Comparative study of tall building response to synoptic and non-synoptic wind action

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Abstract

Until the beginning of the 21st Century, the characteristics of wind for purposes of structural analysis and design, as reflected in wind codes worldwide, were based on the behavior of wind currents in the vicinity of the ground surface observed in so-called synoptic events. It was only recently recognized that the latter are not the only cause of wind damage to buildings and structures, not even its main cause. In view of the absence of any reference to non-synoptic winds in most South American wind codes, their urgent revision to include the effects on non-synoptic winds is badly needed. The *downburst* is a relevant meteorological phenomenon that causes extreme winds in the lower atmospheric boundary layer. The present article briefly describes an introduction to an simplified approach recently proposed by the second author to describe the wind velocity field in this type of meteorological phenomenon, that is, downbursts within instability lines. The method is examined by a comparative study of tall building response to synoptic and non-synoptic wind action.

Keywords: wind action, structural design, synoptic wind, downburst, velocity field, risk, probability of occurrence.

1. Introduction

Wind loads play an important role in structural design, especially in the case of tall or light constructions. Most codes assume that at above plane, horizontal ground, the *mean* velocity vector is constant and parallel to the ground surface. The hypothesis is valid in case of so-called synoptic wind, which is the most frequent type of wind storm in temperate regions, namely extra-tropical storms or Extended Pressure Systems *(EPS)*, and in the case of tropical storms or hurricanes, also designated typhoons when originated in the Pacific Ocean. On the other hand, wind effects caused by *downdrafts* or *downbursts*, typical of thunderstorms *(TS)*, or of combinations of the latter with an *EPS* event, in so-called *instability* or *squall lines*, have not yet been explicitly considered in the wind codes in South America. It is germane to underline at this point that the wind velocity field during squall lines is significantly different from the field assumed in most wind codes, usually based on models valid only for synoptic wind. Important differences between wind originated in *EPS* and *TS* events are the following: records of *EPS* winds may be regarded as samples of *random stationary processes*. Moreover, the frequency content of the process depends on the surface roughness of the upwind terrain. None of these assumptions is valid for *TS* winds. As an obvious consequence, methods prescribed in wind codes for assessing the response of structures subjected to *EPS* winds cannot be directly applied to excitation due to *TS* winds. In temperate regions, *not affected by tropical storms*, around nine out of every ten observations of the maximum annual horizontal component of the wind velocity at the standard 10 m height occur during *EPS* events. In consequence, extreme velocities for return periods that exceed 10 years are almost always due to *TS* events, which should then govern structural design, at least for low construction heights. Evidence in relation with the statement, in general, may be found in Letchford & Lombardo (2015). Data from Brazilian meteorological stations are described in Riera & Nanni (1989) and Riera, Viegas & Santos (1989). The practical importance of determining the probability distribution of maximum annual velocities caused by *TS* events, independently of *EPS* winds, was underlined by Riera *et al* (1977), but not yet implemented in South America, to the author's knowledge. Thom (1967) had earlier suggested the use of a mixed distribution P_V (v) to predict the occurrence in the USA of extreme winds due to *EPS* events and to hurricanes. Later, Riera & Nanni (1989) examining records of 14 weather stations located in southeastern Brazil, concluded that the maximum annual velocities of wind caused by *EPS* or by *TS* phenomena are characterized by *probability distribution functions with different parameters*. It was found for most annual maximum velocity series, the Fisher-Tippett Type I, also known as Gumbel distribution, presented a better fit to the data than either the Types II or III extreme value distributions.

Additional evidence on the importance of winds caused by *TS* events was provided by builders or designers of electrical transmission lines. In fact, according to CIGRÉ (2002), in temperate climates worldwide, more than 83% of failures of transmission towers or lines were caused by downbursts. Finally, at the recent *14th ICWE - International Conference on Wind Engineering,* held in 2015, more than 20 contributions dealt with *TS* winds and their effects. Letchford (2015) discusses the possibility of including in wind codes, the guidelines for designing structures subjected to downbursts, as already proposed by Gomes & Vickery (1978), who are among the first researchers that recognized the need to separate wind velocity records according to the causative meteorological phenomenon. Moreover, recent studies confirm that the wind loads that control structural design in most areas of the continental USA are due to the *TS* event (Lombardo, 2012; De Gaetano *et al,* 2014). These developments led to the consideration of TS winds in the revised map of wind velocities of ASCE Code 7 (2016), following previous advances of the Australia/New Zealand Standard AS / NZS 1170.2.

Riera (2016) recently suggested a simplified procedure to account for the effects of *TS* events in structural design, based on the observation that the horizontal component of the maximum wind velocities at the reference 10 m height during stationary *TS events* (downbursts) very rarely exceeds 30 m/s. This velocity is below the minimum wind design velocity in most regions of the entire South American continent and since the dimensions of the area affected by the wind velocity field are, more often than not, of the same order of magnitude or smaller than the dimensions of the structures under consideration, the resulting wind forces should rarely exceed current code prescriptions for synoptic winds. In consequence, except in special situations, the action of *stationary TS events* (downbursts) should not have any influence on the required structural strength and *needs not be discussed in wind codes*. However, in so-called *instability lines,* also known as *squall lines,* in which the wind velocities caused by the downdraft from the causative cumulonimbus cloud sums up with the velocity of the (usually synoptic) wind that carries the cloud, the horizontal component at 10 m height may exceed 60 m/s, which has to be considered in design. Thus, Riera (2016) proposed a simplified model for the design wind in a squall line, which will be described next.

Thus, in order to assess the predictions of the design procedure suggested by Riera (2016), the probabilistic response of a prismatic building with square cross-section and heights in the range between 20 and 300 m is assessed. To this purpose, the simulation model proposed by Ponte and Riera (2010) and extended by Gheno *et al.* (2014) is employed.

2. Simplified model of a squall line

Let V_o denote *the design velocity* for *TS* winds, defined as the horizontal component of the maximum velocity at the standard 10 m height above ground level, for a period of exposure and probability of occurrence defined by the designer, or specified in the wind code. For recurrence periods of 25 years or more, the *TS* wind would *almost certainly* occur in a squall line, in which case the translation velocity of the downburst may be estimated as 0.35 V_o, as discussed in Riera (2016). On a plane normal to the orientation of the squall line, the horizontal component of the mean velocity at the reference height (z = 10 m) may be represented by the simplified diagram shown in Figure 1.

For static structural analysis or design, only the peak values (V_o or 0.35 V_o) and associated vertical profiles are needed. Note that outside the band of width *b*, the vertical profile and other characteristics of the wind correspond to an *EPS* event, as described in Code NBR 6123 and elsewhere. The parameter *T*, known as *characteristic time* of the *TS* event, depends on several factors, such as the height and other geometrical dimensions of the causative cumulonimbus cloud and the translation velocity, typically increasing with the intensity of the event (Ponte & Riera, 2007). The recommended values of the parameter *T* for *TS* event categories are indicated in Table 1. Finally, within the band of width *b*, the vertical profile may be determined by the Equation (1), proposed by Savory *et al* (2001) for *TS* winds. Note that, employing the heights z_{max} indicated in Table 2, the value of V_{max} may be determined if V_o is specified.

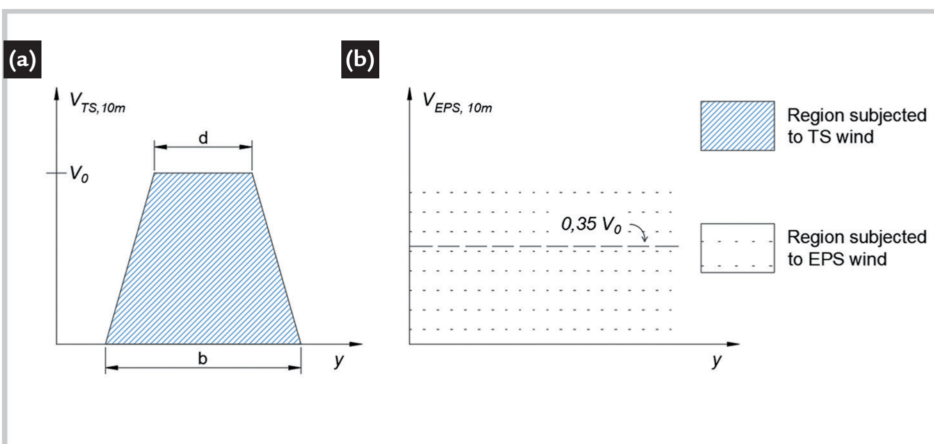

Figure 1
Velocity distribution at the reference height (z = 10 m), in a cross-section normal to the orientation of the velocity for a *TS* event. (a) Downburst wind (TS): width at the top of the trapezoidal region *d*, width at the base *b*. Outside of the width *b*, the downdraft does not contribute to the wind field. (b) Synoptic wind.

Table 1
Parameters of the five categories of downdrafts (CD).

Designation	V_o	d	b	Height of V_{max}	T
(CD)	(m/s)	(m)	(m)	z_{max}(m)	(s)
CD 1	$V_o \leq 30$	10	40	20	60
CD 2	$30 < V_o \leq 40$	20	60	40	120
CD 3	$40 < V_o \leq 50$	40	100	80	180
CD 4	$50 < V_o \leq 60$	60	160	120	300
CD 5	$60 \leq V_o$	80	240	160	480

$$V(\eta) / V_{max} = exp\ (- 0.15\ \eta\ ) - exp\ (- 3.2175\ \eta\ ),\ \eta = z / z_{max} \quad (1)$$

3. Simulation procedure

Ponte & Riera (2010) simulated a series of observations of the horizontal component of the wind velocity at a 10 m height during TS events at the locations of two meteorological stations in Southern Brazil. Probability distributions of the maximum annual velocity based on the simulated series were shown to be close to the distributions based on actual meteorological records from those stations. The approach was extended by Fadel Miguel & Riera (2013) to simulate an annual series of bending moments and shears caused by TS events at the base of a free standing constant cross-section 50 m tall building. The procedure was later employed by Gheno *et al* (2014) to determine by simulation the response of transmission lines. In all those studies, the simulation of annual maximum wind velocity, as well as response data of specific structures at predefined locations required the specification of the following data: geographic region of interest, dimensions of area for the simulation, mean frequency of TS events in the region, pressure drop in thunderstorms, characteristic time, velocity and direction of background wind, height of cumulonimbus cloud, radius of the wind flow at the cloud base. The topics listed and the discussion in connection with the numerical examples adopted herein are found in Gheno *et al* (2014).

Thus, in the present application, the simulation starts by defining the coordinates of the points of interest (building heights). Afterwards, random values of the following variables are generated, considering the probability distribution of the variable under consideration: coordinates of the point of origin of the thunderstorm, height of the anvil, pressure drop, characteristic time of the event, radius of the flow at the cloud base, background wind speed, orientation of background wind (γ). For each height of interest z, the following steps are performed: the tangential velocity of the streamline passing through x,y,z is calculated; the background wind speed is determined considering the height z of the point; the coordinates of the storm center at each time step are calculated; the distance between the storm center and the location of interest for each time step is calculated; the evolution with time of the wind velocity is accounted for with the modulation function and the corrections based on the distance from the point to the origin of the storm are applied; if the point is outside the area of influence of the TS event, the speed is set equal to zero.

Thus, the velocity generated by the TS is defined at each instant of time for pre-defined building heights. From these data, the resulting speed can be calculated by combining the velocity vector generated by the downburst with the background wind velocity vector. To simulate a series of annual maximum shearing forces and bending moments at the building base, the number of years to be simulated and the number of events each year must be defined. Afterwards, for each event, the steps presented above are executed. At the end of each event, the maximum wind speed at 10 m height, and the corresponding shearing forces and bending moments at the building base are recorded. This process is repeated for all the events that occur during the year. By performing this procedure, a series of annual maximum 10 m height velocities is generated, as well as series of annual maximum shearing forces and bending moments at the building base. In previous studies (Ponte & Riera, 2010; Fadel Miguel & Riera, 2013 and Gheno *et al,* 2014), simulations of wind fields during TS events were limited to heights below around 50 m, since they were intended for applications for low height constructions such as transmission lines, silos and industrial structures. The basic assumptions and computer programs were not tested for tall buildings or towers and thus only results for heights below 90 m are herein reported, range in which the influence of the adopted vertical profile proposed in literature should be small.

4. Assessment of response methods

For comparison purposes, a prismatic building with square cross-section and heights in the range between 20 and 300 m was considered in this assessment. A base length of 15 m was adopted in all cases, which means that the slenderness of the buildings varied between 1.3 and 20. Only wind orientation normal to one of the faces was considered in the determination of the resulting shear and bending moment at the base. The response of the buildings was calculated adopting the model suggested by Ponte & Riera (2007, 2010) and the simulation scheme employed by Miguel & Riera (2013), with the modifications introduced by Gheno et al (2014), to estimate the static response of slender structures subjected to TS wind action (Method TS1). Next, the simplified method proposed by Riera (2016) (Method TS2) was employed with the same objective. In both cases, response amplification due to dynamic effects caused by TS wind action was neglected, a simplification that is clearly acceptable for practical structural design of buildings less than about 60 m in height, but requires further verification for taller structures, which are presently in progress. The approach under consideration for assessing the dynamic component of TS wind effects differs from existing proposals in technical literature. It may also be noted that the scheme employed by Le and Caracoglia (2017) to determine the transient dynamic response of a tall building subjected to a specific TS event does not seem feasible for use in engineering design. The same prismatic buildings were analyzed employing the procedures adopted in the Brazilian code NBR 6123 (1988), which are applicable to synoptic winds (EPS). Under the action of EPS winds, the buildings described above were examined under Exposure case 2 (grasslands, open fields), subjected both to static (EPS1) and dynamic (EPS2) excitations.

Table 2
Summary of resulting shears and moments

	Building height	Method TS1				Method TS2	EPS1	EPS2
		1	2	3	Mean		Static	Dynamic
Velocity - V_0 (m/s)	20m	38.5	36.1	38.2	37.6	37.6	37.6	37.6
Shear (kN)		316	287	341	314.	297	301	282
Bending moment (KN.m)		3269	2992	3265	3175.33	3893	3149	3342
Velocity - V_0 (m/s)	30m	35.5	40.1	39.1	38.2	38.2	38.2	38.2
Shear (kN)		481	574	549	534.	706	526	495
Bending moment (KN.m)		7568	9041	8759	8456	13782	8354	9011
Velocity - V_0 (m/s)	40m	37.0	38.6	38.9	38.2	38.2	38.2	38.2
Shear (kN)		678	713	769	720	1170	761	713
Bending moment (KN.m)		14297	15443	16043	15261	29599	16211	17408
Velocity - V_0 (m/s)	50m	37.2	39.9	37.5	38.2	38.2	38.2	38.2
Shear (kN)		887	1042	941	956	1632	1000	931
Bending moment (KN.m)		23063	27547	24479	25030	50047	26728	28478
Velocity - V_0 (m/s)	60m	42.1	36.5	40.2	39.6	39.6	39.6	39.6
Shear (kN)		1400	1113	1292	1268.	2282	1441	1269
Bending moment (KN.m)		43788	34901	40121	39603	81507	46295	46531
Velocity - V_0 (m/s)	90m	36.8	39.5	38.1	38.1	38.1	38.1	38.1
Shear (kN)		1838	2067	1836	1913.	3445	2143	2052
Bending moment (KN.m)		87292	98273	86340	90635	170395	103546	113072
Velocity - V_0 (m/s)	120m	38.2	36.8	41.0	38.7	38.7	38.7	38.7
Shear (kN)					.	4775	3234	3187
Bending moment (KN.m)						294467	208618	234211
Velocity - V_0 (m/s)	150m	35.9	41.1	36.4	37.8	37.8	37.8	37.8
Shear (kN)						5547	4141	3866
Bending moment (KN.m)						402867	334155	351721
Velocity - V_0 (m/s)	180m	34.8	36.8	42.0	37.9	37.9	37.9	37.9
Shear (kN)						6448	5317	4775
Bending moment (KN.m)						531868	515097	517061
Velocity - V0 (m/s)	300m	39.0	37.2	37.4	37.9	37.9	37.9	37.9
Shear (kN)						8878	10619	10353
Bending moment (KN.m)						998278	1715997	1874323

Table 2 summarizes the results of the numerical assessment. First, three independent runs of the simulation model (Method TS1) were performed for each building. Then, the wind speed (horizontal component at 10 m height, 3 s, maximum in a 50-year series) and the corresponding shearing forces and bending moments at the building base were determined. Their average values were used in the ensuing comparisons. Finally, the mean wind velocity previously evaluated, was adopted as design velocity to determine the shear and moment predicted using both the simplified *TS* model (Method TS2) and the procedures prescribed by NBR 6123 for static (EPS1) and dynamic (EPS2) analysis, which were developed for synoptic winds only. Once again, the resulting shear forces and bending moments at ground level were determined. To apply the NBR 6123 dynamic procedure (EPS2), a building mass of 1000 kg/m^2 for each floor was assumed. The critical damping ratio was taken equal to ξ=0.02, the first natural period and the first mode shape were determined by the simplified expressions $T1=0.05+0.015h$ and $x(z)=(z/h)^{1.2}$, respectively.

The shearing forces and the bending moments for EPS2, TS1 and TS2 models are also plotted in Figure 2 and Figure 3, respectively (the dotted and dashed lines are tendency curves adjusted to the obtained results). The dynamic procedure prescribed in NBR 6123 (1987), denoted as model EPS2, may be regarded as a robust estimate of a standard building response to synoptic wind in Exposure type II. Note that model EPS1 indicates the static wind load determined according to the same standard, without taking into consideration the allowable reduction of the incident velocity due to the building size. The response designated EPS1 does not include the reduction due to size effect because of practical difficulties, since the NBR 6123 code presents the appropriate coefficients in the form of tables. As explained in the text, the computed shear and bending moment at the base constitute upper bounds, which add credibility to the EPS2 results: the dynamic response, for all heights, does not differ much from the upper bound of the static approach. Since both shearing forces and bending moments at ground level predicted by models EPS1 and EPS2 are quite similar, the analysis suggests that, for average buildings with heights within the range considered, the reduction of wind loads due to building size in the static analysis is comparable to the dynamic amplification induced by wind turbulence. The observation, however, valid for type II exposure, should not be extended to other surface roughness conditions without additional studies, but lends support to the results presented.

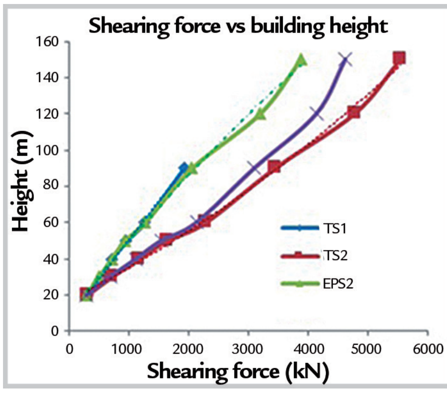

Figure 2
Shearing forces at the building base for building heights up to 150 m, for TS and EPS high intensity winds.

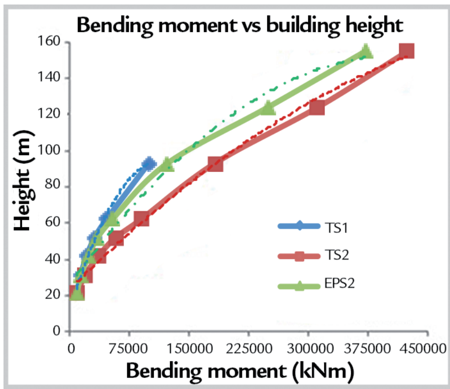

Figure 3
Bending Moment at the building base for building heights up to 150 m, for TS and EPS high intensity winds.

The excitation determined by simulation for non-synoptic wind (TS1), which was employed earlier by the authors to determine the resulting wind pressures for elevations below 50 m (Fadel Miguel & Riera, 2013; Gheno *et al,* 2014) leads to smaller values of both the shearing forces and bending moments for buildings with heights up to 90 m than the proposed model TS2. The difference is attributed to the vertical profile adopted in the latter (Savory *et al,* 2001), which is judged to better explain the distribution of damage observed in recent TS events. Since model TS2 does not take into consideration pressure fluctuations, it is expected to overestimate the predicted non-synoptic wind action, just as model EPS1 overestimates the static synoptic wind action.

5. Conclusions

The need for procedures to determine the response of structures subjected to non-synoptic wind was briefly described, as an introduction to a simple method to assess the wind velocity field for structural design in those situations. Herein, predictions of the shearing load and bending moment at ground level, for prismatic buildings with average properties (stiffness and mass) and heights extending up to 300 m, were determined both for synoptic and non-synoptic winds, to assess the soundness of various approaches, aiming for the confirmation or eventual correction of parameters and assumptions adopted in the proposed approach for TS wind action.

These results indicate that the method TS2 is consistent and its predictions compatible with available experimental evidence. Since size effects tend to reduce the total loads when the dimensions of the building increase, present predictions should be conservative. The expected reductions in total shear forces and bending moments in tall buildings, due to TS winds, as consequence of lack of coherence in wind pressures, are shown graphically herein. However, since dynamic amplification has the opposite effect, and none of it was incorporated in the simplified approach; additional research is necessary before introducing correction factors in the method.

The results presented in this article also confirm that, *for the same horizontal component of the reference velocity at 10 m height,* synoptic winds control the design of structures with heights exceeding approximately 150 m. Conversely, the structural design of buildings with heights below about 50 m, should be controlled by non-synoptic winds. Within the range between 50 and 150 m, the excitations due to synoptic and non-synoptic winds should in general be determined for both loading conditions.

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