-story diagrid structure is carried out based on the directional maximum annual wind speed data for Sacramento, California from 1950 to 1979 collected by the National Institute of Standards and Technology (NIST). This data is summarized in Fig. 1, where reference lines at certain threshold values (e.g., 10-yr, 50-yr & 685-yr MRI wind speed) are shown. The data in Fig. 1 contains directional wind speeds at 10m above ground in open terrain, based on data reported in Simiu et al. (1979). As this is data from California, it does not contain either hurricane or tropical storms.

The statistical analyses will use data from wind tunnel test performed on a scale model of the building described in the next section. This data provides the pressure time histories to run wind analyses on a numerical model of the building. For each wind speed corresponding to a particular year from Fig. 1, a full scale 3D time history analysis (THA) is performed and response quantities (e.g. displacement and acceleration at the building roof, base shear and base overturning moment, etc.) are obtained. Thus, for the thirty 3-sec peak gust velocities shown in Fig. 1, thirty full scale time history analyses are performed. Note that because of similitude and scaling issues, the few seconds of the actual wind tunnel test data results in wind records with durations of ranging from over 3.5 to over 9 hours for the prototype structure.

For a particular response quantity (for example base shear, FX), thirty response values are obtained and fit to an extreme value distribution (a Gumbel distribution is used in this study) to obtain a design value for FX for a particular MRI (say a 685-yr MRI). Denoting this design value as FX^{685}, this value represents the "response hazard" for a particular hazard level (685-yr MRI for this case). For comparison purposes, consider the design wind speed of 46.46 m/s calculated for a 685-yr MRI (bold red line in Fig. 1). Using this 46.46 m/s wind speed, a full scale 3D time history analysis of the building is conducted and the maximum base shear value is recorded. Let denote this design quantity as FX_{685} – this value represent a design response quantity for a particular wind "loading hazard" level (685-yr MRI for this case). Comparing FX^{685} with FX_{685} one can make conclusions regarding whether designing a structure based on "loading hazard" is adequate or not. Following the same procedure, other response quantities of interest can be compared.

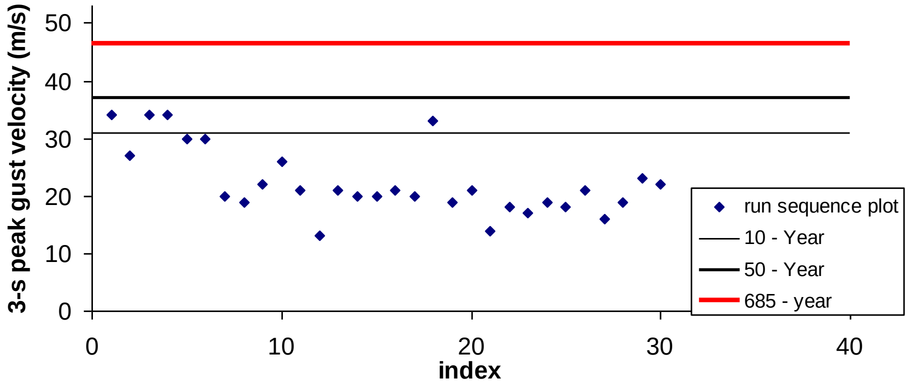

Figure 1. Run sequence plot

II. DESCRIPTION OF THE BUILDING AND WIND TUNNEL TEST

The initial wind data comes from a 1:600 scaled model used in a wind tunnel study for a building in a suburban terrain. This test was conducted at the CRIACIV Boundary layer wind tunnel (Ilaria, 2005). The wind is acting perpendicular to the long face of the building as shown in Fig. 2, where Cartesian axes are also defined along with the dimensions of the 64-story diagrid prototype building (Bhuiyan, 2011). Aerodynamic pressure time series were recorded at 120 pressure tap locations on the building envelope.

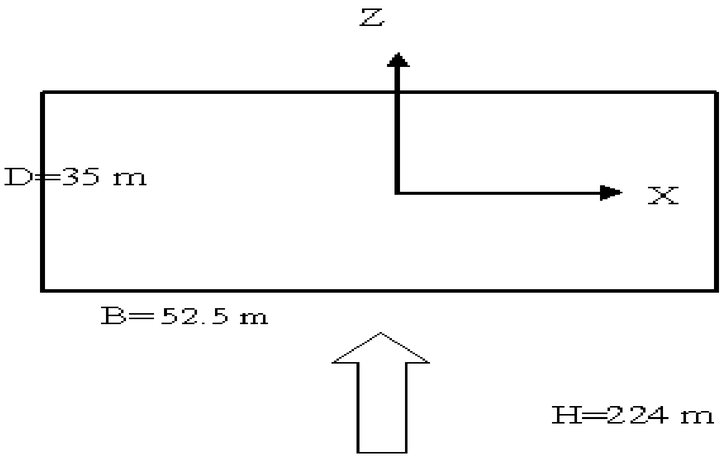

Figure 2. Building Dimensions and Wind Direction

Table 1 presents the calculation summary to convert the scale model pressure time series data into a prototype force time series data to use in the numerical analysis of the model of the full scale building. The load factor in Table 1 is a multiplication factor used to convert the pressure time series into a force time series. In Table 1, U_H represent the wind velocity at the roof level of the building with a 1-hour averaging time in a suburban area and Δt represents the sampling time period for the prototype scale. The last column of Table 1 shows the duration of the force time series in prototype scale. Now using the force time series at 120 location of the building envelope, a 3D time history analysis can be performed. After running full 3D time history analyses, accelerations and displacements at the roof level of the building and base reactions (FX, FY, MX, MY & MZ) are recorded. The expected maximum values of the time series are estimated assuming the marginal probability distribution of the time series as Gaussian (Bhuiyan, 2011).

Table 1. Calculation Summary to convert model scale pressure time series data into prototype force time series data to use in the numerical model of the full scale building

	Analysis for Suburban - 0^o (SE)					
	3-s gust in 10m Open terrain (m/s) SE	Load Factor	U_H (m/s)	Δt (sec)	dt , sec	Duration of time history, hours
1950	34	639.66	32.3163	1.723	0.1723	3.59
1951	27	403.38	25.6629	2.17	0.217	4.52
1952	34	639.66	32.3163	1.723	0.1723	3.59
1953	34	639.66	32.3163	1.723	0.1723	3.59
1954	30	498.01	28.5144	1.953	0.1953	4.07
1955	30	498.01	28.5144	1.953	0.1953	4.07
1956	20	221.34	19.0096	2.929	0.2929	6.1
1957	19	199.76	18.0591	3.083	0.3083	6.4
1958	22	267.82	20.9105	2.663	0.2663	5.55
1959	26	374.06	24.7124	2.253	0.2253	4.7
1960	21	244.02	19.96	2.79	0.279	5.8
1961	13	93.51	12.3562	4.506	0.4506	9.39
1962	21	244.02	19.96	2.79	0.279	5.8
1963	20	221.34	19.0096	2.929	0.2929	6.1
1964	20	221.34	19.0096	2.929	0.2929	6.1
1965	21	244.02	19.96	2.79	0.279	5.8
1966	20	221.34	19.0096	2.929	0.2929	6.1
1967	33	602.59	31.3658	1.775	0.1775	3.7
1968	19	199.76	18.0591	3.083	0.3083	6.4
1969	21	244.02	19.96	2.79	0.279	5.8
1970	14	108.45	13.3067	4.184	0.4184	8.72
1971	18	179.28	17.1086	3.255	0.3255	6.78
1972	17	159.91	16.1581	3.446	0.3446	7.2
1973	19	199.76	18.0591	3.083	0.3083	6.4
1974	18	179.28	17.1086	3.255	0.3255	6.78
1975	21	244.02	19.96	2.79	0.279	5.8
1976	16	141.66	15.2077	3.661	0.3661	7.63
1977	19	199.76	18.0591	3.083	0.3083	6.4
1978	23	292.72	21.861	2.547	0.2547	5.3
1979	22	267.82	20.9105	2.663	0.2663	5.55

III. RESULTS

From each response quantity time series, expected peaks are estimated. Table 2 summarizes the results obtained for the thirty analyses. The column “FX" represents the estimated peaks from the time series for the base shear in the X-direction (FX). The other columns in Table 2 represent similarly estimated peaks.

Fig. 3 shows the fitted Gumbel distribution for the yearly peaks of base shear FX (column 3 of Table 2). From the fitted Gumbel distribution, a design base shear FX of 13.8253×10^6 N is estimated for a hazard level of 685-yr MRI. As described in the last paragraph of Section I, let’s denote this design value as FX^{685} – which may be called “response hazard” as it was predicted from the probability distribution of the response base shears, FX. In Section I, it was estimated that the design wind speed for a 685-yr MRI was 46.46 m/s; this yields a base shear FX equal to 20.5181×10^6 N. Let’s denote this design quantity as FX_{685} – which may be called design base shear FX for a particular wind “loading hazard” level (685-yr MRI for this case). Comparing FX^{685} = 13.8253×10^6 N with FX_{685} =20.5181×10^6 N, it is evident that designing a high-rise building based on design wind speed is adequate and somewhat overconservative.

Table 2. Expected Maximum peaks for several response quantities

	3-s gust (m/s)	FX x10^6 N	FZ x10^6 N	MX x10^8 N-m	MZ x10^8 N-m	Torsion x10^7 N-m	Acc Z mili-g	Acc X mili-g	UZ cm	UX cm
1950	34	8.96	13.50	18.59	13.01	3.74	12.09	24.18	26.15	9.49
1951	27	4.45	8.13	10.82	6.09	2.42	5.85	9.35	15.10	4.62
1952	34	8.96	13.50	18.59	13.01	3.74	12.09	24.18	26.15	9.49
1953	34	8.96	13.50	18.59	13.01	3.74	12.09	24.18	26.15	9.49
1954	30	4.84	10.28	13.86	6.30	2.87	7.91	7.54	19.31	4.67
1955	30	4.84	10.28	13.86	6.30	2.87	7.91	7.54	19.31	4.67
1956	20	1.85	4.46	6.03	2.27	1.12	3.08	1.60	8.40	1.71
1957	19	1.76	3.90	5.20	2.21	1.04	1.88	2.11	7.20	1.67
1958	22	2.55	5.27	7.03	3.29	1.37	2.90	3.85	9.74	2.46
1959	26	3.73	7.64	10.37	4.85	2.09	5.80	6.02	14.47	3.62
1960	21	2.08	4.82	6.50	2.56	1.19	2.53	1.92	9.00	1.90
1961	13	0.71	1.74	2.28	0.87	0.45	0.31	0.39	3.15	0.65
1962	21	2.08	4.82	6.50	2.56	1.19	2.53	1.92	9.00	1.90
1963	20	1.85	4.46	6.03	2.27	1.12	3.08	1.60	8.40	1.71
1964	20	1.85	4.46	6.03	2.27	1.12	3.08	1.60	8.40	1.71
1965	21	2.08	4.82	6.50	2.56	1.19	2.53	1.92	9.00	1.90
1966	20	1.85	4.46	6.03	2.27	1.12	3.08	1.60	8.40	1.71
1967	33	6.83	12.54	17.03	9.38	3.51	11.00	14.32	23.63	7.02
1968	19	1.76	3.90	5.20	2.21	1.04	1.88	2.11	7.20	1.67
1969	21	2.08	4.82	6.50	2.56	1.19	2.53	1.92	9.00	1.90
1970	14	0.83	2.02	2.67	1.01	0.49	0.46	0.48	3.69	0.76
1971	18	1.44	3.44	4.58	1.74	0.94	1.28	0.94	6.34	1.30
1972	17	1.27	3.08	4.07	1.54	0.83	1.07	0.78	5.63	1.14
1973	19	1.76	3.90	5.20	2.21	1.04	1.88	2.11	7.20	1.67
1974	18	1.44	3.44	4.58	1.74	0.94	1.28	0.94	6.34	1.30
1975	21	2.08	4.82	6.50	2.56	1.19	2.53	1.92	9.00	1.90
1976	16	1.06	2.71	3.59	1.27	0.72	0.90	0.43	4.97	0.94
1977	19	1.76	3.90	5.20	2.21	1.04	1.88	2.11	7.20	1.67
1978	23	2.72	5.81	7.83	3.48	1.58	3.31	3.82	10.88	2.59
1979	22	2.55	5.27	7.03	3.29	1.37	2.90	3.85	9.74	2.46

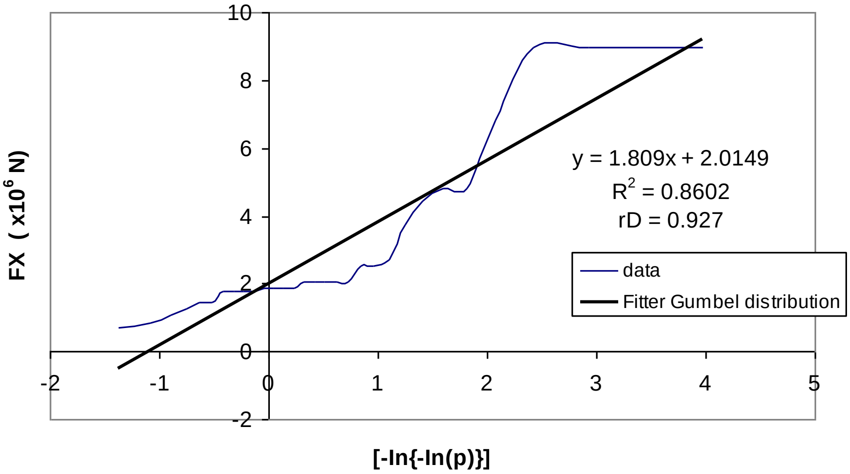

Figure 3. Fitted Gumbel distribution to the yearly peaks of base shear FX

Fig. 4 shows the fitted Gumbel distribution to the displacement UZ at the roof level. From the fitted curve, an UZ^{50} of 28.68 cm is estimated for a hazard level corresponding to a 50-yr MRI. A design wind speed of 37 m/s was estimated from Section I for 50-yr MRI. Using this wind speed a THA is conducted and a roof displacement UZ_{50} of 31.6 cm is recorded. Comparing UZ^{50} = 28.68 cm with UZ_{50} =31.6 cm, it is evident that designing a high-rise building based on design wind speed is adequate.

Following a similar procedure, other design quantities are also estimated. Table 3 summarizes the results where a comparison of design quantities based on "response statistics" and "design wind speed" are presented. Table 3 shows that design quantities based on "design wind speed" always overestimate the response except for maximum roof acceleration. If a high-rise building is designed using "design wind speed", it will give an underestimated value for Acceleration at roof level compared to the acceleration value calculated based on "response statistics".

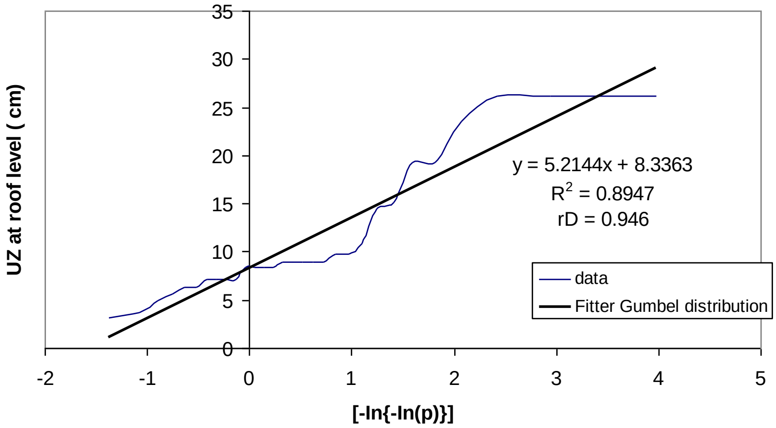

Figure 4. Fitted Gumbel distribution to the yearly peaks of roof displacement UZ

Table 3. Comparison of design quantities estimated based on "response statistics" & "design wind speed"

Hazard Level	Design quantity	Unit	Determined based on Response Statistics	Determined based on Design Wind Speed
685-yr MRI	base shear, FX	$x10^6$ N	13.8253	20.5
685-yr MRI	base shear, FZ	$x10^6$ N	22.0284	26.5
685-yr MRI	base OTM, MX	$x10^8$ N-m	30.2031	36.6
685-yr MRI	base OTM, MZ	$x10^8$ N-m	19.7859	30.7
685-yr MRI	base torsion	$x10^7$ N-m	6.1599	7.11
50-yr MRI	Roof Displacement, UZ	cm	28.68	31.6
10-yr MRI	Z-acceleration at roof	mili-g	**8.67**	**8.01**
10-yr MRI	X-acceleration at roof	mili-g	**13.87**	**8.36**

IV. CONCLUSION

Thirty full scale 3D time history analyses were carried out using pressure time series data from wind tunnel test to determine the structural response-hazard level. The study showed that design quantities based on "design wind speed" always overestimate the response except for maximum roof acceleration. If a high-rise building is designed using "design wind speed", it will give an underestimated value for acceleration at roof level compared to the acceleration value calculated based on "response statistics".

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