NIST NCSTAR 1-2
Federal Building and Fire Safety Investigation of the World Trade Center Disaster

Baseline Structural Performance and Aircraft Impact Damage Analysis of The World Trade Center Towers

(Appendices A-E)

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Appendix A
SALIENT POINTS WITH REGARD TO THE STRUCTURAL DESIGN OF THE WORLD TRADE CENTER TOWERS

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Salient points with regard to the structural design of The World Trade Center towers:

1. The structural analysis carried out by the firm of Worthington, Skilling, Helle & Jackson is the most complete and detailed of any ever made for any building structure. The preliminary calculations alone cover 1,200 pages and involve over 100 detailed drawings.
2. The buildings have been designed for wind loads of 45 lbs. per square foot which is 2¼ times the New York City Building Code requirements of 20 lbs. per square foot, the design load for the Empire State, Pan American and Chrysler Buildings. In addition to static wind loads, a complete dynamic analysis has been made to take into account extremely high velocity gusts.
3. The buildings have been investigated and found to be safe in an assumed collision with a large jet airliner (Boeing 707 - DC 8) travelling at 600 miles per hour. Analysis indicates that such collision would result in only local damage which could not cause collapse or substantial damage to the building and would not endanger the lives and safety of occupants not in the immediate area of impact.
4. Because of its configuration, which is essentially that of a beam 209' deep, the towers are actually far less daring structurally than a conventional building such as the Empire State where the spine or braced area of the building is far smaller in relation to the height.
5. The building as designed is sixteen times stiffer than a conventional structure. The design concept is so sound that the Structural Engineer has been able to be ultra-conservative in his design without adversely affecting the economics of the structure. This is not the case with conventional buildings where a more radical approach must be used if the building is to be constructed at reasonable cost.

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6. The structural concept is new but the design principles, the stress analysis and the theories of mechanics upon which the design is based are well known and are in accordance with good engineering practice.

7. The design has been reviewed by some of the most knowledgeable people in the construction industry. In a letter to John Skilling, the Structural Engineer for The World Trade Center, the Chief Engineer of the American Bridge Division of U. S. Steel Corporation said:
"In reviewing this design with our Operating and Construction Departments, we are very optimistic that you have turned a new page in the design of structural steel. It is high time that some new thinking be applied in our industry. In the words of our General Manager of Operating, Lester Larison, he said - 'It was the best damn thing that he has seen come down the pike in his 46 years of experience. Imagine designing a 100-story building for under 30 pounds per square foot.'"

8. The Engineering News-Record of January 30th carries a series of quotations from people in the building industry with regard to The World Trade Center design.

   A. James Ruderman, one of the outstanding New York Structural Engineers says that "The structural design of the tower buildings shows a commendable job of rethinking, where ideas were given a lot of thought and not just treated routinely."

   B. Harold Bernhard, partner, Shreve, Lamb and Harmon Associates, Architects, says "It's a magnificent project."

9. In an editorial in the same issue of the Record is the comment:
"Thus, the PNYA will not build as high as permitted all over its property, despite the high land costs in downtown Manhattan. Instead, the twin towers will occupy only 12% of the site. This plan should please the numerous vociferous critics of other recent New York projects not surrounded by large open spaces. It also permits the towers to be built

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with no setbacks without violating zoning regulations. Over-all, the design not only appears to be esthetically preferable to a set-back silhouette, but also lends itself to more economical construction and use of space. The PNYA, in addition, has engaged noted architects and consulting engineers to design the project. From the preliminary data released, it appears that the design of the twin towers will mark an important advance in skyscraper construction. Tall buildings are handicapped economically because the cost of structural framing and the space consumed by vertical transportation rise rapidly with increasing height. The Trade Center designers have departed from usually conventional practices to cut these costs."

10. We have been informed that the structural engineering firm of Ammann & Whitney has been approached by a leading New York architect with a request that this structural system be reviewed for possible incorporation in a large office building which the architect is presently designing.

11. The skyscraper is one of America's contributions to World Architecture. New York is the capital of skyscraper construction in the United States. The design of the towers of The World Trade Center is based on the lessons learned in constructing all the tens of millions of square feet of high rise buildings in this great city. The towers may be said to be the first buildings of the 21st Century and the design concepts which they embody will be incorporated in some measure in every future high rise building ever built.

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Appendix B
ESTIMATION OF SECTORIAL EXTREME WIND SPEEDS[1]

Abstract

We present a procedure for estimating extreme wind speeds corresponding to a sector-by-sector approach to the estimation of extreme wind effects. We provide details of the data sets and their treatment, as well as details of the estimates themselves, in a manner intended to be thorough, clear, and transparent. Efforts in the direction of clarity and transparency are in our view necessary if estimates of extreme winds and their effects are to meet the need for effective scrutiny by users and building authorities, and if a solid technical basis for a consensus among practitioners, standards organizations, and professional organizations is to be created in the near future.

Introduction

The estimation of extreme wind speeds at a given site is, in principle, straightforward. However, in practice, for any given location, differences between approaches used by various wind engineers or other professionals can lead to widely divergent estimates. To assess any particular extreme wind speed estimates it is necessary to scrutinize with care the procedure on which that estimate is based. This requires, in turn, that the procedure, each of its steps, and the attendant calculations, be explained clearly, transparently, in sufficient detail, and in a manner that should be independently verifiable by users or building inspection authorities. For an example of detailed assessment of an extreme wind speed estimation methodology and attendant calculations, see (Coles and Simiu, 2003).

At this time no sufficient guidance is available in standards for (a) the estimation of extreme wind speeds on buildings subjected to wind tunnel testing and (b) the integration of those wind speeds with aerodynamic data. Several procedures are used by various practitioners, but no professional consensus appears to exist on how discrepancies between the respective estimates can be reconciled or how the various methods should be amended to avoid situations – which do occur in actual practice – wherein various estimates of wind effects corresponding to the same nominal mean recurrence interval can differ by as much as 50 percent.

Some wind engineering professionals perform estimates of structural responses corresponding to winds blowing from each of a number of *sectors*. The sectors we consider here are the half-octants bisected by the NNE, NE, ENE,….,N compass directions. Those winds are referred to as *sectorial* wind speeds. In this paper we describe the estimation of sectorial wind speeds.

[1] This appendix was co-authored by William P. Fritz and Emil Simiu of NIST.

This paper is intended to serve as a contribution to the professional debate that, in our opinion, is needed to create a robust basis for a consensus on extreme wind estimation. We present here a procedure for estimating sectorial extreme wind speeds in a region with both hurricane and non-hurricane winds, and show in some detail a numerical example illustrating the procedure. To fix the ideas we will consider a site close to New York City (NYC).

Extreme wind speed data

Hurricane wind speed data. We make use in this note of the NIST simulated hurricane wind speed database which, to our knowledge, is the only non-proprietary hurricane database currently in existence. The database is available online at the following link on the worldwide web: ftp://ftp.nist.gov/pub/bfrl/emil/hurricane/datasets/. This subdirectory contains the relevant data sets of simulated hurricane wind speeds in nautical miles per hour (nmi/hr) at 10 meters above ground in open terrain, averaged over 1-min. There are 55 files with data for locations ranging from milepost 150 (file2.dat; near Port Isabel, TX) to milepost 2850 (file56.dat; near Portland, ME), spaced at 50 mile intervals. The structure of each data file is as follows:

Line 1: Milepost identifier, plus other information not needed for the analysis program.

Line 2: Blank, usually. In some files, the milepost number is repeated here.

Line 3: URATE and NSTRMS. URATE is the estimated annual rate of occurrence of hurricanes at and near this milepost, and NSTRMS is the number of simulated storms used to create the data. For all data sets included in this subdirectory, NSTRMS=999.

Lines 4-1003: The wind speed data for each of the NSTRMS simulated storms. There are a total of 18 numbers on each line. The first 16 are the maximum wind speeds in 16 specified directions, beginning with NNE and moving clockwise to N. The 17^{th} number is the maximum wind speed for ANY direction (i.e., the largest of the previous speeds). The final number (18th) number in each line is the storm identifier.

The NIST data sets are based on the "highly regarded work of Batts et al. (1980)," (unpublished report prepared for Insurance Services Office, Inc., New York City, 1994 by Robert H. Simpson, former director of the National Hurricane Center and creator with Herbert Saffir of the well-known Saffir-Simpson hurricane intensity scale). A variety of other hurricane models are currently available, although the data based thereon are, to our knowledge, proprietary. Agreement between wind speeds near the coastline based on the NIST data sets and on data sets based on other models is very good. At milestone 2500 (one of the milestones tabulated in Simiu and Scanlan (1996, p. 117) that is closest to New York City), the estimated hurricane mean hourly speeds at 10 m above ground in open terrain according to Batts et al. (1980), Simiu, Heckert and Whalen (1996) (both based on the NIST database), Georgiou et al. (1983), and Vickery and Twisdale (1995) are, respectively, about 30 m/s, 30 m/s, 30 m/s, and 29 m/s for the 50-year speeds, and 45 m/s, 43 m/s, 47 m/s, and 45 m/s for the 2000-year speeds. In evaluating these differences it should be kept in mind that sampling errors in the estimation of hurricane wind speeds in the New York City area have estimated coefficients of variation of roughly 10% for 50-year speeds and 20% for 500-year speeds (Coles and Simiu, 2003). Note that the sampling errors depend less on the number of simulated hurricanes in the database (999 in our case) than on the number of historical hurricanes (about 100) used to obtain statistics of the climatological parameters on which the simulations are based (i.e., radii of maximum wind speeds, atmospheric pressure defect, hurricane translation speed and direction, and so forth). Those statistics differ relatively little among the various simulation packages. It is the authors' understanding that hurricane wind speeds for the State of Florida, corresponding to various probabilities of exceedance, are currently being estimated by the NOAA Hurricane Research Division. In our opinion it would be desirable that this effort be expanded to cover all U.S. hurricane-prone regions.

Treatment of hurricane wind speed data. The data listed in the NIST database need to be rank-ordered for reasons explained subsequently in this note. The rank-ordered data for the location of interest (file 50, milestone 2550 – nearest to NYC – in the NIST database) and for the 202.5°and 225° sectors of interest are listed in Table 1. Note that for these sectors hurricane translation speeds and the relevant vortex speeds within the hurricanes at and near NYC are in many instances of opposite signs, resulting in relatively small and therefore negligible, or even vanishing, total hurricane wind speeds. It is therefore sufficient to show in the table only the largest 55 of the total of 999 data, while keeping in mind that all the 999 data should be accounted for in the calculations.

Table 1. Rank-ordered wind speeds (nmi/hr at 10m above ground in open terrain, averaged over 1-min) from NIST database for 202.5°and 225° sectors at milepost 2550.

Rank,*m*	SSW 202.5°	SW 225°	Rank,*m*	SSW 202.5°	SW 225°	Rank,*m*	SSW 202.5°	SW 225°
1	88.81	86.73	21	0	29.56	41	0	22.64
2	74.49	61.79	22	0	28.96	42	0	21.59
3	73.75	52.37	23	0	28.95	43	0	21.56
4	46.59	47.91	24	0	27.89	44	0	21.25
5	39.68	42.82	25	0	27.79	45	0	20.62
6	17.46	41.97	26	0	27.74	46	0	20.09
7	14.35	41.59	27	0	27.59	47	0	20.04
8	13.81	37.13	28	0	27.35	48	0	19.07
9	13.51	36.4	29	0	27.13	49	0	18.82
10	6.8	35.85	30	0	27.01	50	0	18.55
11	4.88	34.77	31	0	26.63	51	0	16.97
12	3.49	33.64	32	0	26.59	52	0	16.67
13	0	32.41	33	0	26.45	53	0	15.49
14	0	31.79	34	0	25.82	54	0	15.14
15	0	31.75	35	0	25.58	55	0	0
16	0	31.13	36	0	25.28			
17	0	30.64	37	0	24.16			
18	0	30.59	38	0	23.58			
19	0	30.01	39	0	23.04			
20	0	29.86	40	0	22.98			

Non-hurricane extreme wind speed data. In this paper we make use of wind speeds recorded using ASOS (Automated Surface Observing System) during the period 1983-2002, made available to NIST by the NOAA's National Climatic Center for three airports near NYC: La Guardia (LGA), Newark International Airport (EWR), and John F. Kennedy International Airport (JFK). The wind speed data sets include the peak 5-s gust speed multiplied by a factor of 10, for every hour within the period of record, in m/s. The data were recorded at 20 ft (6.1m) above ground until May 1, 1996 at LGA and JFK and until July 1, 1996 at EWR. They were recorded at 10 m above ground thereafter.

Treatment of non-hurricane wind speed data. The results being sought are expressed in terms of 3-s peak gust speeds at 10 m above ground in open (airport) terrain. Therefore, all data need to be transformed from 5-s peak gust speeds to 3-s peak gust speeds. This can be done to within a sufficient approximation through multiplication of the 5-s speeds by a factor of 1.02 (see ASCE 7-02 Standard, Figure C6.2). The data not recorded at 10m must also be adjusted to correspond to a 10 m elevation above ground. This involves the use of the power law

$$V(z_1)/V(z_2)=(z_1/z_2)^{\hat{\alpha}} \quad (1)$$

where, for 3-s peak gust speeds, the exponent $\hat{\alpha} = 1/9.5$ for Exposure C (see ASCE 7-02 Standard).

Note that in the data sets each wind speed is associated with the direction from which the wind is blowing. The directions from which the wind is blowing are measured in a clockwise direction from true north, and are recorded for 36 angles in 10 degree increments.

Data should be excluded from the analysis if (1) the record provides no direction for a recorded wind speed (this is the case for a relatively small number of speeds), and (2) if the data have a quality code other than 'good', as provided explicitly in the NOAA data set. Only one measurement at JFK (the maximum speed in the 50° sector in 1987) and two measurements at LGA (the maximum speeds in the 210° sector in 1983 and in the 200° sector in 1984) had a quality code other than 'good'.

Maximum wind speeds are extracted from an airport data set for each of the 36 wind directions for each year of record. For example, 20 years of maximum hourly wind speeds produce 36 × 20 values. The dates of major hurricanes of record for NYC during these 20 years should be checked against the dates of each tabulated maximum wind speed. Data recorded on September 27 and 28, 1985 (hurricane Gloria) and August 19 and 20, 1991 (hurricane Bob) (Neumann et al., 1993) should not be considered and the largest *non*-hurricane wind speeds in the records should be used instead.

The 36 directions are reduced through an appropriate scheme to 16 directions that match the NIST hurricane data. This can be accomplished by defining the wind speed data set associated with, say, the 22.5° sector as the set of maximum yearly wind speeds from the NOAA data sets for the 10°, 20°, 30° and 40° sectors. This definition is somewhat conservative, since the 22.5° sector is associated with the narrower sector 11.25° to 33.75°, rather than the sector 5°-45°. However, in our opinion this conservatism is warranted by the fact that the data samples at our disposal are limited to 20 years. A longer than 20-year data set for the 11.25° to 33.75° sector may contain wind speeds that, during a 20-year interval, have actually blown within the small sectors 5° to 11.25° and 33.75° to 45°. This minor conservatism affecting wind speeds is an empirical and reasonable way of accounting for possible sampling errors with respect to the direction of extreme speeds, for which to our knowledge no applicable theory is available to date.

Estimates of extreme wind speeds

Estimation of extreme wind speeds regardless of whether they are associated with hurricanes or non-hurricane winds. Estimates of extreme wind speeds at 10 m above ground in open terrain at or near the site must take into account both hurricane and non-hurricane winds. We are interested in estimates of sectorial wind speeds, that is, wind speeds that occur in a specified sector defined by the azimuth of its bisector and the total angle swept by the sector. For specificity, in this note we illustrate our estimates of sectorial wind speeds for the 22.5° sectors defined by the bisectors with a 202.5° and a 225° azimuth (i.e., for the SSE and SE directions).

Let the probability of non-exceedance of the wind speed v be denoted by $P(V<v)$. This probability represents the probability that hurricane wind speeds do not exceed v *and* that non-hurricane wind speeds do not exceed v. Denoting the probability that hurricane wind speeds do not exceed v by $P_H(V<v)$ and the probability that non-hurricane speeds do not exceed v by $P_{NH}(V<v)$, and noting that the occurrences of hurricane and non-hurricane speeds are independent events, we have

$$P(V<v)= P_H(V<v)\, P_{NH}(V<v). \qquad (2)$$

The corresponding mean recurrence interval of the wind speed V is, by definition,

$$N=1/[1-P(V<v)]. \qquad (3)$$

Estimation of probabilities $P_H(V<v)$. For wind speeds blowing from any one of the 16 compass directions (corresponding to the 16 half-octants) the following procedure is used:

- Extract from the NIST database the hurricane mean rate of arrival (μ = 0.305/year) and, for the wind direction of interest, the 999 hurricane wind speed data for New York City (milestone 2550).
- Rank-order the 999 data. (This was done in Table 1.) If the hurricane mean arrival rate URATE (henceforth denoted in this paper by μ) was 1/year, the highest speed would have a 999-year (or approximately 1,000-year) mean recurrence interval. However, if μ <1, then the mean recurrence interval of the highest speed in the set is 999/ μ. (For example, if the mean arrival rate were one hurricane every two years (μ =0.5), then the mean recurrence interval of the highest speed in the set would be 999/0.5=1998, or about 2000 years.)
- The m-th largest speed in the set of 999 speeds corresponds to a mean recurrence interval N=999/(μ m). For example, if – as is the case for New York City area – the estimated mean rate of arrival is 0.305, the mean recurrence intervals of the first highest, second highest, and 65 highest speed are about 999/0.305=3275 years, 999/(0.305 x 2)=1640 years, and 999/(0.305 x 65)=50 years, respectively. Conversely, the hurricane wind speed with an N_H-year mean

recurrence interval corresponds to the m-th largest wind speed in the set, where m=999/(μN_H).

- The probability that this wind speed does not exceed v is defined as follows:

$$P_H(V<v)=1-1/N_H. \tag{4}$$

Other estimation procedures are available, however to date there is no definitive consensus on which procedure is to be preferred. Some analysts believe that extreme value distributions are inadequate owing to their validity, strictly speaking, under asymptotic assumptions only; others believe that Weibull distributions are not appropriate since they are distributions of the smallest values, rather than distributions of the largest values. In spite of its theoretical non-optimality in terms of the precision of some estimates. the non-parametric approach used in this paper appears to be relatively non-controversial and appears to have been adopted by other analysts of hurricane wind speeds.

Estimation of probabilities $P_{NH}(V<v)$. The N_{NH}-year mean recurrence interval may be estimated by using techniques discussed in Simiu and Scanlan (1996, Appendix A1.7). Although other distributional models may be adopted, the least controversial model for extreme wind speeds of non-hurricane origin appears to date to be the Type I extreme value distribution. The mean recurrence interval associated with the non-hurricane wind speed V is then

$$N_{NH} = \exp\left[\frac{V-\bar{v}}{0.78s} + 0.577\right] \tag{5}$$

The mean, $\bar{v}$, and standard deviation, s, are calculated from the yearly maximum wind 3-s peak gust speeds at 10 m above ground in open terrain for the sector of interest. The probability that the wind speed, V, does not exceed v is

$$P_{NH}(V<v)=1-1/N_{NH}. \tag{6}$$

The requisite probability $P(V<v)$ can be obtained from Eqs. 2, 4, and 6.

Numerical example

We seek the 50-, 500- and 720-year winds blowing from the sectors nominally associated with the 202.5° and 225° sectors for the area around New York City. We use 20 years of non-hurricane wind speed data measured at LGA and the NIST hurricane wind speed data for those sectors. The choice of the LGA data set is commented upon subsequently.

Let us first consider the 3-s peak gust speed V=100 mph at 10 m above ground in open terrain, and calculate its mean recurrence interval (Eq. 3). Recall that the estimated hurricane arrival rate at milepost 2550 is μ = 0.305/year. The 100 mph, 3-sec gust wind speed is divided by 1.525 (for conversion to mean hourly speeds), then divided by 1.15 (for conversion to nmi/hr) and finally multiplied by 1.25 (for conversion to 1-min averaging time) (see ASCE 7-02 Standard, Figure C6.2). The 1-min speed at 10 m above ground in open terrain corresponding to the 100 mph peak 3-s speed is therefore 71.3 nmi/hr. This value ranks in Table 1 m = 3.1 and m=1.6 for the 202.5° and 225° sectors, respectively. The mean recurrence intervals of a 100 mph, 3-sec gust hurricane speed are therefore:

$$N_{H,202.5^\circ} = \frac{999}{0.305(3.1)} = 1057 \text{ years}$$

$$N_{H,225^\circ} = \frac{999}{0.305(1.6)} = 2047 \text{ years}$$

and the probability that the 100 mph, 3-sec wind does not exceed v is

$$P_{H,202.5^\circ}(100\text{ mph, 3 - s} < v) = 1 - \frac{1}{1057} = 0.99905$$

$$P_{H,225^\circ}(100\text{ mph, 3 - s} < v) = 1 - \frac{1}{2047} = 0.99951\,.$$

Note that if a Poisson-based approach to the estimation of the mean recurrence intervals was adopted, instead of the approach used in this paper, the results would be identical for practical purposes. The mean recurrence interval obtained by the Poisson-based approach is $N=1/\{1-\exp\{-\mu[m/(999+1)]\}\}$. This yields 1058 years for 202.5° sector and 2049 years for the 225° sector.

For non-hurricane winds, maximum hourly wind speeds at LGA airport are shown in Table 2 for the two directions considered and for each of 20 consecutive years (1983 to 2002). The original speeds in m/s, averaged over 5-sec, and affected by a scale factor of 10 from the NOAA data set are provided in Table 2 along with their converted values in 3-s peak gusts in mph at 10 meters. Also shown are the four directions of the NOAA data from which the maximum value is drawn for the 202.5° and 225° sectors. The mean ($\bar{v}$) and standard deviation (s) of each set of 20 values are also provided.

Table 2. Maximum non-hurricane wind speeds (mph, 3-s), LaGuardia (LGA).

	202.5° 190°,200°,210°,220°		225° 210°,220°,230°,240°	
Year	0.1m/s,5-sec	Mph,3-sec	0.1m/s,5-sec	mph,3-sec
1983	319	77	267	64
1984	268	65	268	65
1985	118	28	108	26
1986	113	27	103	25
1987	170	41	118	28
1988	154	37	134	32
1989	149	36	154	37
1990	154	37	113	27
1991	113	27	149	36
1992	138	33	118	28
1993	128	31	128	31
1994	118	28	128	31
1995	118	28	113	27
1996	154	37	103	24
1997	113	26	149	34
1998	118	27	118	27
1999	144	33	118	27
2000	134	31	113	26
2001	123	28	123	28
2002	123	28	123	28
mean		35.3		32.6
std		13.0		11.5

The mean recurrence interval of the 100 mph, 3-sec gust as a non-hurricane wind is therefore:

$$N_{NH,202.5^\circ} = \exp\left[\frac{100-35.3}{0.78(13.0)} + 0.577\right] = 1051 \text{ years}$$

$$N_{NH,225^\circ} = \exp\left[\frac{100-32.6}{0.78(11.5)} + 0.577\right] = 3265 \text{ years}$$

and the probability that a 100 mph, 3-sec wind does not exceed v is

$$P_{NH,202.5^\circ}(100 \text{ mph,3 - s} < v) = 1 - \frac{1}{1051} = 0.99905$$

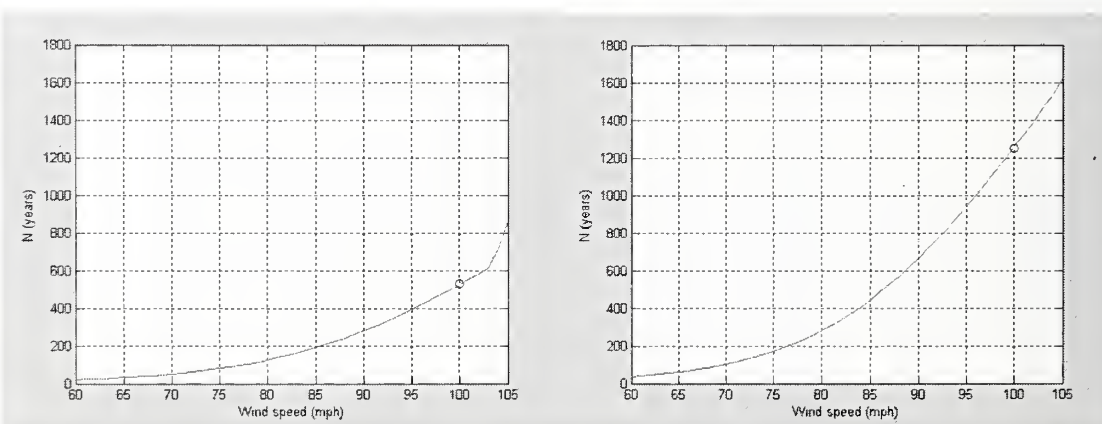
$$P_{NH,225^\circ}(100 \text{ mph,3 - s} < v) = 1 - \frac{1}{3265} = 0.99969.$$

In our opinion it would be desirable that a concerted effort be made that would engage NOAA on the one hand and wind and structural engineering professionals on the other, aimed at making wind speed observations archived by NOAA available in a suitable, user friendly format to the structural engineering community. The mean recurrence interval for the peak 3-s gust 100 mph speed, regardless of whether it is associated with hurricane or non-hurricane winds, is calculated using Eqs. 2, 4, and 6:

$$N_{202.5^\circ} = \frac{1}{1-P(100<v)} = \frac{1}{1-(0.99905)(0.99905)} = 527 \text{ years}$$

$$N_{225^\circ} = \frac{1}{1-P(100<v)} = \frac{1}{1-(0.99951)(0.99969)} = 1250 \text{ years.}$$

The procedure just described was followed for wind speeds between 60 and 105 mph. The mean recurrence interval of the wind speeds – regardless of whether they are associated with hurricane or non-hurricane winds – is plotted in Figure 1 for the two sectors. The mean recurrence intervals for the V=100 mph above are marked with a circle in the respective plots.



(a) (b)

Figure 1. Combined mean recurrence intervals as a function of peak 3-s gust wind speed for the (a) 202.5° and (b) 225° sectors.

Estimates of the 50-, 500- and 720-year, 3-s peak gust winds are obtained from Figure 1 and are shown in Table 3.

Table 3. Estimates of the NYC 50-, 500- and 720-year speeds, regardless of whether they are associated with hurricane or non-hurricane winds, at 10m above ground in open terrain for the 202.5° and 225° sectors.

	N-year wind (mph,3-s)		
Sector	50-yr	500-yr	720-yr
202.5°	69.8	99.1	104.1
225°	63.0	86.3	91.1

Choice of LGA sectorial data versus EWR and/or JFK sectorial data

The estimated sectorial wind speeds associated with the 202.5° and 225° directions were found to differ significantly for the LGA and EWR records, on the one hand, and the JFK record on the other. This may be due to relatively large sampling errors associated with wind directionality. In view of the uncertainties associated with sectorial wind speeds it appeared prudent to consider the LGA data above, whose variability for the sectors of interest is largest. Had the EWR data been considered instead, the final results would have been marginally lower. However, had the JFK results been used, the results would have been significantly smaller. This is due to the absence in the JFK record of some of the relatively high wind speeds that are present in the sectors of interest for LGA and EWR. This is an example of the occurrence of significant sampling errors in a sectorial wind speed record.

Rather than making use of the LGA data set alone, the analyst may be tempted to use a "super-station" comprising the data from the LGA, EWR, and JFK stations. However, in our opinion this consolidation of the three data sets into one larger data set would provide an inadequate basis for performing more precise estimates. The reason for this statement is that the three stations are relatively close to each other. The respective wind speed records are not necessarily independent, and gust speeds contain variabilities associated with turbulent fluctuations that may mask the actual correlations between the three records. In our opinion the issue of superstations constructed for stations that are geographically close needs to be researched in the future.

Comparison of extreme wind speed estimates at the three NYC airports

It was noted in the previous section that sectorial speeds can vary fairly significantly from station to station. It is of interest to compare extreme wind speed estimates at EWR, JFK and LGA without regard to wind direction. To do this, maximum wind speeds, regardless of their direction, are used in the procedure described earlier in lieu of sectorial wind speeds. That is, we consider hurricane winds from column 17 in file 50 of the NIST database and maximum yearly non-hurricane winds from the NOAA data set. Thus, non-hurricane data consist of 20 observations for each of the three NYC airports. Mean recurrence intervals of wind speeds at each airport, regardless of whether they are associated with hurricane or non-hurricane winds, and regardless of their direction, are plotted in Figure 2. The 50-year 3-s peak gust speed at each airport, regardless of direction, is 112.2 mph.

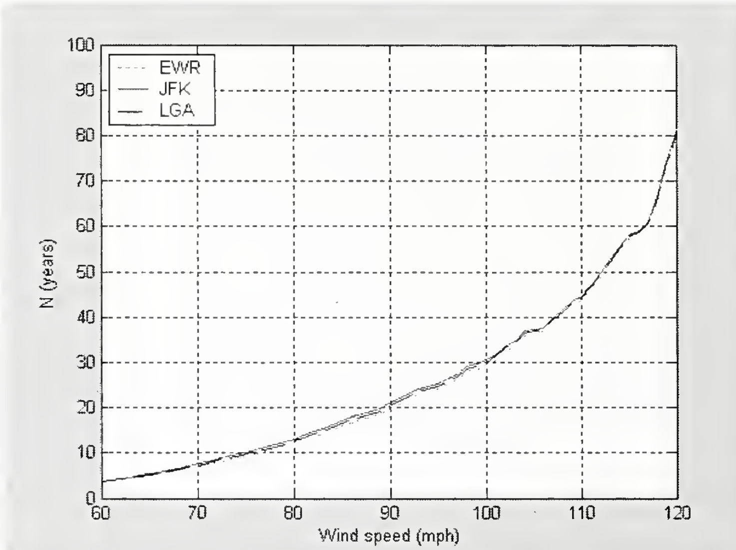


Figure 2. Mean recurrence intervals of wind speeds – regardless of whether they are associated with hurricanes or non-hurricane winds, and regardless of direction – for LGA, EWR, and JFK airports.

For any specified wind speed, the mean recurrence interval is generally shorter for winds regardless of their direction than for winds blowing from one sector only. The remarkable agreement between the estimates of extreme wind speeds at the three airports contrasts with the far less satisfactory agreement observed for the sectorial wind speeds. In other words, sectorial wind speeds appear to exhibit significant sampling errors for which, as mentioned earlier, no applicable theory or research appear to be available to date. This justifies, in our opinion, the use of the data set among the three available airport data sets that yields the most conservative results. In light of these remarks, we believe that caution is also warranted on the use of overly refined schemes for estimating extreme wind speeds for any one angular sector in approaches to wind directionality effects other than the sector-by-sector approach, e.g., the up-crossing approach.

Summary and conclusions

We presented a procedure for estimating extreme wind speeds corresponding to a sector-by-sector approach to the estimation of extreme wind effects. We provided details of the data sets and their treatment, as well as details of the estimates themselves, in a manner intended to be both clear and transparent. Efforts in the direction of clarity and transparency are in our view indispensable if estimates of extreme winds and their effects are to meet the need for effective scrutiny by users and building authorities, and if a solid technical basis for a consensus practitioners, standards organizations, and professional organizations is to be created in the near future.

In the authors' opinion it would be desirable (1) that the NOAA's Hurricane Research Division expand in the future its current efforts aimed at estimating hurricane wind speeds, with a view to covering all U.S. hurricane-prone regions, and (2) that NOAA's wind speed archives for non-hurricane wind speeds be made available to the wind and structural engineering communities in a suitable, user-friendly format to be agreed upon by NOAA and qualified representatives of those communities.

Acknowledgement

We wish to thank William Brown of the National Climatic Center (National Weather Service) for providing valuable help on the LaGuardia, Newark International Airport, and John F. Kennedy International Airport data sets, and information on the anemometer height history for those sets.

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Appendix C
Wind Tunnel Testing and the Sector-by-Sector Approach to Wind Directionality Effects[1]

ABSTRACT

We examine the sector-by-sector approach used by some wind tunnel operators to specify extreme wind effects. According to this criterion the design of a structural member subjected to wind loads is adequate if the stresses induced by the largest sectorial wind speed with a 50-yr mean recurrence interval does not exceed the maximum allowable wind-induced stress for that member, sectorial wind speeds with a 50-yr mean recurrence interval being estimated separately for each of the eight 45° (or the sixteen 22.5°) azimuthal sectors. We show that this approach leads to estimates of wind effects that are unconservative (i.e., on the unsafe side), owing to their failure to consider the overall effects of winds blowing from all sectors.

INTRODUCTION

The sector-by-sector approach to the estimation of wind directionality effects consists of estimating, separately, the wind speeds with a 50-yr mean recurrence interval (MRI) for winds blowing from each of the eight 45° sectors of the horizontal plane. Those wind speeds are referred to as the 50-yr sectorial speeds. For definiteness we consider the case of eight 45° sectors and of a 50-yr MRI, but the same definition can be extended for sixteen 22.5° sectors and any desired MRI.

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[1] To appear in the *Journal of Structural* Engineering, ASCE, July, 2005. This appendix was coauthored by Emil Simiu, ASCE, NIST Fellow, Structures Group, National Institute of Standards and Technology, Gaithersburg, MD 20899-8611, and James J. Filliben, Leader, Statistical Engineering Group, National Institute of Standards and Technology, Gaithersburg, MD 20899-8980.

Some wind tunnel operators specify wind effects based on the following criterion, henceforth referred to as the *sectorial design criterion*: for any given member, the maximum allowable wind-induced effect, R, (e.g., the maximum allowable wind-induced stress) must not be exceeded by the largest of the wind effects $Q_{j,50}$ (j=1,2,..,8) induced by the eight 50-yr sectorial speeds $v_{j,50}$. We denote by k the sector where this largest wind effect, denoted by $Q_{k,50}$, occurs. The purpose of this work is to show that the sectorial design criterion is unconservative (i.e., on the unsafe side) relative to the physically-based criterion, henceforth referred to as the *regular design criterion*, which states that the maximum allowable wind-induced effect R should not be exceeded by the 50-year effect induced by wind blowing from any direction (rather than just from the sector k).

It would be desirable to address this question by making use of the joint extreme value probability distributions (including correlations) of the wind speeds at the location of interest. Unfortunately, to our knowledge, expressions for such distributions do not exist. Bounds for the joint probabilities of interest may be estimated (Simiu et al., 1985; Simiu, Leigh, and Nolan, 1986), but such an approach can be unwieldy owing to combinatorial explosion problems. For the purposes of this work, which is addressed to structural engineers, it also has the drawback of not being sufficiently intuitive.

ASSESSMENT OF THE SECTORIAL DESIGN CRITERION

Intuitive Approach. Let $v_{j,Nj}$ denote the sectorial wind speeds that blow from the sector j ($1 \leq j \leq 8$) and cause the allowable wind effect R (the subscript N_j denotes the mean recurrence interval of the wind speed $v_{j,Nj}$). For $j=k$ we have N_k = 50 years. For $j \neq k$ the mean recurrence intervals N_j exceed 50 years. (If N_j were 50 years or less for any $j \neq k$, then R would be attained under sectorial wind speeds $v_{j,50}$, rather than under the sectorial wind speed $v_{k,50}$, which would be contrary to the sectorial design criterion.)

Let $F_Q(Q \leq R)$ denote the probability that the largest yearly wind effect Q, regardless of the direction from which the wind blows, does not exceed R. If the number of sectors were limited to one, then we would have, with notations similar to those used earlier,

$$F_Q(Q \leq R) = \text{Prob}\,(v_1 \leq v_{50}) = 1 - 1/50 = 0.98,$$

where v_1 denotes the wind speed inducing the effect Q. In this particular case the sectorial design criterion would be adequate.

For multi-directionally defined wind speeds and responses the following relation is consistent with the use of the sectorial design criterion:

$$F_Q(Q \leq R) = \text{Prob}(v_1 \leq v_{1,N1},\ v_2 \leq v_{2,N2}, \ldots,\ v_8 \leq v_{8,N8}) \quad (1)$$

in which one of the indexes j=1, 2, .., 8 has the value k, to which there corresponds the sectorial speed $v_{k,Nk}$ with N_k=50 years, all other N_j's being larger than 50 years. Let us consider the following three cases: positively correlated speeds, independent speeds, and negatively correlated speeds. For each of these cases we will examine the probability $F(Q \leq R)$. If it were true that $F(Q \leq R)=0.98$, the sectorial design criterion design would be adequate. If $F(Q \leq R)<0.98$, the design performed in accordance with the sectorial design criterion would be unconservative. If $F(Q \leq R)>0.98$ the opposite would be the case.

Case 1. The speeds $v_1, v_2, \ldots, v_8$ are *perfectly, positively correlated.* This means that for all $j \neq k$, we have $v_j = \alpha_j\, v_k$, where α_j are constants. Therefore,

$$F_Q(Q \leq R) = \text{Prob}(v_k \leq v_{k,50}) \quad (2)$$

$$= 0.98.$$

Equation 2 is valid because, by the definition of the sectorial design criterion, the occurrence of the event $v_k \leq v_{k,50}$ implies the occurrence of the events $v_j \leq v_{j,Nj}$ for all j. It follows that in Case 1 the sectorial design criterion is adequate.

Case 2. The speeds $v_1, v_2, \ldots, v_8$ are mutually *independent*. The mutual correlations of pairs of sectorial speeds then vanish. This implies

$$F_Q(Q \leq R) = \text{Prob}(v_1 \leq v_{1,N1}, v_2 \leq v_{2,N2}, \ldots, v_8 \leq v_{8,N8}) \quad (3a)$$

$$= \text{Prob}(v_1 \leq v_{1,N1})\ \text{Prob}(v_2 \leq v_{2,N2}) \ldots\ \text{Prob}(v_8 \leq v_{8,N8}) \quad (3b)$$

$$\leq 0.98, \quad (3c)$$

i.e., the mean recurrence interval of the event $Q \leq R$ is equal to or less than 50 years. The inequality (3c) holds because in Eqs. 3, as in Eq. 1, one of the indexes j=1, 2, .., 8 has the value k, to which there corresponds the sectorial speed $v_{k,Nk}$ with N_k=50 years, and all other N_j's are equal to or larger than 50 years. Consider, for example, the case in which the effects from one of the sectors were dominant, that is, the mean recurrence interval of the event that winds from that sector would cause R to be exceeded would be 50 years, while for the other sectors the corresponding mean recurrence intervals would be much longer, say 250 years. Then, $F_Q(Q \leq R) = (1 - 1/50) \times (1 - 1/250)^7 = 0.98 \times 0.996^7 \approx 0.95$, corresponding to a mean recurrence interval of the event $Q > R$ equal to 1/(1 - 0.95)=20 years. In other words, *the sectorial design criterion would lead to an underestimation of the wind effect.* It is reasonable to expect that this statement remains true even if the correlations do not vanish but are relatively small.

Case 3. The speeds $v_1, v_2, \ldots, v_8$ have *negative correlations*. To illustrate the significance of this case from the point of view of the problem considered in this note, we consider the model consisting of one die with two sets of numbers, one in blue and one in red, as follows. For faces 1, 2, 3, 4, 5, 6, the blue numbers are 1, 2, 3, 4, 5, 6, and the red numbers are 6, 5, 4, 3, 2, 1, respectively. The correlation coefficient between the red and blue outcomes is −1. The probability of the event of throwing a 4 or larger number, regardless of color, is 1 -- to which there corresponds a *mean recurrence interval of one throw*. (Blue and red numbers would correspond in our analogy to north and south winds, say.)

Instead the model just described, we now consider a model consisting of one die with two sets of numbers, one in blue and one in red, but with the following sets of numbers for faces 1, 2, 3, 4, 5, 6. Blue: 1, 2, 3, 4, 5, 6, and red: 1, 2, 3, 4, 5, 6, respectively. In this case the correlation coefficient between the red and blue outcomes is 1 (*perfect positive correlation*). The probability of throwing a 4 or larger number, regardless of color, is 1/2, to which there corresponds a *mean recurrence interval of two throws*, rather than one throw, as in the case of the die with negative correlation. If exceeding the critical value 4 is undesirable, it is seen that the case of negative correlation is more unfavorable than the case of positive correlation (the undesirable outcome occurs more frequently in the former than in the latter case).

It is of interest to also consider the case of throwing two ordinary dice, one with the blue numbers 1, 2, 3, 4, 5, and 6, and the other with red numbers 1, 2, 3, 4, 5, and 6. In this case *the correlation vanishes*, and the probability of getting in a throw of the two dice an outcome of 4 or larger is 27/36=0.75, i.e., *the mean recurrence interval of this outcome is 1.33 throws*. Again, this outcome occurs more frequently than in the case of positive perfect correlation, which is consistent with our earlier comparison between Case 1 and Case 2.

The preceding arguments suggest that *considering the case of strongly positive correlation when the correlation is in fact low or negative would overestimate the mean recurrence interval of the critical event.* This statement is valid not only for the cases of perfect positive correlation and negative or zero correlation. This can be checked by considering, for example: (a) Instead of a die with perfectly negatively correlated red and blue outcomes, one in which the blue and the red numbers are 1, 2, 3, 4, 5, 6, and 4, 3, 2, 2, 1, 1, respectively; for this die the correlation coefficient is -0.75, and the mean recurrence interval of an outcome of 4 or larger, regardless of color, is 1.5 throws. (b) Instead of the two dice considered earlier, two dice with blue and red numbers 1, 2, 3, 4, 5, 6, and 1, 1, 2, 2, 3, 4; in this case the correlation coefficient is again zero, and the mean recurrence interval of a blue or red outcome of at least four is 1.7 throws. (c) Instead of the die with perfectly positive correlation, one in which the blue and red numbers are 1, 2, 3, 4, 5, 6, and 1, 1, 2, 2, 3, 4, respectively; in this case the correlation coefficient is 0.86 and the mean recurrence interval of an outcome of 4 or larger, regardless of color, is 2 throws. Thus, the mean recurrence interval of this outcome is, again, shorter for both the uncorrelated case (1.7 throws) and the negatively correlated (1.5 throws) case that it is for the positively correlated case (2 throws).

Our choice of an intuitive argument is deliberate – it is intended to render our finding as clear as possible to practicing structural engineers, who may or may not have a theoretical probabilistic background. More basic probabilistic arguments are now adduced that strengthen and generalize our finding, without injecting unduly elaborate probabilistic manipulations.

Probabilistic approach. The advantage of a probabilistic argument is that is it more general. We invoke the definition of conditional probability:

$$P(E_1 \mid E_2) = \frac{P(E_1, E_2)}{P(E_2)} \tag{4a}$$

$$P(E_2 \mid E_1) = \frac{P(E_1, E_2)}{P(E_1)} \quad . \tag{4b}$$

from which it follows:

$$\begin{aligned} P(E_1, E_2) &= P(E_1 \mid E_2)P(E_2) \\ &= P(E_2 \mid E_1)P(E_1). \end{aligned} \tag{5a,b}$$

In Eqs. 4 and 5 $P(E_1,E_2)$ is the probability of occurrence of both events E_1 and E_2, $P(E_1|E_2)$ is the conditional probability of occurrence of event E_1 given that event E_2 has occurred, $P(E_2)$ is the probability of event E_2, and similar definitions hold for the second the above equalities. It follows from Eqs. 5 that

$$P(E_1,E_2) \le \min\{P(E_1), P(E_2)\} \tag{6a}$$

For three events E_1, E_2, and E_3, it can be shown that

$$P(E_1,E_2,E_3) \le \min\{P(E_1),P(E_2),\mathrm{P}(E_3)\}, \tag{7}$$

By induction, Eq. 7 may be extended for any number of events E_m (m=1,2,...).

Let the event $v_j \le v_{j,50}$ be denoted by Ej. The application of the extension of Eq. 7 for 8 events E_j (i.e., to Eq. 1) shows that $F_Q(Q \le R) \le 0.98$.

Another, more intuitive way of conveying this result is the following. If the structure was strengthened so that it could fail only in direction k, the return period of the exceedances of R would be 50 years. Hence for the unstrengthened structure the return period must be shorter.

CONCLUSION

We conclude that, except for the case of strong positive correlations between sectorial wind speed – a case that is rarely if ever encountered in nature, – designs based on the sectorial design criterion underestimate the 50-year wind-induced effects, and are therefore unconservative (on the unsafe side). Results of calculations based on Bonferroni bounds (Simiu et al., 1985, and Simiu, Leigh, and Nolan, 1986) are consistent with this conclusion. However, owing to combinatorial explosion issues those calculations could not be conducted to the degree of usefulness rendered possible by current computational capabilities. We believe similar calculations should be performed in the future by using such capabilities. Pending such calculations, the assumption of independence among sectorial wind speeds provides a lower bound of the actual mean return period of interest.

A rigorous estimation of probabilities $F_Q(Q \leq R)$ by reducing the multidirectional problem to a one-dimensional problem was described by Rigato, Chang, and Simiu (2001) for structures with no dynamic amplification effects. A similar solution applicable for structures exhibiting dynamic effects is in progress.

References

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Appendix D
SOM Project 2, Progress Report No. 3, WTC Wind Load Estimates

SOM

NIST – World Trade Center Investigation

PROJECT 2: Baseline Structural Performance and Aircraft Impact Damage Analysis

Progress Report No. 3
WTC Wind Load Estimates

Outside Experts for Baseline Structural Performance

13 April 2004

Skidmore, Owings & Merrill LLP

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1.0 Table of Contents

2.0 Overview

2.1 Project Overview

The objectives for Project 2 of the WTC Investigation include the development of reference structural models and design loads for the WTC Towers. These will be used to establish the baseline performance of each of the towers under design gravity and wind loading conditions. The work includes expert review of databases and baseline structural analysis models developed by others as well as the review and critique of the wind loading criteria developed by NIST.

2.2 Report Overview

This report covers work on the development of wind loadings associated with Project 2. This task involves the review of wind loading recommendations developed by NIST for use in structural analysis computer models. The NIST recommendations are derived from wind tunnel testing/wind engineering reports developed by independent wind engineering consultants in support of insurance litigation concerning the WTC towers. The reports were provided voluntarily to NIST by the parties to the insurance litigation.

As the third party outside experts assigned to this Project, SOM's role during this task was to review and critique the NIST developed wind loading criteria for use in computer analysis models. This critique was based on a review of documents provided by NIST, specifically the wind tunnel/wind engineering reports and associated correspondence from independent wind engineering consultants and the resulting interpretation and recommendations developed by NIST.

3.0 NIST-Supplied Documents

3.1 Rowan Williams Davies Irwin (RWDI) Wind Tunnel Reports

Final Report
Wind-Induced Structural Responses
World Trade Center – Tower 1
New York, New York
Project Number: 02-1310A
October 4, 2002

Final Report
Wind-Induced Structural Responses
World Trade Center – Tower 2
New York, New York
Project Number:02-1310B
October 4, 2002

3.2 Cermak Peterka Petersen, Inc. (CPP) Wind Tunnel Report

Data Report
Wind-Tunnel Tests – World Trade Center
New York, NY
CPP Project 02-2420
August 2002

3.3 Correspondence

Letter dated October 2, 2002
From: Peter Irwin/RWDI
To: Matthys Levy/Weidlinger Associates
Re: Peer Review of Wind Tunnel Tests
World Trade Center
RWDI Reference #02-1310

Weidlinger Associates Memorandum dated March 19, 2003
From: Andrew Cheung
To: Najib Abboud
Re: ERRATA to WAI Rebuttal Report

Letter dated September 12, 2003
From: Najib N. Abboud/Hart-Weidlinger
To: S. Shyam Sunder and Fahim Sadek (sic)/NIST
Re: Responses to NIST's Questions on:
"*Wind-Induced Structural Responses, World Trade Center,* Project Number 02-1310A and 02-1310B October 2002, By RWDI, Prepared for Hart-Weidlinger"

Letter dated April 6, 2004
From: Najib N. Abboud /Weidlinger Associates
To: Fahim Sadek and Emil Simiu
Re: Response to NIST's question dated March 30, 2004 regarding "Final Report, Wind-Induced Structural Responses, World Trade Center – Tower 2, RWDI, Oct 4, 2002"

3.4 NIST Report

Estimates of Wind Loads on the WTC Towers
Emil Simiu and Fahim Sadek
April 7, 2004

4.0 Discussion and Comments

4.1 General

This report covers a review and critique of the NIST recommended wind loads derived from wind load estimates provided by two independent private sector wind engineering groups, RWDI and CPP. These wind engineering groups performed wind tunnel testing and wind engineering calculations for various private sector parties involved in insurance litigation concerning the destroyed WTC Towers in New York. There are substantial disparities (greater than 40%) in the predictions of base shears and base overturning moments between the RWDI and CPP wind reports. NIST has attempted to reconcile these differences and provide wind loads to be used for the baseline structural analysis.

4.2 Wind Tunnel Reports and Wind Engineering

The CPP estimated wind base moments far exceed the RWDI estimates. These differences far exceed SOM's experience in wind force estimates for a particular building by independent wind tunnel groups.

In an attempt to understand the basis of the discrepancies, NIST performed a critique of the reports. Because the wind tunnel reports only summarize the wind tunnel test data and wind engineering calculations, precise evaluations are not possible with the provided information. For this reason, NIST was only able to approximately evaluate the differences. NIST was able to numerically estimate some corrections to the CPP report but was only able to make some qualitative assessments of the RWDI report. **It is important to note that wind engineering is an emerging technology and there is not consensus on certain aspects of current practice.** Such aspects include the correlation of wind tunnel tests to full-scale (building) behavior, methods and computational details of treating local statistical (historical) wind data in overall predictions of structural response, and types of suitable aeroelastic models for extremely tall and slender structures. It is unlikely that the two wind engineering groups involved with the WTC assessment would agree with NIST in all aspects of its critique. This presumptive disagreement should not be seen as a negative, but reflects the state of wind tunnel practice. It is to be expected that well-qualified experts will respectfully disagree with each other in a field as complex as wind engineering.

SOM's review of the NIST report and the referenced wind tunnel reports and correspondence has only involved discussions with NIST; it did not involve direct communication with either CPP or RWDI. SOM has called upon its experience with wind tunnel testing on numerous tall building projects in developing the following comments.

4.2.1 CPP Wind Tunnel Report

The NIST critique of the CPP report is focused on two issues: a potential overestimation of the wind speed and an underestimation of load resulting from the method used for integrating the wind tunnel data with climatic data. NIST made an independent estimate of the wind speeds for a 720-year return period. These more rare wind events are dominated by hurricanes that are reported by rather broad directional sectors (22.5 degree). The critical direction for the towers is from the azimuth direction of 205 to 210 degrees. This wind direction is directly against the nominal "south" face of the towers (the plan north of the site is rotated approximately 30 degrees from the true north) and generates dominant cross-wind excitation from vortex shedding. The nearest sector data are centered on azimuth 202.5 (SSW) and 225 (SW). There is a substantial drop (12%) in the NIST wind velocity from the SSW sector to the SW sector. The change in velocity with direction is less dramatic in the CCP 720-year velocities or in the ARA hurricane wind roses included in the RWDI report. This sensitivity to directionality is a cause for concern in trying to estimate a wind speed for a particular direction. However, it should be noted that the magnitude of the NIST interpolated estimated velocity for the 210 azimuth direction is similar to the ARA wind rose. The reduction of forces has been estimated by NIST based on a square of the velocity, however, a power of 2.3 may be appropriate based on a comparison of the CPP 50-year (nominal) and 720-year base moments and velocities.

The NIST critique of the CPP use of sector by sector approach of integrating wind tunnel and climatic data is fairly compelling. The likelihood of some degree of underestimation is high but SOM is not able to verify the magnitude of error (15%) which is estimated by NIST. This estimate would need to be verified by future research, as noted by NIST.

4.2.2 RWDI Wind Tunnel Report

The NIST critique of RWDI has raised some issues but has not directly estimated the effects. These concerns are related to the wind velocity profiles with height used for hurricanes and the method used for up-crossing.

NIST questioned the profile used for hurricanes and had an exchange of correspondence with RWDI. While RWDI's written response is not sufficiently quantified to permit a precise evaluation of NIST's concerns, significant numerical corroboration on this issue may be found in the April 6 letter (Question 2) from N. Abboud (Weidlinger Associates) to F. Sadek and E. Simiu (NIST).

NIST is also concerned about RWDI's up-crossing method used for integrating wind tunnel test data and climatic data. This method is computationally complex and verification is not possible because sufficient details of the method used to estimate the return period of extreme events are not provided.

4.2.3 Building Period used in Wind Tunnel Reports

SOM noted that both wind tunnel reports use fundamental periods of vibrations that exceed those measured in the actual (north tower) buildings. The calculation of building periods are at best approximate and generally underestimate the stiffness of a building thus overestimating the building period. The wind load estimates for the WTC towers are sensitive to the periods of vibration and often increase with increased period as demonstrated by a comparison of the RWDI base moments with and without P-Delta effects. Although SOM generally recommends tall building design and analysis be based on P-Delta effects, in this case even the first order period analysis (without P-Delta) exceeds the actual measurements. It would have been desirable for both RWDI and CPP to have used the measured building periods.

4.2.4 NYCBC Wind Speed

SOM recommends that the wind velocity based on a climatic study or ASCE 7-02 wind velocity be used in lieu of the New York City Building Code (NYCBC) wind velocity. The NYCBC wind velocity testing approach does not permit hurricanes to be accommodated by wind tunnel testing as intended by earlier ASCE 7 fastest mile versions because it is based on a method that used an importance factor to correct 50-year wind speeds for hurricanes. Because the estimated wind forces are not multiplied by an importance factor, this hurricane correction is incorporated in analytical methods of determining wind forces but is lost in the wind tunnel testing approach of determining wind forces.

4.2.5 Incorporating Wind Tunnel Results in Structural Evaluations

It is expected that ASCE 7 load factors will also be used for member forces for evaluating the WTC towers. Unfortunately, the use of ASCE 7 with wind tunnel-produced loadings is not straight forward. Neither wind tunnel report gives guidance on how to use the provided forces with ASCE 7 load factors.

The ASCE 7 load factors are applied to the nominal wind forces and, according to the ASCE 7 commentary, are intended to scale these lower forces up to wind forces associated with long return period wind speeds. The approach of taking 500-year return period wind speeds and dividing the speeds by the square root of 1.5 to create a nominal design wind speed; determining the building forces from these reduced nominal design wind speeds; and then magnifying these forces by a load factor (often 1.6) is, at best, convoluted. For a building that is as aerodynamically active as the WTC, an approach of directly determining the forces at the higher long return period wind speeds would be preferred. The CPP data did provide the building forces for their estimates of both 720-years (a load factor of 1.6) and the reduced nominal design wind speeds. A comparison of the wind forces demonstrates the potential error in using nominal wind speeds in lieu of directly using the underlying long period wind speeds.

It should also be noted that the analytical method of calculating wind forces in ASCE 7 provides an importance factor of 1.15 for buildings such as the WTC in order to provide more conservative designs for buildings with high occupancies. Unfortunately, no similar clear guidance is provided for high occupancy buildings where the wind loads are determined by wind tunnel testing. Utilizing methods provided in the ASCE 7 Commentary would suggest that a return period of 1800 years with wind tunnel-derived loads would be comparable to the ASCE 7 analytical approach to determining wind loads for a high occupancy building.

It would be appropriate for the wind tunnel private sector laboratories or NIST, as future research beyond the scope of this project, to address how to incorporate wind tunnel loadings into an ASCE 7-based design.

4.2.6 Summary

The NIST review is critical of both the CPP and RWDI wind tunnel reports. It finds substantive errors in the CPP approach and questions some of the methodology used by RWDI. It should be noted that boundary layer wind tunnel testing and wind engineering is still a developing branch of engineering and there is not industry-wide consensus on all aspects of the practice. For this reason, some level of disagreement is to be expected.

Determining the design wind loads is only a portion of the difficulty. As a topic of future research beyond the scope of this project, NIST or wind tunnel private sector laboratories should investigate how to incorporate these wind tunnel-derived results with the ASCE 7 Load Factors.

4.3 NIST Recommended Wind Loads

NIST recommends a wind load that is between the RWDI and CPP estimates. The NIST recommended values are approximately 83% of the CPP estimates and 115% of the RWDI estimates. SOM appreciates the need for NIST to reconcile the disparate wind tunnel results. It is often that engineering estimates must be done with less than the desired level of information. In the absence of a wind tunnel testing and wind engineering done to NIST specifications, NIST has taken a reasonable approach to estimate appropriate values to be used in the WTC study. However, SOM is not able to independently confirm the precise values developed by NIST.

The wind loads are to be used in the evaluation of the WTC structure. It is therefore recommended that NIST provide clear guidelines on what standards are used in the evaluations and how they are to incorporate the provided wind loads.

5.0 References

[1] American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures*, ANSI/ASCE 7-02, 2002.

[2] American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures*, ANSI/ASCE 7-93, 1993.

APPENDIX E
STILL IMAGES OF THE VIDEO RECORDS USED IN CHAPTER 6

This appendix provides still images of the video records (Figures E–1 through E–9) used to estimate the initial impact conditions of the aircraft that impacted World Trade Center (WTC) 1 and WTC 2 (see Chapter 6). A short description of each of these videos is provided in Table 6–1.

Figure E–1. Still image from Video V1 (WTC 1 impact).

Figure E–2. Still image from Video V2 (WTC 1 impact).

Figure E–3. Still image from Video V3 (WTC 2 impact).

Figure E–4. Still image from Video V4 (WTC 2 impact).

Figure E–5. Still image from Video V5 (WTC 2 impact).

Figure E–6. Still image from Video V6 (WTC 2 impact).

Figure E–7. Still image from Video V7 (WTC 2 impact).

Figure E–8. Still image from Video V8 (WTC 2 impact).

Figure E–9. Still image from Video V9 (WTC 2 impact).

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