High-Rise Reinforced Concrete Structures: Database-Assisted Design for Wind

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Abstract: Advances in wind pressure measurement and computer technology have made time-domain analyses of wind effects on high-rise structures possible in recent years. Time-domain solutions use aerodynamic and wind climatological databases and provide full phase information on wind-induced response that is lost in the frequency-domain approach; therefore, they can account rigorously for the superposed effects of any number of modes of vibration of any shape; for mode coupling; for wind directionality effects; and for the joint contributions of axial forces, bending moments, and shear forces in interaction equations used for structural design. Unlike the frequency-domain approach, in the time-domain approach, the process of determining wind effects and the structural design process, referred to jointly as database-assisted design (DAD), are integrated, transparent, and fully auditable. The objective of this study is to present the DAD approach as applied to high-rise reinforced concrete (RC) buildings. Given the time histories of pressures, measured in the wind tunnel at a sufficient number of taps on the exterior faces of the building envelope for a sufficient number of mean speed directions and a preliminary design of the building, the structural engineer can calculate, as functions of wind speed and direction: (1) demand-to-capacity indexes for any number of members and cross sections, (2) interstory drift, (3) and top floor accelerations. These responses are properties of the structure independent of the wind climate, and constitute response databases used in conjunction with a wind climatological database to obtain the requisite wind effects for any specified mean recurrence interval. The design, which accounts for both wind and gravity effects, is performed iteratively until the design specifications are satisfied. **DOI: 10.1061/(ASCE)ST.1943-541X.0000394.**

CE Database subject headings: Databases; Reinforced concrete; Wind pressure; High-rise buildings; Measurement; Computer applications.

Author keywords: Database-assisted design (DAD); Mean recurrence interval; Reinforced concrete; Time-domain analysis; Wind effects.

Introduction

The ASCE 7-05 Standard (ASCE 2005) specifies two types of methods for determining wind loads: analytical methods, and the wind tunnel method. Analytical methods exclude buildings subjected to across-wind effects. Therefore, insofar as high-rise buildings typically experience such effects, the analytical procedure is usable only for preliminary design purposes. The wind tunnel method is specified in ASCE 7-05 only in very general terms. This is one of the reasons why estimates of wind effects can vary among independent laboratories or even within the same laboratory by as much as 50% or more [Coffman et al. 2010; Fritz et al. 2008; SOM 2004 (also accessible in Simiu 2011, Appendix A5)].

The dynamic response of buildings to wind is estimated on the basis of wind tunnel data by using either (1) an approach that uses measurements of strains in a high-frequency force balance (HFFB) at the base of a rigid model or in the spine of an aeroelastic model, or (2) a time-domain approach by using simultaneous pressure time histories on a rigid model (Simiu et al. 2008) or an aeroelastic model (Diana et al. 2009). Time-domain solutions use aerodynamic and wind climatological databases and provide full phase information on wind-induced response that is lost in the frequency-domain approach; therefore, they can account rigorously for the superposed effects of any number of modes of vibration of any shape, for mode coupling, for wind directionality, and for the joint contributions of axial forces, bending moments, and shear forces in interaction equations used for structural design. Unlike for the frequency-domain approach, the process of determining wind effects and the structural design process, referred to jointly as database-assisted design (DAD), are integrated, transparent, and fully auditable.

The DAD approach, as applied in this report, has been developed with a view to exploit fully the potential of time-domain approaches (Simiu et al. 2008; Spence 2009). Note the use of the term "design" in this designation. DAD is not aimed merely at providing the structural engineer with wind loads because of spatially averaged pressures. Rather, DAD is an integrated design methodology that includes member sizing. The member sizing is dictated by (1) the building's aerodynamic and structural properties, (2) the structure's wind environment and its directional interaction with those properties, and (3) the design criteria for strength and serviceability. The DAD approach enables the automation of the design of individual structural members of buildings, whether they do or do not experience aeroelastic effects. This study is limited to the case in which aeroelastic effects are not significant. Research on the application of DAD to buildings with significant aeroelastic effects is planned for a future study.

The DAD approach allows a clear separation of the wind engineer's and structural engineer's tasks. The wind engineer's task is to produce (1) the requisite pressure time histories from wind tunnel testing or, as is likely to be the case in the future, from CFD (computational fluid dynamics) simulations, and (2) directional wind

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Note. This manuscript was submitted on July 1, 2010; approved on February 10, 2011; published online on February 12, 2011. Discussion period open until April 1, 2012; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Structural Engineering***, Vol. 137, No. 11, November 1, 2011. ©ASCE, ISSN 0733-9445/2011/11-1340–1349/$25.00.

speeds for standard micrometeorological conditions, recorded at a weather station reasonably representative of the wind climate near the building site and/or developed by, e.g., Monte Carlo simulations. In addition, the wind engineer must produce (3) the ratio between those directional wind speeds and their counterparts at the top of the building. This ratio enables the transformation of wind tunnel pressure measurements into prototype pressures on the building envelope. Once the preceding items (1), (2), and (3) are produced by the wind engineer, the structural engineer can use them for quasi-static or dynamic analyses and for accurately determining individual member demand-to-capacity indexes, interstory drift, and top floor accelerations, corresponding to any specified mean recurrence interval (MRI). The demand-to-capacity index is an indicator of structural strength and adequacy. It incorporates relevant ACI 318 (ACI 2008) and ASCE 7 Standard requirements (ASCE 7-05, 7-10; ASCE 2005, 2010).

The objective of this paper is to develop and apply the DAD approach to reinforced concrete high-rise buildings. The DAD approach has, in our opinion, marked advantages from the point of view of physical modeling, accuracy, and convenience to the designer. In addition, the approach is transparent, meaning that it can be followed and understood by structural engineers and public officials charged with verifying structural calculations, including their wind engineering components. That wind and structural engineering approaches should satisfy the requirement of transparency would seem obvious. However, as was pointed out by SOM (2004), this requirement is not currently met satisfactorily by conventional approaches. The database-assisted design approach has been developed with this requirement in mind.

Description of DAD Procedure

The DAD approach as applied to high-rise buildings entails the phases represented in Fig. 1. The processes within the dotted box constitute the main algorithm of the High-Rise Database-Assisted Design for Reinforced Concrete structures (HR_DAD_RC) software (Yeo 2010). The processes outside the box describe information provided by the wind engineer and the structural engineer. Their tasks are clearly separated in the DAD methodology. The yellow (bright) blocks and the blue (dark) blocks correspond to the wind engineer's and the structural engineer's tasks, respectively. The DAD procedure consists of the following phases:

1. A preliminary design on the basis of wind speeds specified in the relevant code or provided by the wind engineering consultant is performed by the structural engineer, for example by using the algorithm of ASCE 7-05, (ASCE 2005; Section 6.5). This yields an *initial set of building member dimensions*. The fundamental natural frequencies of vibration for the preliminary design can be obtained by modal analysis by using a finite-element analysis (FEA) program. The damping ratios are specified by the structural engineer.
2. Dynamic analyses of the building with the member dimensions determined in Phase 1 employ combinations of gravity and wind loads specified in ASCE 7-05, Section 2.3. These combinations can be easily modified if the standard requirements change. The analyses are performed by considering the resultant of the wind forces at each floor's mass center, for each wind direction and for reference mean hourly wind speeds at the top of the building of, say, $20, 30, \ldots, 80$ m/s, depending upon the wind speed range of interest at the building location. This phase of the procedure is performed by the structural engineer by using as input the directional aerodynamic pressures database provided by the wind engineer. The outputs of this phase are the floor displacements, floor accelerations, and effective (aerodynamic plus inertial) lateral forces at each floor corresponding to the specified set of directional mean hourly speeds at the top of the building (e.g., $20, 30, \ldots, 80$ m/s).
3. The *influence coefficients*, which yield the internal forces in any member, and the displacements and accelerations of interest, attributed to unit loads with specified directions acting at the mass center of any floor, are calculated by the structural engineer.
4. For each direction and specified wind speed, internal forces induced in members are calculated using the influence coefficients (Phase 3) multiplied by the effective floor loads at mass centers (Phase 2). The wind-induced forces are added to the respective internal forces induced by factored gravity loads (by using the gravity load factor specified in ASCE 7-05, Section 2.3). Demand-to-capacity indexes indicating the extent to which a member is or is not safe are then calculated (see the Appendix). The output of this phase is a *response database* providing the demand-to-capacity index for the structural members, the interstory drift along the building height, and the top floor accelerations. The response database is a property of the structure that incorporates its aerodynamic and mechanical characteristics and is independent of the wind climate.
5. A matrix of directional wind speeds at 10 m above ground in open exposure (i.e., a *wind climatological database*) is developed for a location close to a building of interest. Where necessary, a sufficiently large matrix of wind speeds for example, each of 36 or 16 directions to be considered is developed from measured or simulated wind speed data by using the procedure described by Grigoriu (2009). Each row of the matrix corresponds to one storm event (if a peaks-over-threshold estimation procedure is used) or to the largest yearly speed (if an epochal estimation procedure is used). The columns of the matrix correspond to the specified wind directions. For hurricane winds, a similar matrix of wind speeds is used. The directional wind speed matrix is provided by the wind engineering consultant. By using micrometeorological relations, wind tunnel data, or CFD data, the wind engineer also provides a counterpart to this matrix, containing the directional mean hourly wind speeds at the top of the building, in lieu of the directional wind speeds at 10 m above ground in open exposure.
6. By using interpolation procedures, the response database (Phase 4) is used in conjunction with the directional wind speed matrix containing the directional hourly mean speeds (Phase 5) to calculate a matrix containing the response of interest for each direction of each storm event (or year). However, for each storm event (or year), only the largest of the directional responses is of interest from a design viewpoint and is therefore retained. A one-dimensional vector of the maximum response induced by each storm event is thus created. This vector is then rank-ordered, and the peak responses corresponding to the required mean recurrence intervals are obtained by using nonparametric estimation methods (Phase 6, e.g., Simiu and Miyata 2006, p. 33). Note that the peak response of interest can consist of the demand-to-capacity indexes for any member, the interstory drift, and the peak acceleration for the respective specified MRIs.
7. The procedure outlined in the preceding Phases is repeated as needed until the results obtained satisfy the design criteria.

Design of a Commonwealth Advisory Aeronautical Research Council Building

A 60-story reinforced concrete building with rigid diaphragm floors (Fig. 2) was designed by using the HR_DAD_RC software. The

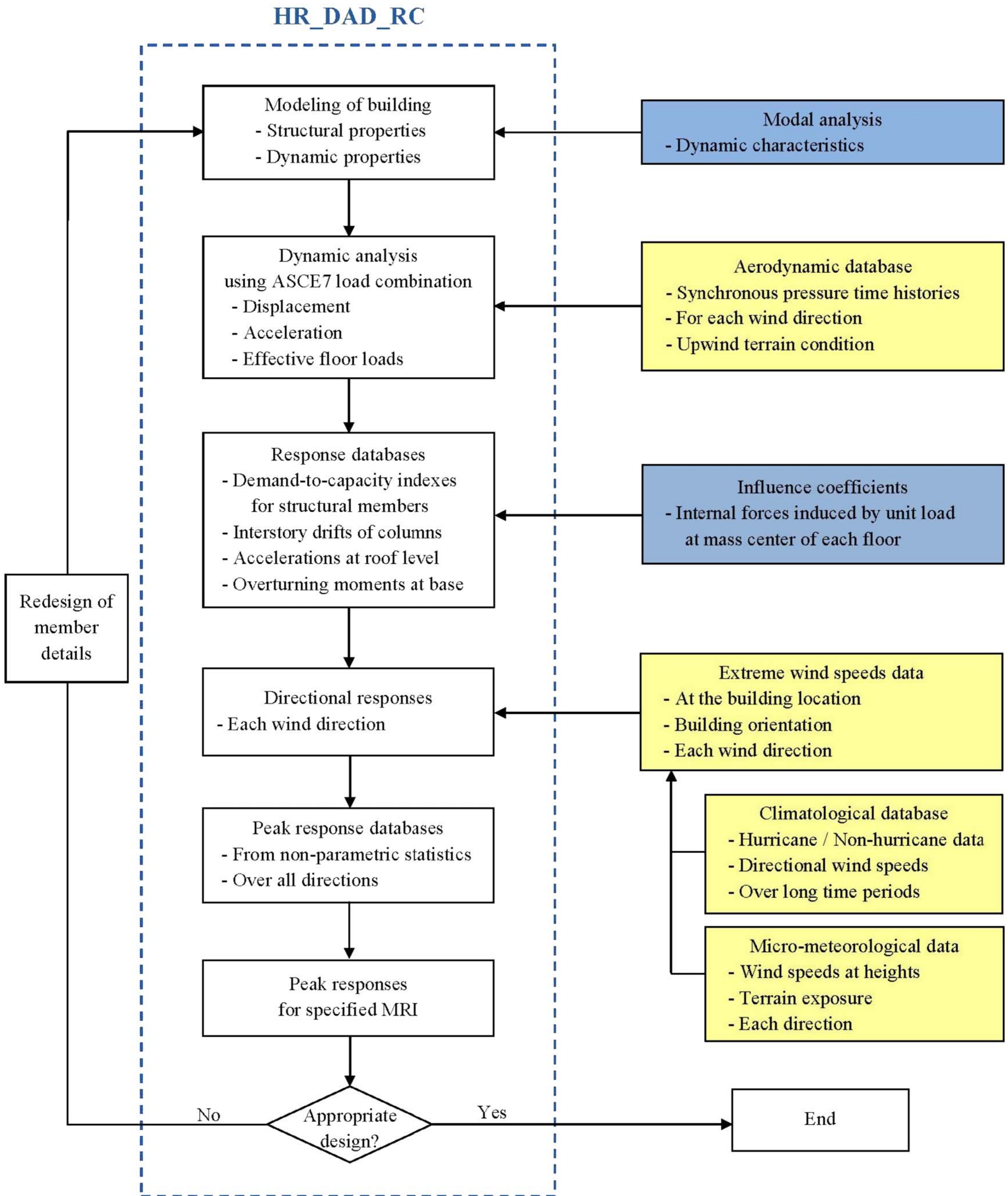


Fig. 1. HR_DAD_RC procedure

dimensions of the building are 45.72 m in width (B), 30.48 m in depth (D), and 182.88 m in height (H), and define the Commonwealth Advisory Aeronautical Research Council (CAARC) building studied by various researchers (Melbourne 1980; Venanzi 2005; Wardlaw and Moss 1971). The building has a moment-resisting frame structural system similar to the structural system studied by Teshigawara (2001) and consists of 2,880 columns and 4,920 beams, in addition to rigid diaphragm slabs. The building was assumed to be located near Miami, Florida, and to have suburban exposure.

Modeling of the Building

The building was first designed by using the algorithm of ASCE 7-05 (ASCE 2005; Section 6.5). Once initial dimensions of members in the building were obtained, natural frequencies of vibration and mode shapes were calculated by modal analysis by using a finite-element analysis program. The modal damping ratios were assumed to be 2% in all three modes considered in this study (Table 1).

As shown in Figs. 2 and 3, structural members of the building consist of columns, beams, and slabs. (The design of the slabs was not performed in this study.) Columns were categorized as corner and noncorner columns. Beam members were divided into exterior (spandrel) and interior beams. The building consists of six sets of members. Each set consists of 10 stories for which the member dimensions are the same. The first set applies to the first 10 stories, the second to the next 10 stories, and so forth. The compressive strengths of concrete for all members are 80 MPa from the 1st to the 40th story and 60 MPa from the 41st to the 60th story. The dimensions and reinforcement details are described in Yeo (2010). Columns have longitudinal reinforcement uniformly distributed

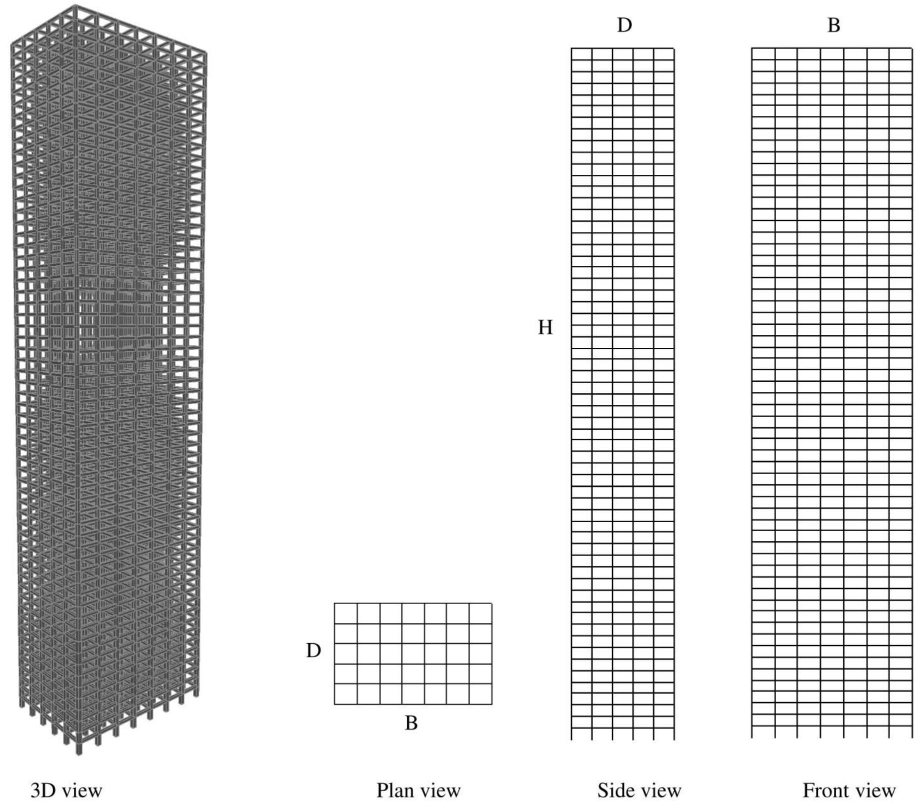


Fig. 2. Schematic views of a 60-story building

Table 1. Dynamic Properties of a Building

Mode	1st (y-direction)	2nd (x-direction)	3rd (θ-direction)
Natural frequency (Hz)	0.165	0.175	0.200
Damping ratio (%)	2.0	2.0	2.0

along the sides and hoops; beams have tensile and compression reinforcement and stirrups. The yield strengths of the reinforcements are 520 MPa for longitudinal bars and 420 MPa for hoops or stirrups. Wind effects were calculated for a typical set of 96 members (Fig. 3).

Dynamic Analysis by Using Aerodynamic Database

Time histories of aerodynamic wind loads on each floor were calculated from time series of pressures on a rigid wind tunnel model of the CAARC building, measured for winds with directions with 10° increments for suburban terrain exposure. The wind tunnel tests were performed at the Inter-University Research Center on Building Aerodynamics and Wind Engineering (CRIACIV-DIC) boundary layer wind tunnel in Prato, Italy (Venanzi 2005). For buildings sensitive to aeroelastic phenomena, synchronous pressures must also be measured on an aeroelastic model under a range of wind speeds and directions for which aeroelastic responses occur (Diana et al. 2009). However, in this study, aeroelastic effects are assumed not to be present.

Dynamic analyses of the building were performed by using wind loads corresponding to wind speeds of 20–80 m/s in increments of 10 m/s, by using the directional pressure data obtained from the wind tunnel tests for suburban terrain exposure. Time series of displacements and accelerations at the mass center on each floor were obtained, and effective lateral loads on all floors were calculated for each wind speed and wind direction.

Two load combinations, LC1 and LC2, associated with wind and gravity loads for each wind direction and speed [Eqs. (1)], were considered for strength design, and one loading case [Eq. (2))] was considered for serviceability design

$$1.2D + 1.0L + 1.0W,\ (\mathrm{LC1}) \qquad 0.9D + 1.0W,\ (\mathrm{LC2}) \tag{1}$$

$$1.0D + 1.0L + 1.0W \tag{2}$$

in which D = total dead load; L = live load; and W = wind load. The load factor is not applied to the wind load because the DAD approach accounts for structural responses under a wide range of wind speeds of interest and produces the corresponding wind effects with the requisite mean recurrence intervals.

Fig. 4 represents time histories and power spectral densities of the across-wind force F_x, the along-wind force F_y, and moment M_θ, acting on the mass center of the 41st floor and induced by 50 m/s hourly mean winds (at the building height) normal to the longer wall of the building (i.e., the front wall in Fig. 3). The loads were calculated from pressures measured at the Prato wind tunnel laboratory (Venanzi 2005). The effect of the vortex shedding on the wind loads is clearly seen in the spectral plot. Note that, for buildings for which vortex shedding frequencies are not close to natural frequencies of vibration, lock-in phenomena are not significant.

Response Database

Response databases for demand-to-capacity indexes, interstory drift, and peak acceleration were calculated for the wind directions and speeds considered in each load combination case. For strength design, the response databases for the demand-to-capacity indexes

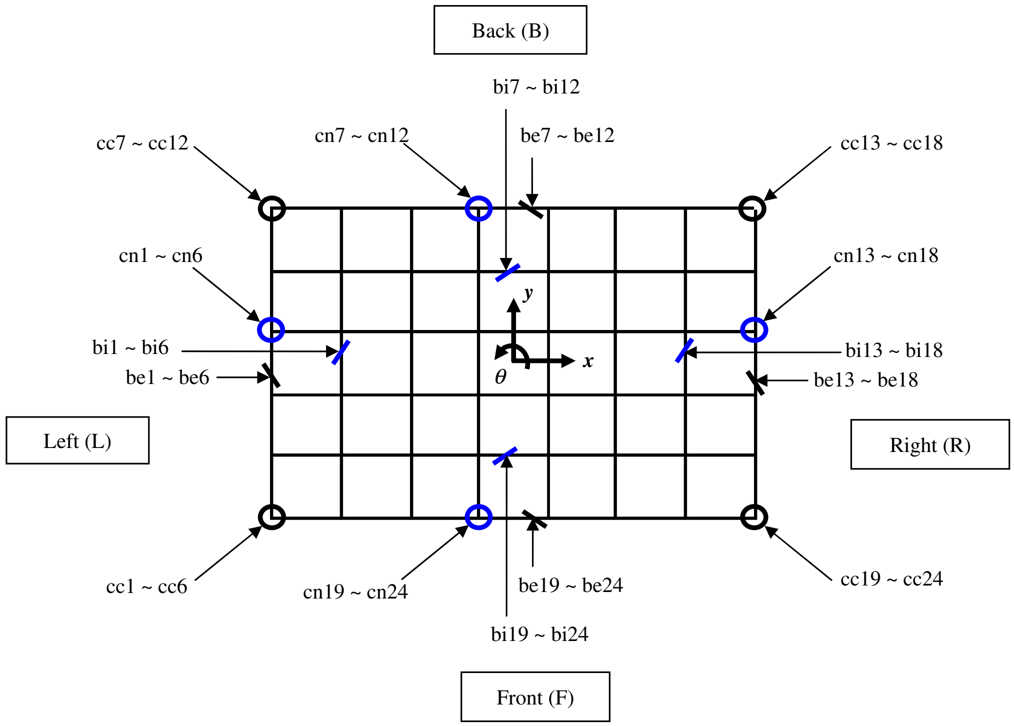


Fig. 3. Plan view of building with locations of selected members

(see appendix) were calculated for each individual member. For serviceability design, response databases for interstory drift and acceleration were calculated for a column line and a top floor corner, respectively (see Yeo 2010).

The response database of a demand-to-capacity index is shown in Fig. 5 for a corner column (cc) under LC2 in Eq. (1); θ_w is the wind direction (clockwise rotates from the principal axis of x in Fig. 3).

Directional Responses

Structural responses under wind at the building location were obtained by applying to the response databases the directional wind speeds from the climatological database near the building location. The climatological database used in the study is a dataset of 999 simulated hurricanes with wind speeds for 16 directions near Miami, Florida (Milepost 1450), and was obtained from www.nist.gov/wind. The angles indicating those directions are from 22.5° to 360° clockwise from the north in 22.5° increments. In this study, the orientation angle of the building is 90° clockwise from the north, that is, the front side of the building faces south.

This study assumed suburban terrain exposure (i.e., Exposure Category B) in all directions. DAD obtained directional responses by calculating responses corresponding to hourly mean wind speeds (m/s) and the associated directions at the rooftop in suburban terrain exposure. The wind speeds were converted from 1-min hurricane wind speeds (knots) at 10 m above ground in open terrain exposure to hourly wind speeds at the elevation of the top of the building, and then applied to the response databases, given the building orientation α_0. Veering effects (see Yeo and Simiu (2010) were not considered in this study.

The peak response database consists in each case of the respective vector of the 999 largest responses. Examples of peak response databases for LC2 are shown for demand-to-capacity indexes of a corner column of cc7 (Fig. 6), interstory drift of the front-left corner at the 43rd story (Fig. 7), and peak accelerations of the front-left corner of the top floor (Fig. 8). The figures show that the peak responses increase monotonically with MRI. The peak responses of interstory drifts and accelerations along both principal axes do not occur at the same time.

Adjustment of Demand-to-Capacity Indexes

As an option, DAD accounts for the ASCE 7-05 design requirement that forces and pressures estimated through wind tunnel testing are to be limited to not less than 80% of its ASCE 7-based counterpart (see ASCE 2005; Section C6.6). If the moments in DAD are less than 80% of those determined in accordance with Section 6.5 of ASCE 7-05, the demand-to-capacity index is adjusted as follows:

$$B^*_{ij} = \gamma B_{ij} \qquad \gamma = \frac{0.8}{M_o^{\mathrm{DAD}}/M_o^{\mathrm{ASCE7}}} \tag{3}$$

in which M_o^{DAD} and M_o^{ASCE7} = overturning moments obtained from DAD and Section 6.5, ASCE 7-05, respectively; and γ = index adjustment coefficient. If the moment in DAD is not less than 80% of the ASCE 7-05 value, the index need not be modified (i.e., $B^*_{ij} = B_{ij}$). (For material on the demand-to-capacity indexes, see the appendix.)

This study calculated ASCE 7-based overturning moments along the principal axes (i.e., x and y axes) for Risk Category "II" and "III or IV" and compared them with the peak overturning moments determined by the DAD procedure for MRI = 700 years and 1,700 years, respectively.

As shown in Table 2, ratios of overturning moments from DAD to those from ASCE 7 are less than 0.8 for both MRIs; the corresponding index adjustment factors γ [Eq. (3)] are 1.16 and 1.19, respectively. Adjusted peak demand-to-capacity indexes for both MRIs were obtained by multiplying the indexes by the adjustment factors.

Peak Response for Specific MRIs

Peak responses were obtained for adjusted demand-to-capacity indexes corresponding to the MRIs of 700 and 1,700 years specified

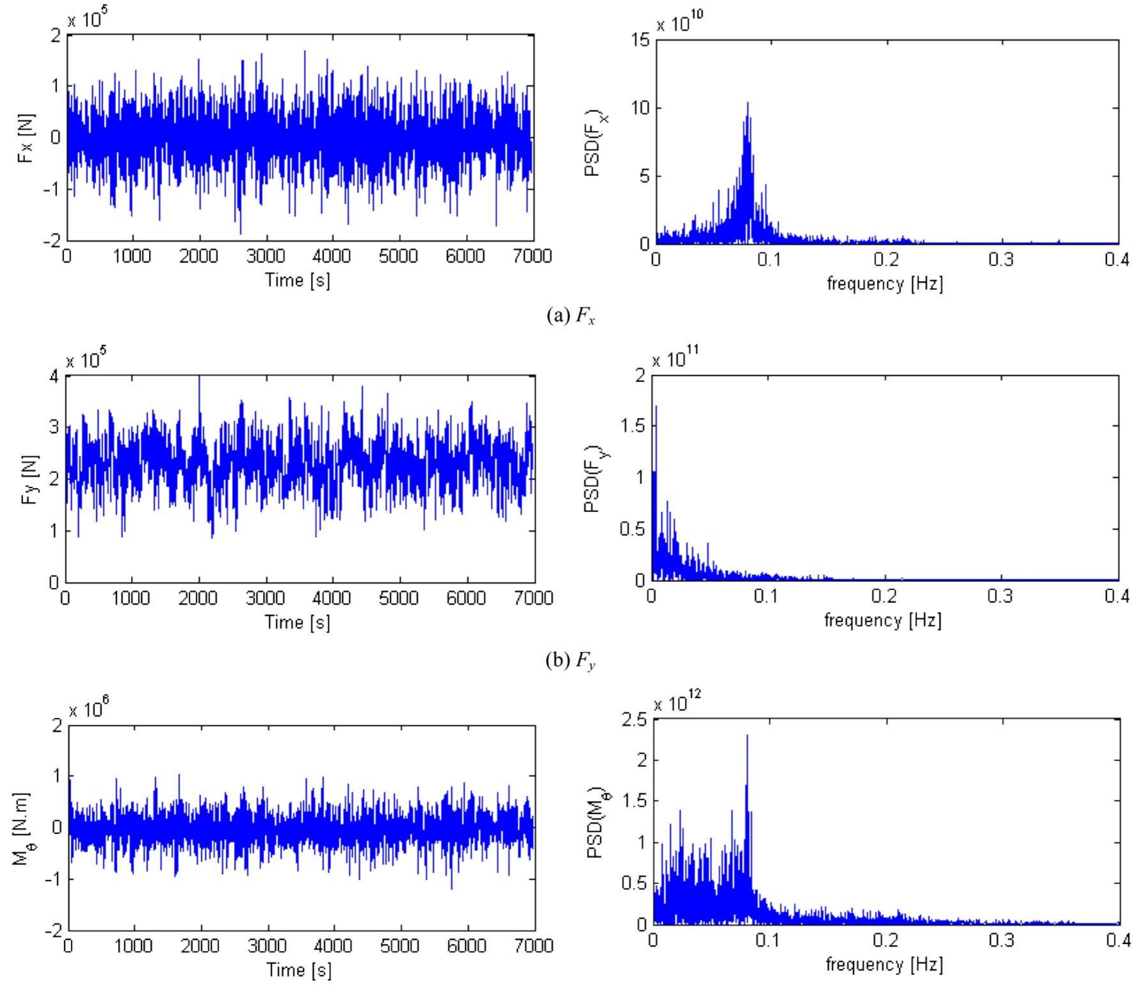


Fig. 4. Wind loads (time histories and frequency distribution)

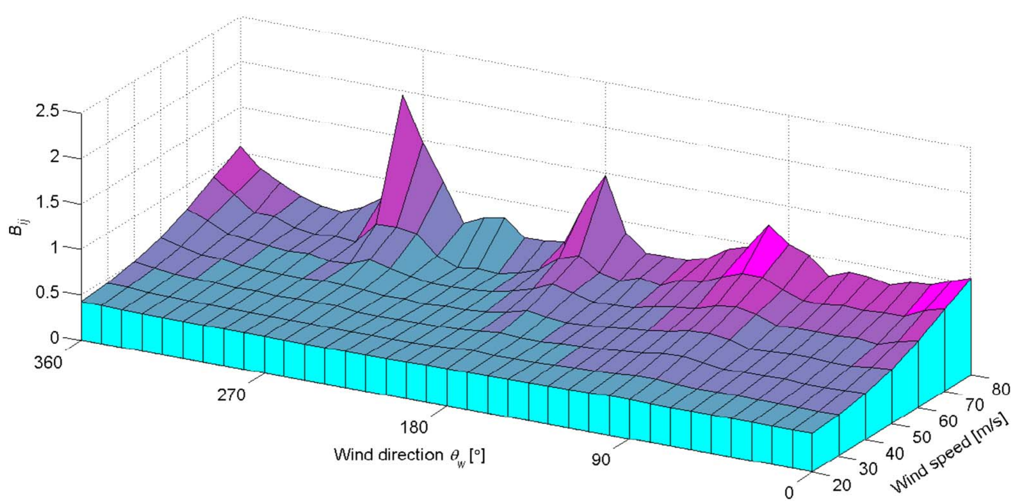


Fig. 5. Response database: demand-to-capacity index B_{ij}^{PM} (member ID = cc7)

in the ASCE 7-10 Standard as a function of risk category (ASCE 2010). Maximum values of peak adjusted demand-to-capacity indexes for the 96 selected members are summarized in Table 3 as functions of the load combination under consideration. For the index $B_{ij}^{PM^*}$, the load combination case LC1 governs. Significant differences between the LC1 and LC2 cases occur for the columns. In particular, the index for corner columns changes noticeably if the MRI increases from 700–1,700 years. This is attributed to the fact that lower axial compression forces reduce the flexural strength of a column with a tension-controlled section. For the index $B_{ij}^{VT^*}$,

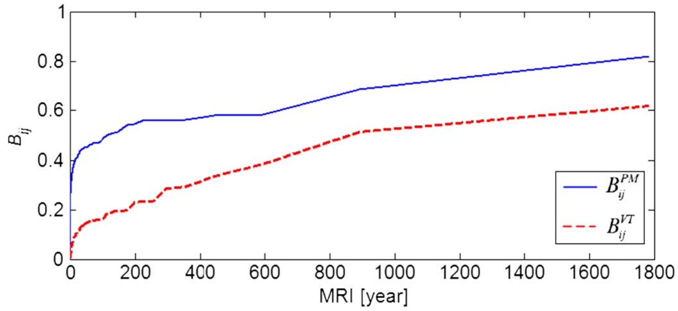


Fig. 6. Peak response database: demand-to-capacity index in LC2

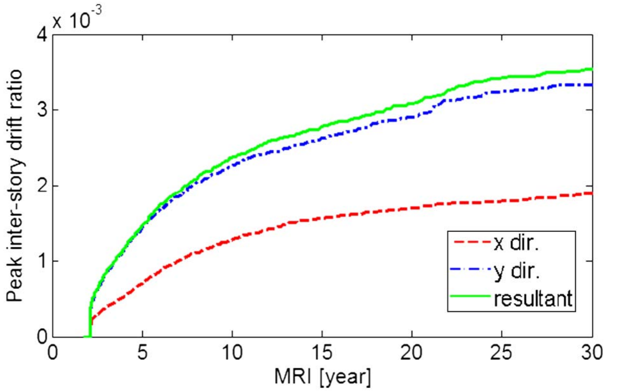


Fig. 7. Peak response database: interstory drift

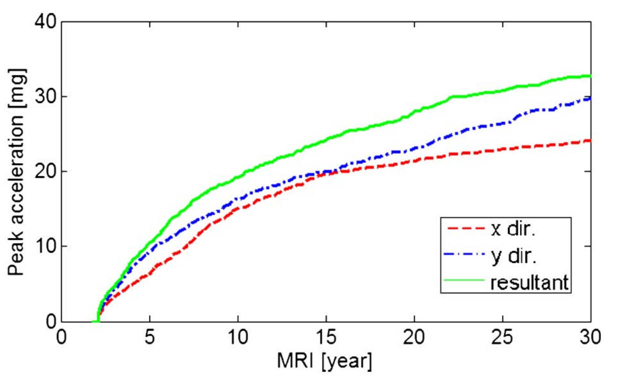


Fig. 8. Peak response database: acceleration

Table 2. Overturning Moments and Adjustment Coefficient

	MRI = 700 years		MRI = 1,700 years	
	ASCE 7	DAD	ASCE 7	DAD
$M_{ox}(\times 10^6\ \text{kN}\cdot\text{m})$	6.10	4.22	7.01	4.70
$M_{oy}(\times 10^6\ \text{kN}\cdot\text{m})$	3.36	2.64	3.87	2.86
$M_{ox}^{\text{DAD}}/M_{ox}^{\text{ASCE7}}$	0.69		0.67	
$M_{oy}^{\text{DAD}}/M_{oy}^{\text{ASCE7}}$	0.79		0.74	
γ	1.16		1.19	

LC2 governs for columns and LC1 for beams. It is notable that the increases in both indexes are generally larger for columns than for beams as the MRI changes from 700–1,700 years.

Peak interstory drift for MRI = 20 years and peak acceleration of the building for MRI = 10 years were obtained from their peak response databases (Figs. 7 and 8). Their maximum values in the two principal axes (x and y) and the associated resultant are summarized in Table 4. Interstory drift and acceleration are not modified by the adjustment factor γ.

Table 3. Adjusted Peak Demand-to-Capacity Indexes

		MRI = 700 years		MRI = 1,700 years	
		LC1	LC2	LC1	LC2
Corner column	B_{ij}^{PM*}	0.94	0.74	1.04	1.01
	B_{ij}^{VT*}	0.31	0.50	0.55	0.73
Noncorner column	B_{ij}^{PM*}	1.00	0.74	1.08	0.84
	B_{ij}^{VT*}	0.39	0.45	0.53	0.59
Exterior beam	B_{ij}^{PM*}	0.60	0.59	0.73	0.72
	B_{ij}^{VT*}	0.50	0.47	0.60	0.56
Interior beam	B_{ij}^{PM*}	0.67	0.66	0.79	0.78
	B_{ij}^{VT*}	0.64	0.60	0.75	0.70

It is of interest to assess the modal contributions to peak wind effects. These contributions were calculated by using the HR_DAD algorithm accounting for the first five modes. The first mode corresponds to drift along the x-axis, the second to drift along the y-axis, the third mode to the rotation θ, the fourth to drift along the x-axis, and the fifth mode to drift along the y-axis. For example, for the peak resultant acceleration at a building top floor corner, Fig. 9 shows peak resultant accelerations with a 10-year MRI calculated by accounting for the first mode only, for the first two modes, for the first three modes, for the first four modes, and for the first five modes. The calculations show that the ratio between the acceleration based on the first mode only and the acceleration based on the first five modes is 0.62. The ratios are 0.83, 0.93, and 0.98 for the acceleration on the basis of the first two, three, and four modes, respectively. Modes with natural frequencies higher than those of the fundamental modes have insignificant contributions in this case. This is attributable to the configuration and relatively modest height (180 m) of the building. However, for

Table 4. Peak Interstory Drifts and Peak Acceleration

	ASCE 7		DAD		
	x-direction	y-direction	x-direction	y-direction	Resultant
Interstory drift ratio ($\times 10^{-4}$)	13	23	17	29	31
Acceleration (mg)	12.7	19.5	15.0	16.4	19.6

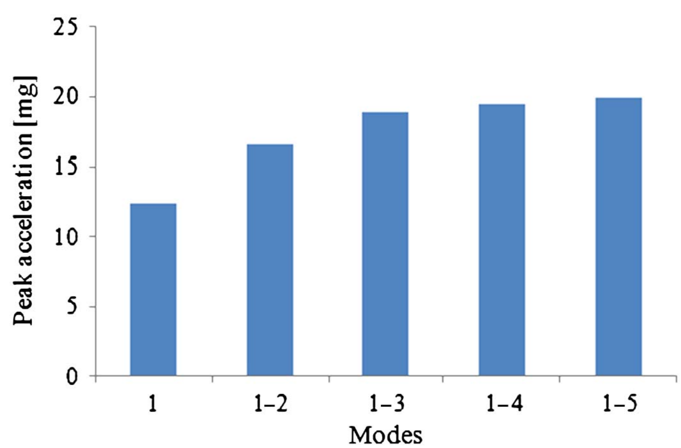


Fig. 9. Modal contribution to wind effects

buildings of 400–800-m height, the contributions of the higher modes may well be far more significant. One of the strengths of the DAD software is that the contributions of any number of modes can be accounted for in one fell swoop.

Compliance with Design Criteria

Once peak structural responses for specified MRIs are obtained, DAD verifies if the peak responses satisfy design criteria for safety and serviceability. Fig. 10 shows adjusted peak demand-to-capacity indexes accounting for ASCE 7 limitations on overturning moments. The indexes in the figure are the maxima of the load combination cases LC1 and LC2. They indicate that structural members were adequately designed for shear strength and have the capacity to resist effects of interacting shear forces and torsional moment (i.e., $B_{ij}^{VT^*} \leq 1$) corresponding to both MRIs. However, some members do not have adequate axial and flexural strengths (i.e., $B_{ij}^{PM} > 1$) for MRI $= 1{,}700$ years. (A higher-than-unity index means than the corresponding member must be redesigned to achieve stronger capacity.) The overall DAD results show that structural members used in this study were designed more conservatively at higher floor levels because the indexes typically decrease with height. Differences between peak responses corresponding to the two MRIs are member-dependent.

Table 4 shows peak interstory drift ratios for MRI $= 20$ years and peak top floor accelerations for MRI $= 10$ years. The peak interstory drift ratio based on DAD is 0.0029 in the y-direction. The ASCE 7-05 commentary suggests limits on the order of $1/600$ to $1/400$ (see ASCE 2005; Section CC.1.2). In this study, this criterion is not satisfied.

The peak top floor resultant acceleration based on DAD is 19.6 mg. This study assumed a limit of 25 mg for a 10-year MRI for office buildings (Isyumov et al. 1992). The limit is greater than the peak acceleration determined in this study. The design is therefore adequate for peak acceleration.

According to the strength criteria, the design is not adequate for the axial and flexural strengths of columns for the 1,700-year MRI. The design is also inadequate for peak interstory drift corresponding to a 10-year MRI. Therefore, the procedure outlined in Sections 5.2 to 5.6 should be repeated with a modified structural design until the corresponding results satisfy the design criteria. This iterative, trial-and-error procedure is time-consuming. Therefore, an automated optimization procedure would be needed for both economy and computational efficiency. Such a procedure, which makes use of the DAD estimates of the response, is currently under development. Provisions for P-delta effects are being added to the software.

Comparisons of DAD- and ASCE 7-Based Designs

Fig. 11 shows, for selected members, ratios of demand-to-capacity indexes based on DAD to those based on the ASCE 7 analytical method

$$R = \frac{B_{ij}^{\mathrm{DAD}} - B_{ij}^{\mathrm{ASCE7}}}{B_{ij}^{\mathrm{ASCE7}}} \qquad (4)$$

in which M_{ij}^{ASCE7} = demand-to-capacity index from ASCE 7; and B_{ij}^{DAD} = adjusted index from DAD.

The results indicate that differences between DAD- and ASCE 7-based results are significant in the column members. The ratio varies according to the individual member, owing to the stronger dependence on individual members of the index based on DAD. For columns, the ASCE 7-based results overestimate values of B_{ij}^{PM} for lower floors, e.g., by approximately 30% for corner columns in comparison with DAD-based results, but underestimate values of B_{ij}^{PM} for higher floors. ASCE 7 overestimates most B_{ij}^{VT} indexes, by up to approximately 65% for lower floors. For beams, ASCE 7 overestimates the indexes at lower floors and underestimates them at higher floors, by up to approximately 20%. These comparisons show that the ASCE 7 analytical method can result in structural members that are either stronger or weaker than the more realistically designed members based on DAD.

In addition, maximum interstory drift ratio for MRI $= 20$ years and acceleration for MRI $= 10$ years were calculated by the ASCE 7-based method (ASCE 2005, Section C6.5.8), see Table 4,

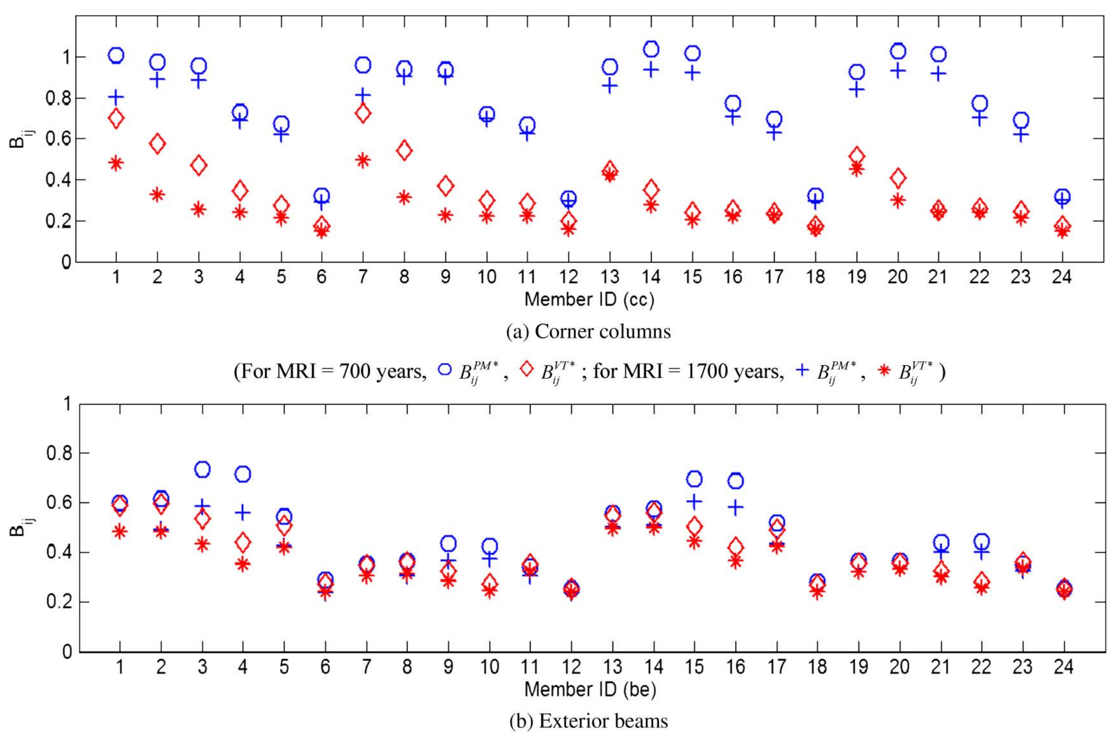
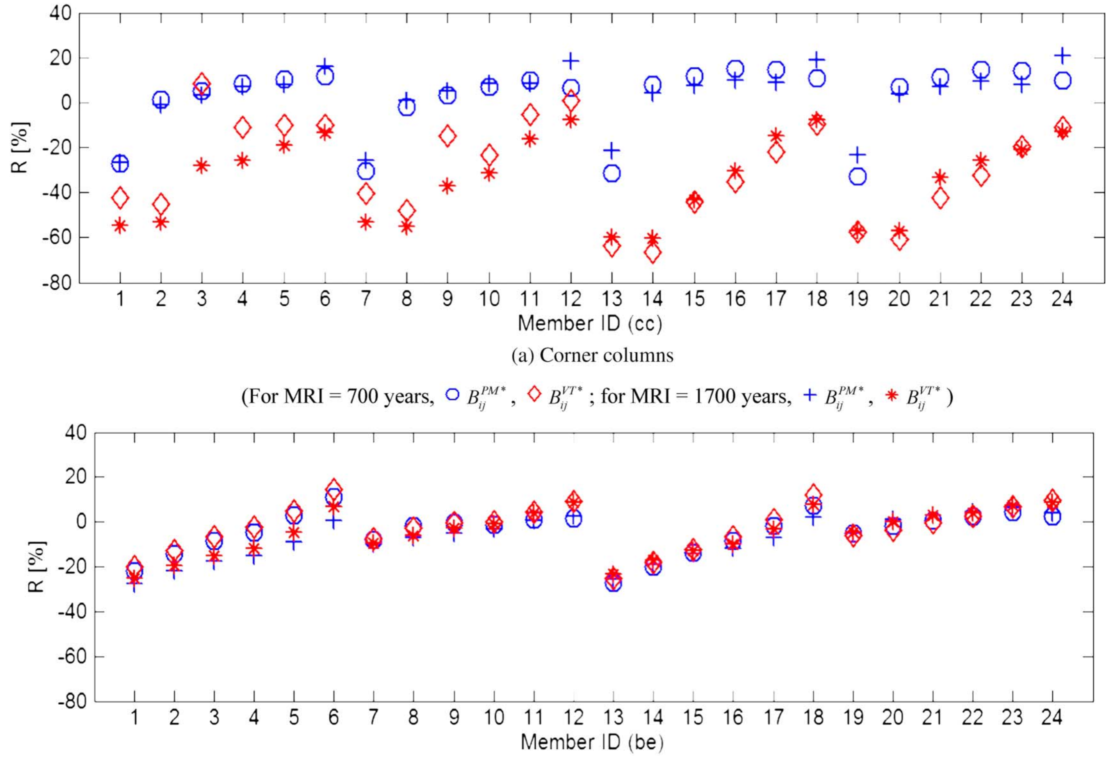


Fig. 10. Design results for MRI $= 700$ and 1,700 years (DAD)



(a) Corner columns

(For MRI = 700 years, ○ B_{ij}^{PM*}, ◇ B_{ij}^{VT*} ; for MRI = 1700 years, + B_{ij}^{PM*}, ∗ B_{ij}^{VT*})

(b) Exterior beams

Fig. 11. Ratio R [Eq. (4)]

yielding a maximum interstory drift ratio of 0.0023. This is less than the value obtained by DAD, which accounts for translational and rotational responses. The maximum acceleration is 19.5 mg in the y-direction. This is close to the 19.6 mg resultant obtained by DAD. The ASCE 7 method can yield peak interstory drift or peak acceleration values lower than those yielded by DAD, meaning that the ASCE 7 estimates, based as they are on a physically simplified model, can be unconservative. The larger DAD drift and acceleration values can be explained in part by the fact that across-wind and torsional effects are accounted for in the DAD method.

Conclusions

This report presented the development of a database-assisted design (DAD) procedure for reinforced concrete buildings, and its application to a 60-story building. The DAD procedure performs dynamic analyses by using simultaneous time series of aerodynamic pressure data obtained in the wind tunnel. It obtains displacement and acceleration time histories, effective lateral load time histories at the mass center at each floor, and time series of demand-to-capacity indexes for axial force and moments, and for shear force and torsion for any desired mean recurrence interval. Response databases for each index were established for a sufficiently wide range of wind speeds and for a sufficiently large number of wind directions. Response databases of interstory drift and acceleration were also obtained. The databases depend on the building's aerodynamic, geometric, structural, and dynamical features but are independent of the wind climate.

The study employed directional wind speed data of hurricanes for a Miami location, obtained from the directional hurricane wind speed database listed on www.nist.gov/wind. Demand-to-capacity indexes were adjusted in accordance with ASCE 7-05 requirements.

The DAD methodology has the following advantages over frequency-domain procedures: (1) it preserves all phase relationships, so structural responses attributable to combined effects (e.g., combined effects in the directions of the principal axes of the building) are calculated by superposing individual effects; (2) wind loads along the building height are on the basis of the actual distribution of the pressures as measured in the wind tunnel; (3) any modal shape, higher modes of vibration, and mode coupling are easily accounted for.

DAD appropriately accounts for wind directionality by using wind climatological data that may need to be augmented by simulation, aerodynamic data, and micrometeorological data (i.e., ratios of directional wind speeds in open exposure at 10 m above ground to their mean hourly counterparts at the top of the structure). Estimated peak responses obtained from DAD are estimated for the requisite mean recurrence intervals. This requires that the estimates be performed in the wind effects space. The procedure requires the use of structural engineering design information in the form, notably, of appropriate interaction equations specific to reinforced concrete members.

The procedure was illustrated through its application to a specific design of the CAARC building. The conclusions resulting from this application would clearly differ for different types of building or design. Software for implementing the DAD procedure used in this study is available on www.nist.gov/wind.

DAD clearly separates the wind engineer's and the structural engineer's tasks. The wind engineer's task is to produce the requisite pressure time histories, wind climatological directional data, and ratios of directional wind speeds at standard elevation in open terrain exposure to the corresponding directional hourly mean wind speeds at the top of the structure. Once these data are available, the structural engineer performs the requisite structural analyses and accurately determines members' demand-to-capacity indexes, interstory drift, and top floor accelerations. Therefore, the DAD procedure allows the design process to be controlled and scrutinized by the structural engineer. DAD renders the design process transparent, and makes the partners in the design process clearly accountable to all stakeholders, including owners and building inspectors. The design approach presented in this paper provides more accurate and clearer predictions of wind effects than conventional approaches, and is expected to be more economical and efficient when used in conjunction with optimization.

Appendix. Demand-to-Capacity Index (B_{ij}) for RC Members

The demand-to-capacity index is a quantity used to measure the degree to which structural members are designed adequately. In general, this index is defined as a ratio of the strength required to resist the effects of the design loads to the strength of a member available to resist those effects. The design strength in HR_DAD_RC is based on the *Building Code Requirements for Structural Concrete and Commentary* (ACI 318-08; ACI 2008). Indexes higher than, equal to, and lower than unity correspond, respectively, to demand higher than, equal to, and lower than the capacity. Two demand-to-capacity indexes are used in HR_DAD_RC: the index for bending moment or interaction of axial force and bending moments ($B_{ij}^{PM^*}$), and the index for interaction of shear forces and torsion ($B_{ij}^{VT^*}$). They are described in the NIST report (Yeo 2010) available from www.nist.gov/wind.

Acknowledgments

The authors would like to thank Dr. H.S. Lew of the National Institute of Standards and Technology for valuable advice and comments, and acknowledges with thanks the previous development of HR_DAD software by Mihai Iancovici, William P. Fritz, René D. Gabbai, and Seymour M.J. Spence during their postdoctoral research associate or guest worker tenures at the National Institute of Standards and Technology. Helpful comments by the reviewers are also gratefully acknowledged. The wind tunnel data developed at the CRIACIV-DIC Boundary Layer Wind Tunnel were kindly provided by Dr. Ilaria Venanzi of the University of Perugia.

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