

# Wind Effects on a Tall Building with Square Cross-Section and Mid-Side Base Columns: Database-Assisted Design Approach

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**Abstract:** This paper illustrates the application of the database-assisted design (DAD) method to the wind design of high-rise buildings. The paper uses publicly available wind tunnel data and DAD procedures to compare responses to (1) corner winds and (2) face winds of a high-rise building of square cross-section supported by a central core column and four mid-side legs. The responses being considered consist of overturning moments, and of demand-to-capacity indexes (DCIs) of selected members, including multistory chevron braces. The analysis accounts for structural dynamics and second-order load-deformation effects. The results show that corner winds are less demanding than face winds, both globally (overturning moments) and locally (DCIs). The along-wind and across-wind overturning moments in the corner wind case are about 20% and 50% lower, respectively, than their counterparts in the face-wind case. The peak axial forces in the legs (peak refers to absolute value) and the peak DCIs in the mid-side mast columns (continuation of the legs) induced by corner winds are lower by 20%–30% than their counterparts due to face winds. The investigation confirms that the building code of the City of New York in effect in the early 1970s can be interpreted as meaning that the design for wind of structures with a square shape in plan may be performed by assuming the wind loads to act normal to a face of the building. The building analyzed in this paper is similar to the Citicorp Building (completed in 1977, later renamed Citigroup Center, now called 601 Lexington) and the results of the analyses presented herein suggest that a re-examination of the history of the Citicorp Building design and retrofit may be warranted. DOI: 10.1061/(ASCE)ST.1943-541X.0002328. © 2019 American Society of Civil Engineers.

**Author keywords:** Aerodynamics; Citicorp building; Corner winds; Database-assisted design (DAD); Demand-to-capacity index (DCI); Face winds; High-rise building; Mid-side columns; Overturning moments.

## Introduction

This note investigates effects of wind direction on the response of a tall building with a square horizontal cross-section supported by a central core column and four mid-side legs. The investigation is carried out using a state-of-the-art database-assisted design (DAD) procedure in conjunction with publicly available wind tunnel data measured with multichannel pressure scanners. Given the similarity between the structure analyzed in this paper and the Citicorp Building [Fig. 1(b)], the results of the analysis presented here may help elucidate certain aspects of the design of the Citicorp Building (completed in 1977, later renamed Citigroup Center, now called 601 Lexington), and suggest that a re-examination of the history of the design and repair of that building using modern

pressure measurement and structural analysis technologies may be warranted.

## Historical Perspective and Motivation

The original structural design of the Citicorp Building was based upon the specifications of the building code of the City of New York, as amended in March 1970 (NYC 1970; Hartley 1978). According to this code, the design for wind of structures with a square shape in plan is governed by wind loads acting normal to a face of the building. Morgenstern (1995) reported that the structural engineer in charge of the Citicorp Building design received a call from an undergraduate student at work on a thesis, who asked whether the case of corner winds was considered in the design. (That former student, Diane Hartley, is acknowledged in this paper.) The structural engineer was intrigued by the question and consulted the wind engineer who had supervised wind tunnel tests of a model of the building. The consultation concluded that the building as constructed was unsafe under corner winds. This prompted the structural engineer to undertake an urgent strengthening of the building (Morgenstern 1995).

Much has been written about lessons to be learned from this case (Kremer 2002; Vardaro 2013; Whitbeck 2006). A post-mortem, 30-page study performed by the engineering design firm, entitled Project SERENE, “Special Engineering Review of Events Nobody Envisioned,” mentioned by Morgenstern (1995) and Vardaro (2013), and a wind tunnel test report on measurements of the building (Isyumov et al. 1975) have been completed but apparently have not been released to the public.

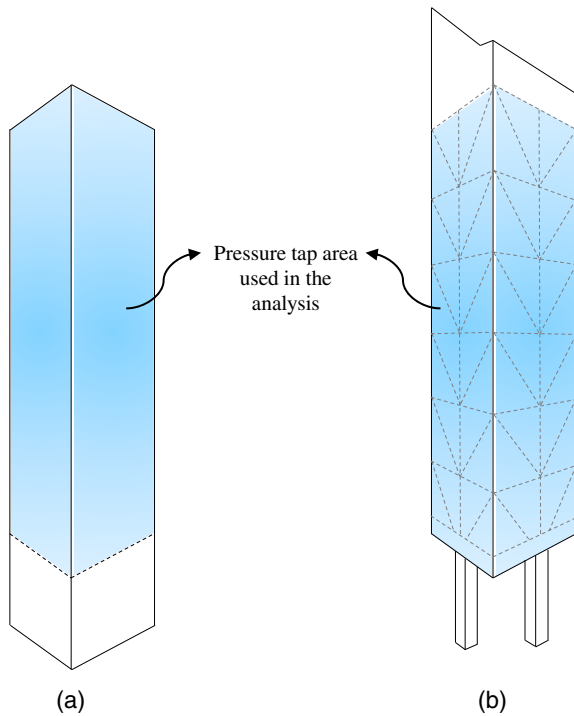
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Note. This manuscript was submitted on May 14, 2018; approved on November 8, 2018; published online on March 6, 2019. Discussion period open until August 6, 2019; separate discussions must be submitted for individual papers. This technical note is part of the *Journal of Structural Engineering*, © ASCE, ISSN 0733-9445.



**Fig. 1.** (a) TPU pressure taps on square building of aspect ratio 5; and (b) schematic of Citigroup Center. The figure shows 100 out of 125 taps per each façade. 400 taps in total were used, shaded areas.

The Citicorp Building story has captured the imagination of a generation of structural engineers and wind engineers like no other story since the Tacoma Narrows bridge collapse. This paper addresses the belief on the part of the designer that corner wind effects were more severe than the effects of face winds.

### Determining Wind Effects Using Database-Assisted Design

Wind effects on a structural system similar to the Citicorp Building were determined in this work by using DAD, a rigorous, transparent, and effective wind design procedure made possible by (1) the development of hardware capable of simultaneously measuring and recording time histories of wind pressures at multiple taps and by (2) the availability of computer resources needed for processing large amounts of data economically and rapidly. The following sequence of steps briefly describes the structural design performed by structural engineers using DAD (Park and Yeo 2018):

Task 1. Select the structural system and determine its preliminary member sizes based on a static wind loading from standard provisions. The structural design so achieved is denoted by  $D_0$ .

Task 2. For the design  $D_0$  determine the system's mechanical properties, including the modal shapes, natural frequencies of vibration, and damping ratios, as well as the requisite influence coefficients; and develop a lumped-mass model of the structure.

Task 3. From the time histories of simultaneously measured pressure coefficients, determine the time histories of the randomly varying aerodynamic loads induced at all floor levels by mean wind speeds from, depending upon location, 10–80 m/s in increments of 10 m/s, say, with directions from  $0 \leq \theta_w < 360^\circ$  typically in increments of  $10^\circ$ .

Task 4. Perform the dynamic analysis based on the lumped-mass model of the structure to obtain the time histories of the inertial forces induced by the respective aerodynamic loads, and the effective wind-induced loads consisting of the sums of the aerodynamic and inertial force time histories. The effective lateral loads are determined at all floor levels of the building. The dynamic analysis can be performed by commercial software, e.g., SAP 2000 version 17 (CSI 2015), up to half a dozen or so modes of vibration.

Task 5. For each cross-section of interest, use the appropriate influence coefficients to obtain time series of the demand-to-capacity indexes (DCIs) induced by the combination of effective lateral loads determined in Task 4 with factored gravity loads. The DCIs are the left-hand sides of the design interaction equations, and are typically used to size members subjected to more than one type of internal force (ANSI/ASCE 2010a).

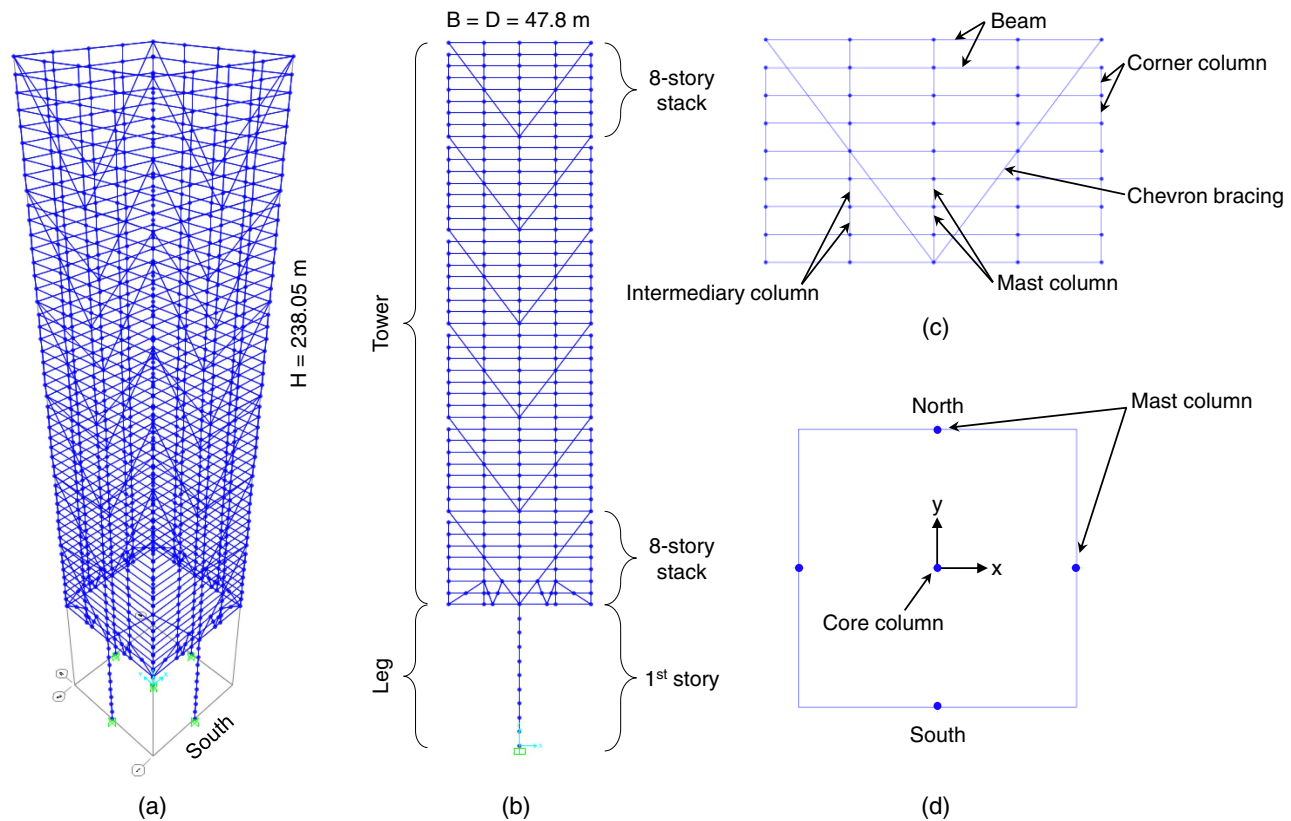
Task 6. Construct the response surfaces of the peak (peak refers to absolute value) combined effects (e.g., DCIs, interstory drift ratio, accelerations) as functions of wind speed and direction. For each of the directional wind speeds defined in Task 3, determine for each cross-section of interest the peak of the DCI time series, and construct from the results so obtained a peak DCI response surface. The response surface is a property of the aerodynamic and mechanical characteristics of the structure, independent of the wind climate, that provides for each cross-section of interest the peak DCIs (or other wind effects) as a function of wind speed and direction.

Task 7. Use the information contained in the response surfaces and in the matrices of directional wind speeds at the site to determine, by accounting for wind directionality, the design DCIs with the specified design mean recurrence interval, for the cross-sections of interest.

In general, the preliminary design  $D_0$  does not satisfy the strength and/or serviceability design criteria. The structural members are then resized to produce a modified structural design  $D_1$ . This iterative process continues until the final design is satisfactory. Tasks 2–7 are repeated as necessary until the design DCIs are close to unity, to within serviceability constraints. For further details on DAD (Park and Yeo 2018; Simiu and Yeo 2015; Yeo and Simiu 2011).

Since the actual wind tunnel tests of Citicorp Building remained proprietary, aerodynamic pressure time histories were obtained from the Tokyo Polytechnic University (TPU) aerodynamic database (Tamura 2012) for an isolated building in open terrain with square cross-section and depth to height ratio 1:5, modeled at a scale of 1:478. The reference wind speed considered in the analysis was 11 m/s at the top of the model, corresponding to 44.6 m/s [mean hourly wind speed for building Categories III and IV, ASCE 7 (ASCE 2016)] at the rooftop of the prototype building. The aerodynamic pressures at 500 tap locations [Fig. 1(a)] were acquired for 30 s at a sampling rate of 1,000 Hz. Wind pressure for the numerical analysis presented here on the part of Citicorp Building [Fig. 1(b)] between the wedge crown and the legs was estimated from the corresponding (top) 400 TPU taps. Measurements for two wind directions,  $0^\circ$  (south) and  $45^\circ$  (southwest) (Fig. 3), were used, and pressures for winds with directions  $90^\circ$ ,  $180^\circ$ ,  $270^\circ$ ,  $135^\circ$ ,  $225^\circ$ , and  $315^\circ$  (Fig. 5) were obtained by symmetry. These wind pressures were used to determine which of these directions resulted in more unfavorable wind effects on the structure.

Fig. 2 shows a schematic view of the 49-story building being considered. The building height is 238 m, and its horizontal cross-section is square, with dimensions  $47.8 \times 47.8$  m. The structural system, similar to that of the Citicorp Building in New York City, is shown in Fig. 2(b). The system was designed to transfer all overturning wind loads and one half of the gravity load via an



**Fig. 2.** Structural system: (a) 3D view; (b) side view; (c) 8-story stack; and (d) plan view.

exterior set of chevron braces to four mast columns located at the middle of the sides of the building. The remainder of the gravity load is carried by the central core (ENR 1976; Hartley 1978, p. 102). The 48 office floors are divided into six stacks of eight floors, and for each stack and each side of the building, a chevron brace transfers loads to the mid-side mast columns. The corner columns are interrupted at the top floor of each stack for ease of analysis (mostly hand calculations in the 1970s) [Fig. 2(b)]. A two-story truss system distributes the vertical loads from the lowest 8-story stack to the legs, which are the continuation of the mid-side mast columns. The six stacks of eight stories each are connected by a massive core column and intermediate (between corner and mid-side) columns, in addition to the four mid-side mast columns. The analysis models each floor as a rigid horizontal diaphragm. Within each stack, the cross section of the mast column and that of the chevron braces remain constant. The cross-sections of the beams, intermediate, corner and core columns remain unchanged between stacks.

The core column and the four mid-side legs consist of hollow structural sections (HSS). All other structural members consist of rolled wide-flange (W-) sections selected from the Steel Construction Manual [AISC 360 (ANSI/AISC 2010a); ANSI/AISC 2010b]. The modal damping ratios are assumed to be 1.5% in all six modes considered in this study.

The applicable load combination per ASCE 7 (ASCE 2016) is

$$1.2D + 1.0L + 1.0W \quad (1)$$

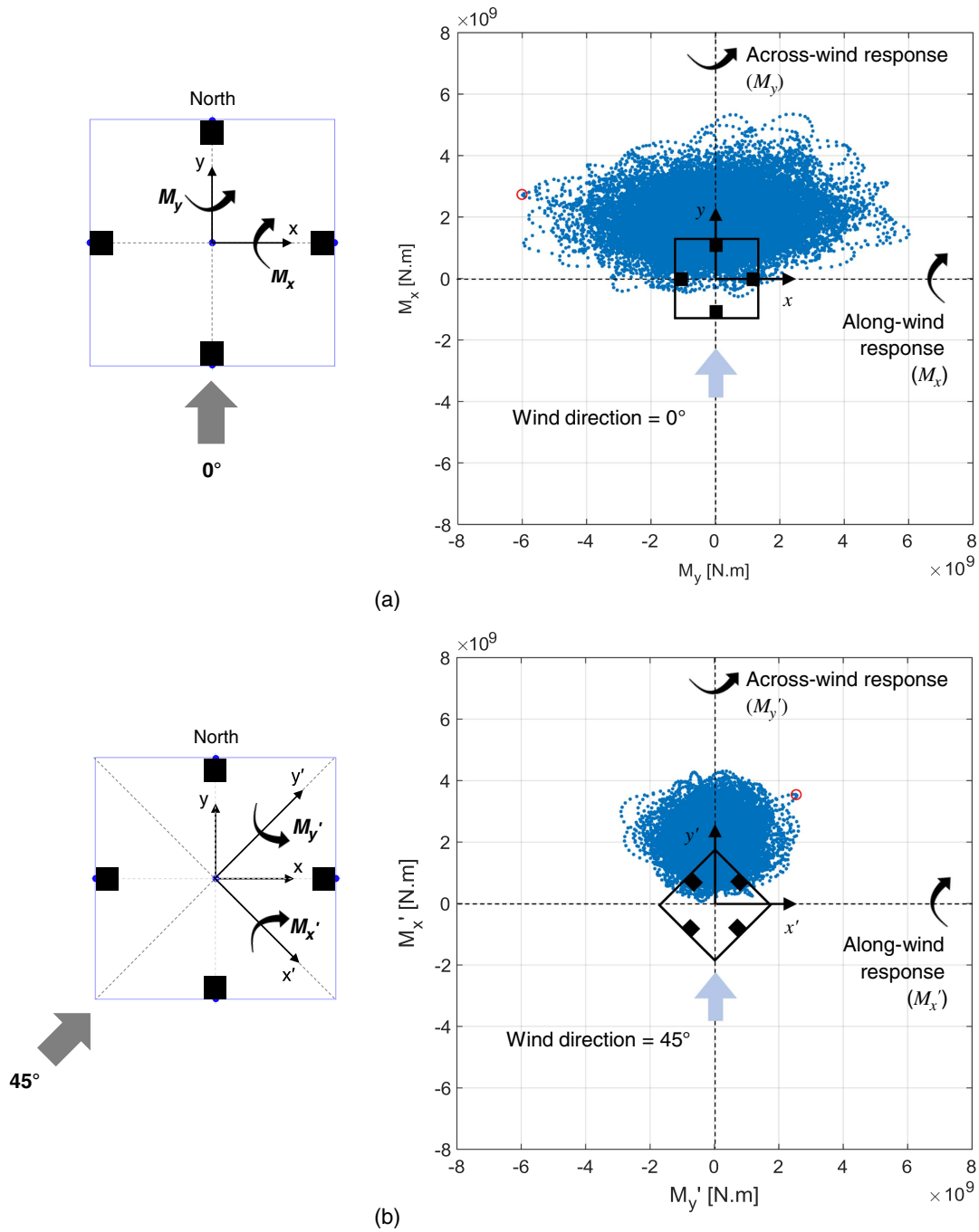
where  $D$  is the dead load,  $L$  the live load, and  $W$  the wind load.

As a complete set of blueprints of Citicorp Building was not available, initial dimensions of structural members were selected to resist wind loads  $W$  specified by the ASCE 7 Standard

(ASCE 2016) for flexible buildings of all heights. The resulting preliminary design, denoted by  $D_0$ , was then subjected to the aerodynamic loading  $W$  derived from the pressure coefficient time series extracted from the Tokyo Polytechnic University database (Tamura 2012). As member sizes were not necessarily the same as in the actual building, a second iteration was performed, resulting in design  $D_1$ , for which the DCIs were adequate. [A  $DCI \ll 1$  means the member is safe, but uneconomical. A  $DCI \geq 1$  means the member has reached or exceeded a limit state specified in Load and Resistance Factor Design, ANSI/AISC 360 (ANSI/AISC 2010a)]. The analysis used  $D_1$  and accounted for second-order load-deformation effects using the approach of Park and Yeo (2018). The natural frequencies and mode shapes were determined using SAP 2000 version 17 (CSI 2015).

The time series of base overturning moments (positive in the directions of the curved arrows; in this paper, unless numbers have a sign, they refer to magnitudes or absolute values) in the along-wind and across-wind directions are represented by dots in Fig. 3(a) for wind normal to a building face (face wind  $0^\circ$ ) and Fig. 3(b) for quartering wind (corner wind  $45^\circ$ ). Each cloud has 30,000 dots corresponding to 30 s of wind pressure data sampled at 1,000 Hz. Fig. 3(a) shows that for face winds, the peak across-wind overturning moment is about 12% higher than the peak along-wind overturning moment. For corner winds [Fig. 3(b)], the along- and across-wind overturning moments are about 20% and 50% lower than their counterparts induced by face winds. Table 1 summarizes the peak along-wind, across-wind, and resultant overturning moments (marked by circles in Fig. 3).

Table 2 lists the peak axial forces for the load combination [Eq. (1)] in the four mid-side legs for face and corner winds. Forces due to corner winds are significantly (between 20% and 30%) smaller than those due to face winds.



**Fig. 3.** Along-wind and across-wind overturning moments: (a)  $\theta_w = 0^\circ$ ; and (b)  $\theta_w = 45^\circ$ . Circles mark the peak resultant overturning moments.

**Table 1.** Peak overturning moments (GN · m)

Wind direction	Along-wind	Across-wind	Resultant
0° face wind	5.35	6.01	6.58
45° corner wind	4.31	2.90	4.35

**Table 2.** Peak axial forces (MN) in legs

Wind direction	North leg	South leg	West leg	East leg
0° face wind	91.8	91.8	104	104
45° corner wind	73.4	73.4	73.7	73.6

DCIs were calculated for the load combination [Eq. (1)] for 12 selected members on the west façade of the building (Fig. 4): one corner column (CC) in the 11th story; three mast columns (MC1, MC2, and MC3) in the 11th, 27th, and 43rd stories; six chevron braces (V11, V12, V21, V22, V31, and V32) in the 1st, 3rd, and 5th 8-story stacks; and the lower and upper parts of one leg

(L1 and L2). Note that the legs count as the bottom ten stories. Their DCIs reflected the interaction between axial forces and bending moments and were calculated for face winds (i.e.,  $\theta_w = 0^\circ, 90^\circ, 180^\circ,$  and  $270^\circ$ ) and corner winds (i.e.,  $\theta_w = 45^\circ, 135^\circ, 225^\circ,$  and  $315^\circ$ ), with a wind speed of 44.6 m/s at rooftop (Fig. 5). Note that structural members on one building face are affected differently by

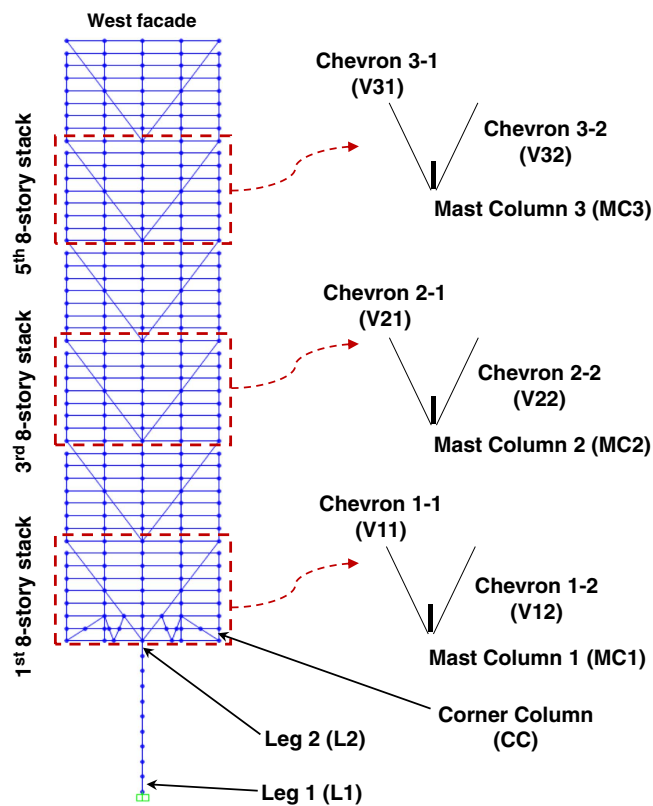


Fig. 4. Selected members of interest.

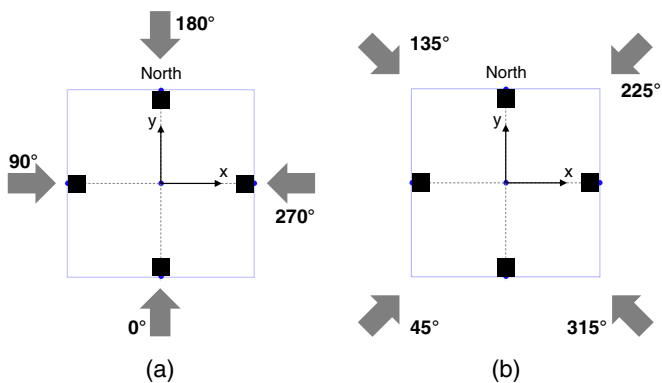


Fig. 5. Wind loading cases for strength design: (a) perpendicular-face winds case; and (b) cornering winds case.

the four different face winds, and the same is true for the four different corner winds. For details of the expression of the DCIs for steel members (Park and Yeo 2018).

Fig. 6 shows the percentage deviation of the peak DCIs for the 12 selected members for the corner wind ( $DCI^C$ ) and face wind ( $DCI^F$ ) cases. Table 3 summarizes the peak DCIs of 12 selected members for both sets of cases. Fig. 6 and Table 3 show that the diagonal or chevron braces are much less stressed by corner winds than by face winds, in combination with gravity loads. Also, the corner column (CC) shows no difference between the corner and face wind cases, since CC is interrupted every eight floors and unable to carry wind loads. The  $DCI^C$  for other members, i.e., mast columns, legs, and chevron braces, are smaller by 20%–30% than their  $DCI^F$  counterparts.

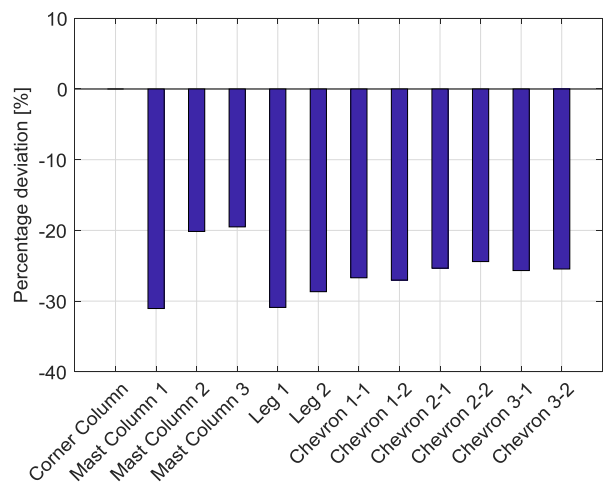


Fig. 6. Percentage deviations of  $DCI^C$  from  $DCI^F$  for 12 selected members.

Table 3. DCIs of selected members for two wind directions

Member	Face wind	Corner wind
Corner column	0.306	0.306
Mast column 1	0.997	0.687
Mast column 2	0.826	0.660
Mast column 3	0.745	0.600
Leg 1	0.887	0.613
Leg 2	0.716	0.511
Chevron 1-1	0.721	0.529
Chevron 1-2	0.724	0.529
Chevron 2-1	0.818	0.611
Chevron 2-2	0.808	0.611
Chevron 3-1	0.728	0.541
Chevron 3-2	0.726	0.541

Results for base shear are similar to those for overturning moments and are omitted for brevity. It is also noted that uncertainties in the pressure measurements and the structural analysis would affect the face wind case and the corner wind case equally, and therefore would not affect the overall conclusion.

## Summary and Conclusions

Using aerodynamic pressures measured by the Tokyo Polytechnic University on an isolated building model in open terrain (in the absence of the actual wind tunnel measurements on a scaled model of the Citicorp Building in an urban environment), this paper investigates the overturning moments and the DCIs of selected structural members caused by face and corner winds on a square high-rise building supported by a central core column and four mid-side legs. The reference wind speed at the rooftop of the building is 44.6 m/s and the building is assumed to remain in the linear elastic range. The main conclusion of this study is that corner winds are less demanding, both globally, as estimated by overturning moments, and locally, as estimated by DCIs, than face winds. The along-wind and across-wind overturning moments of the corner wind case (angle of attack  $\theta_w = 45^\circ, 135^\circ, 225^\circ,$  and  $315^\circ$ ) are about 20% and 50% lower than those of the face wind case ( $\theta_w = 0^\circ, 90^\circ, 180^\circ,$  and  $270^\circ$ ), respectively. The peak axial forces in each mid-side leg induced by the corner winds are lower by 20%

to 30% than their counterparts caused by face winds. The peak DCIs induced by corner winds on selected structural members, including the chevron braces, are also between 20% and 30% smaller than those induced by face winds.

It is noted that, in this study, wind loads on the wedge-shaped crown of the structure is neglected, but this should not affect the conclusion on the relative importance of the effects of corner winds and face winds. It is further noted that wind loading in this study is for open terrain, whereas in actuality, there is significant interference from adjacent buildings. How this affects the conclusion is impossible to ascertain in the absence of the actual wind tunnel test measurements. To the authors' knowledge, the interference of neighboring buildings on the wind loads has never been mentioned in discussions of the Citicorp Building's structural design and strengthening.

The present study illustrates the capabilities of DAD, a state-of-the-art procedure made possible by (1) the development of hardware capable of simultaneously measuring and recording time histories of wind pressures at multiple taps, and (2) the availability of computer resources needed for processing large amounts of data economically and rapidly. The investigation confirms that the building code of the City of New York in effect in the early 1970s can be interpreted as meaning that the design for wind of structures with a square shape in plan may be performed by assuming the wind loads to act normal to a face of the building. It is noted that, when the building was designed, wind tunnel testing was at an early stage of its development, and even modestly reliable statistics of hurricane wind speeds and directions on the basis of which mean recurrence intervals of wind-induced effects could be estimated were not available. Structural analysis was still largely done by hand, as was evidenced by the interruption of the corner columns at the top of the 8-story stacks to make the stacks statically determinate. In view of these facts, and given the similarity between the structure analyzed in this paper and the Citicorp Building, the results of the analysis presented here suggest that a re-examination of the history of the Citicorp Building design and retrofit may be warranted. In particular, building science might benefit from a careful scrutiny of the basis for the decision to redesign and strengthen the structure after its completion.

## Acknowledgments

The authors are grateful to Diane Hartley for enlightening discussions of her undergraduate thesis, and to Christa Cleeton of the Seeley G. Mudd Manuscript Library at Princeton University for making available high-resolution copies of the Citicorp Building blueprints. This paper is dedicated to the memory of David Billington, an inspiring teacher to one of the authors (D.D.) and thesis supervisor to Hartley.

## Disclaimer

Some commercial products are identified in this paper for traceability of results. Such identification does not imply recommendation or endorsement by the National Institute of Standards and Technology, nor does it imply that the products identified are necessarily the best available for the purpose.

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