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Two case studies on structural analysis of transmission towers under downburst

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Abstract. Downbursts are of great harm to transmission lines and many towers can even be destroyed. The downburst wind field model by Chen and Letchford was applied, and the wind loads of two typical transmission towers in inland areas and littoral areas were calculated separately. Spatial finite element models of the transmission towers were established by elastic beam and link elements. The wind loads as well as the dead loads of conductors and insulators were simplified and applied on the suspension points by concentrated form. Structural analysis on two typical transmission towers under normal wind and downburst was completed. The bearing characteristics and the failure modes of the transmission towers under downburst were determined. The failure state of tower members can be judged by the calculated stress ratios. It shows that stress states of the tower members were mainly controlled by 45 degree wind load. For the inland areas with low deign wind velocity, though the structural height is not in the highest wind velocity zone of downburst, the wind load under downburst is much higher than that under normal wind. The main members above the transverse separator of the legs will be firstly destroyed. For the littoral areas with high deign wind velocity, the wind load under downburst is lower than under normal wind. Transmission towers are not controlled by the wind loads from downbursts in design process.

Keywords: downburst; wind speed profile; transmission tower; wind load; failure mode

1. Introduction

Transmission tower is one of the wind-sensitive structures, and researches on structural dynamic responses and wind disaster prevention technology for transmission towers have been widely focused in the worldwide (Holmes 2008). Downburst was originally defined as a strong downdraft by Fujita (Fujita 1985, ASCE 2010). Downburst is a high intensity wind near the ground surface, which is usually induced by the impact of the subside airflow on the ground surface. For downbursts can usually produce a gust speed of 50m/s or greater, the transmission tower located in the open country will easily be damaged and even destroyed.

In many cases, the main reason of transmission tower failure is the loading arising under off-design conditions due to the actions of High Intensity Winds (HIW). The investigation results by America, Australia, and the South Africa show that about 80% to 100% of all weather-related failures of transmission towers are due to HIW (Eric *et al*. 2001). In recent years, the hazards of

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transmission tower induced by HIW especially the downburst in China appear much more frequently and seriously. For examples, on June 14th, 2005, ten transmission towers of 500kV RenShang transmission line in JiangSu province were collapsed simultaneously. On July 27th, 2007, six transmission towers of 500kV ZhengXiang transmission line in HeNan province were destroyed. On July 24th, 2009, eight transmission towers of 500kV XinPeng transmission line in HeBei province were destroyed, and the typical destroyed tower in this disaster is shown in Fig. 1.

Downburst is one of the extremely destructive HIW type, Holmes (Holmes 2005, Holmes *et al*. 2008) and some other wind-engineering researchers have demonstrated the importance of downburst in new generation standards of wind loads. However, regulations about downburst have not been put forward in most of the design standards for transmission lines (ASCE 2010, AS/NZS 2010, BS 2005, IEC 2003, DL/T 5154—2012). Only in ASCE (ASCE 2010) and Australia/New Zealand (AS/NZS 2010) standards, the design documents have begun to require designers to take account of the localized storm events including downburst. AS/NZS standard proposes a wind region zoning system for transmission lines, showing those parts of Australia for which designers need only take account of either synoptic winds or convective downdrafts, and the south and eastern coastal regions in which designers should take account of both types of event. In the design process of transmission tower, only the atmospheric boundary wind (also called normal wind) is considered in many countries. In fact, downburst has many differences from the normal wind in the profile of wind velocity, the scale characteristics and some other properties. Therefore, it is necessary to study the wind load characteristics of downburst and its effect on the bearing capacity of transmission towers.

Recently, some researchers have studied on the wind load characteristics and the structural bearing features of transmission towers under downburst (Li *et al*. 2009). Eric Savory *et al*. (2001) proposed two models for the wind velocity time-histories of transient tornado and downburst events separately, as well as the wind loads on a transmission tower. Failure modes of a tangent transmission tower were determined for these two types of HIW events. Shehata *et al*. (2005) established a numerical model of a transmission tower under downburst loading. A transmission line which suffered previously from significant damage due to a downburst event was considered as a case study. Comparison between the results of the downburst analysis and those of the normal wind were completed. It reveals the importance of considering HIW loads when attempting the structural design of transmission towers. Mara *et al*. (2016) assessed the load-deformation curve of a transmission tower under downburst wind loading, and compared it with that obtained for a normal wind loading profile. Based on Chen and Letchford's downburst model (Chen and Letchford 2004) and quasi-steady assumption, Lou *et al*. (2009) proposed a horizontal force model of moving downburst acting on transmission towers. The features of downburst impact on a typical transmission tower were studied. Influence of the characteristic parameters of downburst on the axial forces of a transmission tower was especially discussed by a sensitivity analysis of downburst model parameters. Qu and Ji (2013) systematically studied the features of the downburst wind field, the wind load simulating methods, as well as the catastrophic effects on transmission lines. Wang and Qu (2009) studied the failure modes of transmission towers under downburst based on the ultimate moment method. These research results can provide important theoretical method and reference for studying the bearing capacity and failure modes of transmission towers under downburst. However, the design values and the calculation method of wind load under downburst have not been proposed for transmission lines. The recent regulations cannot provide sufficient guidance for the engineers and designers of transmission lines.

Fig. 1 Failure mode of the typical tower

Based on the design regulations on the transmission lines in high intensity wind (HIW) areas given by ASCE, the design load values under downbursts for transmission lines were proposed. The downburst wind field model proposed by Chen and Letchford was applied, and the wind loads of two typical transmission towers in inland areas and littoral areas were calculated separately. Spatial finite element models of the transmission towers were established and the structural analysis was completed. The bearing characteristics and the failure modes of the transmission towers under downburst were determined.

2. Wind field model of downburst

Chen and Letchford (2004) modified a shortcoming of Holmes and Oliver's model (2000) not includes turbulent fluctuations, which may cause considerable additional response of structures. Then a deterministic-stochastic hybrid model of downburst was proposed. The mean velocity of downburst at anytime in any height can be factorized as the product of a vertical profile and a time function as follows.

$$U(z,t) = V(z) \times f(t) \tag{1}$$

where V(z) is the vertical profile of the maximum mean wind velocity. $f(t)$ is a time function with its maximum value of 1.0, which describes how the mean velocity evolves with time. $f(t)$ is relative to some factors including the moving velocity, the trace and the radial wind velocity of the downburst.

Based on the wind tunnel test and CFD simulation, Oseguera and Bowles (1988), Wood and Kwok (1998), Vicroy (1991) proposed an empirical model for describing the vertical profile of downburst separately. The Vicroy model used in this study is expressed as

$$V(z) = 1.22 \times \left[e^{\left(-0.15\frac{z}{z_{\max}}\right)} - e^{\left(-3.2175\frac{z}{z_{\max}}\right)} \right] \times V_{\max} \tag{2}$$

where $V(z)$ is the maximum mean wind velocity at height z. $V_{\max}$ is the maximum velocity in the vertical profile, a normal value is usually 80 m/s. $z_{\max}$ is the height at which the maximum velocity occurs, a normal value is usually 70 m.

Parameters of the normal wind are determined according to Davenport’s suggestions in suburb areas. The roughness degree of the ground surface is with a value of 0.16. The gradient level of the wind field is with a value of 300 m. The vertical profiles of the three classical downburst models and the normal wind model are compared in Fig. 2.

3. Wind load of downburst

The ASCE and AS/NZS standards have begun to consider the HIWs in transmission line design. The HIWs include tornados, downburst and microburst, etc. The wind loads acting on transmission tower, the conductor and the ground wire are regulated in these standards.

3.1 ASCE guidelines

Tornados, downburst and microburst are the HIWs discussed in ASCE guidelines. As we know, the transmission lines will be seriously destroyed in some stochastic locations under tornado and downburst. However, the occurring locations of HIWs are uncertain and the wind velocity is very high, by considering the economic and safe factors comprehensively (Magdi and Brian 1995), American power administrations have indicated that it isn't necessary to defend HIWs in a range of whole transmission lines. Combination of the wind field and the moving path of tornado and downburst are very complex. Until now, there is no proper engineering wind field model for design of transmission lines. ASCE proposes a method to calculate the wind loads of transmission lines under tornados rated at F_1 or F_2. For F_2 scale tornados with the highest frequency in America, the three-seconds gust wind velocity in is the range of 50.5 m/s to 70.2 m/s. Only the wind loads acting on the tower structure are considered in the structural design. One possible tornado loading is a wind loading corresponding to a moderate tornado (scale F_1 or F_2) applied only to the transmission structure over the full structure height from any direction with a consistent velocity. It can be assumed that tornado loading applied to the conductors and the ground wires is neglected, because the tornado path widths are small which usually are 60 m to 150 m, and the wind force mechanism applied to the conductors and the ground wires are complex.

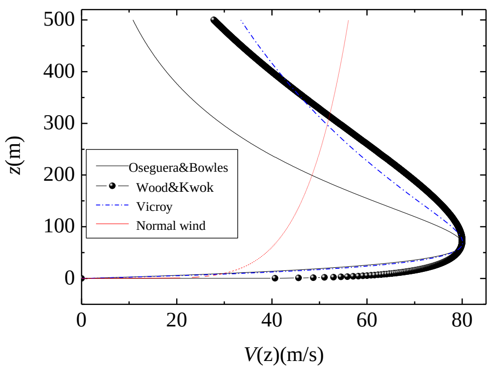


Fig. 2 Comparison of the vertical wind profile

Downburst are usually associated with the more severe thunderstorm cells and seldom reach the intensity levels or wind velocities of F_2 scale tornados, but they can have relatively wide gust fronts so that two or three spans may be affected. Downbursts are usually evidenced by elliptical damage patterns to vegetation. Normal gust width of microburst is expected to be 100 m to 200 m (ASCE 2010). The normal procedure for providing protection or defense against downburst effects is either application of tornado-type narrow front loadings described above, or simply relying on the extreme wind loadings as normal wind with a gust response factors closer to 1.0. The economy of including HIW load cases in the structure designs will depend on the other local loads. Structures designed for light winds and little ice might not be made to withstand HIW economically, but structures already designed for high wind or heavy ice might require just a small cost increase to include HIW loads.

Under the HIWs including downburst, the wind load acting on the surface of transmission towers can be determined by Eq. (3).

$$F = QK_z K_{zt} V^2 G_t C_f A_m \tag{3}$$

where Q is a numerical constant with the value of 0.613. K_z is the velocity pressure exposure coefficient. K_{zt} is the topographic factor. G_t is the gust response factor for transmission towers. V is the three-seconds gust design wind velocity, in m/s. C_f is the drag force coefficient. A_m is the area of all members normal to the wind direction, in m^2.

3.2 AS/NZS standards

Be different from the regulations in ASCE guidelines, AS/NZS standards propose that both the wind actions on tower structures and the wind actions on conductors and ground wires should be considered in the structure design. The wind load acting on tower structures can be calculated by Eq. (4).

$$F_s = 0.6(VM_{z,cat} M_s M_t)^2 C_d A \tag{4}$$

where V is the three-seconds gust design wind velocity, in m/s. $M_{z,cat}$ is the gust wind velocity multiplier for terrain category at height z, and it can be determined according to Eq. (5). M_s is the shielding multiplier which is valued as 1.0. M_{td} is the topographic multiplier for gust wind velocity. C_d is the drag force coefficient. A is the area of all members normal to the wind direction, in m^2.

$$M_{z,cat} = \begin{cases} 1.0 & 0 < z \le 50 \\ 1.0 - 0.5(z-50)/50 & 50 < z \le 100 \\ 0.5 & z > 100 \end{cases} \tag{5}$$

$$M_{td} = 1 + 0.5\,(M_{ts} - 1) \tag{6}$$

where M_{ts} is the topographic multiplier of gust wind velocity for normal wind which also called synoptic wind in AS/NZS standards.

The calculation method for the wind loads on conductors or ground wires under downburst is same to the normal wind, except for the span reduction factor (*SRF*). The *SRF* can be determined by Eq. (7).

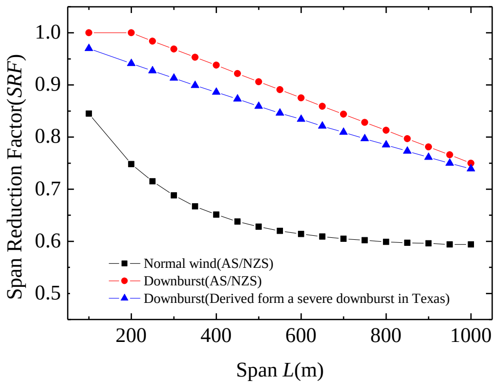


Fig. 3 Comparison of the span reduction factor (*SRF*)

$$SRF = \begin{cases} 1.0 & L \le 200 \\ 1.0 - (L - 200) \times 0.3125/1000 & 200 < L \le 1000 \end{cases} \tag{7}$$

For tension calculations on tension sections with the length greater than 1000 m, the *SRF* should be determined by the method for normal wind. The *SR*F curves with different span lengths for downburst and normal wind are compared in Fig. 3. It can be seen that the *SRF* values in downburst are greater than those calculated for normal wind. The main reason (by Holmes 2008) for this is that there is a large underlying 'running-mean' component which is nearly fully correlated over large separation distances. Fig. 3 shows that the effective reduction for a span of 400 m is only about 10%.

3.3 Wind loads of downburst

From Fig. 2, it can be seen that the horizontal wind velocity of downburst is higher than 50 m/s when the reference height is higher than 10 m. If the wind load actions on the conductors and the ground wires are considered, the cost of the transmission lines will be increased by a large amount. Furthermore, as discussed in Section 3.1, the occurring location of downburst is very stochastic and non-repeated, it is difficult to realize that the whole transmission line can fully defend downburst impact. In the regions which the downburst happens frequently, it is reasonable to only consider the wind loads acting on transmission towers in the process of checking the bearing capacity. By this method, the anti-wind capacity of transmission towers may be enhanced by a proper extent, as well as the economic requirement can also be balanced. So, this method is a feasible means for calculating the wind loads under downburst.

The wind action effects of downburst on transmission lines are related to some factors including the initial location, the moving track, the translation velocity and the wind field scale, etc. The relative distance between the downburst center and the transmission tower is uncertain. For the design of transmission lines in the downburst regions with a high occurring probability, it is necessary to consider that the downburst with maximum velocity may experience any transmission tower.

The gust response effect of HIWs including downburst is not considered in ASCE guidelines and AS/NZS standards. The effect of fluctuating wind on the structural dynamic responses is considered by the wind vibration coefficient β_z in China standards. Lou *et al.* (2009) completed the dynamic analysis on a 178 meters high transmission tower under downburst, and the wind vibration coefficient based on the structural dynamic displacements is about 1.40, which is lower than the value corresponding to normal wind. There are no widely approved conclusions on the wind vibration coefficient under downburst, so the values for normal wind will be applied in this analysis.

As the explanations above all, the wind loads acting on transmission towers under downburst can be calculated by Eq. (8).

$$W_s = \frac{U^2(z)}{1600} \times \mu_s \beta_{zd} A_s \tag{8}$$

where $U(z)$ is the mean wind velocity of downburst at height z, in m/s. The Vicroy model is used to describe the vertical profile of downburst. μ_s is the drag coefficient of the tower members. β_{zd} is the wind vibration coefficient under downburst, which is valued as the normal wind(DL/T 5154—2012). A_s is the area of all members normal to the wind direction, in m^2.

For yawed wind, k_θ is applied to consider the wind load component in the longitudinal direction and the transversal direction, which is defined as the factor for angle of incidence θ wind to transmission line. k_θ can be calculated by Eq. (9).

$$k_\theta = 1 + 0.2\sin^2 2\theta \tag{9}$$

4. Case studies on struntural anlysis of two typical transmission towers

Until now, only few downburst atmospheric process at or near the disaster sites have been caught. Based on the radar monitoring results and research results in recent years, an assumed downburst process was applied in this study. As discussed by Chen and Letchford (2004), when the moving time is up to about 170s, the combination wind velocity is up to the maximum value 58.89 m/s. The time history function $f(t)$ is 1.0 at this time.

Based on the calculation method approved in Section 3.3, the wind loads of two typical transmission towers in inland areas (110 kV) and littoral areas (500 kV) were calculated separately. Spatial finite element models of the transmission towers were established and the structural analysis was completed. The general FEA software ANSYS was applied for the tower structural analysis with foundation deformation. The leg members and the secondary members of the tower are simulated by BEAM4 element and Link8 element, respectively. The wind loads as well as the dead loads of conductors and insulators are simplified and applied on the suspension points by concentrated form. The bearing characteristics and the failure modes of the transmission towers under downburst were determined. It is assumed that the member would be destroyed when its stress is higher than the design stress. The buckling failure state can be considered by stability coefficient of steel members. The calculation method in Technical Regulation of Design for Tower and Pole Structures of Overhead Transmission Line (DL/T 5154—2012) was used to determine the critical axial stress of axially compressed members.

4.1 Case I: 110 kV transmission tower in inland areas

The first analysis object is the No.18 tangent transmission tower of 110 kV TangDa transmission line. As shown in Fig. 4, this tower was collapsed on 24th August, 2012. The terrain around the tower is an opening area. Some pounds are located in this area. The micro weather process such as localized convention may be easily formed. According to monitoring data of the lighting locating system managed by the power department, high intensity convention weather happened near the destroyed towers. Furthermore, some transmission towers of the end-of-life transmission lines were destroyed due to HIWs in these areas in 2008 and 2010. The conductor type of this transmission line is LGJ-240. The ground wire type of is GJ-50. The design wind span length and vertical span length is 350 m and 600 m, respectively. The design wind velocity at the height of 10 m every ten minutes is 23.4 m/s.

The height of the No.18 tangent transmission tower is 30.3 m. The profile and wind pressure subsections of the tangent tower are presented in Fig. 5. The real span lengths of the conductors and the ground wires are used in this analysis. The finite element analysis (FEA) model was established by spatial beam and truss elements. Firstly, according to the regulations in Technical Regulation of Design for Tower and Pole Structures of Overhead Transmission Line (DL/T 5154—2012), it is assumed that the transmission tower is under the normal wind and the wind velocity is design wind velocity. The forces and stress states of some main members of the transmission tower was calculated. The wind load cases include zero-degree wind, 45-degree wind, 60-degree wind and 90-degree wind. For an arbitrary wind direction with respect to the face of the transmission tower, the wind incidence factor is calculated according to Eq. (9), which is same to the regulations in IEC standard(IEC 2003).

The compression and tension bearing capacities of the main members at the tower body and tower legs are controlled by the 60-degree wind load case. The calculated axial forces, the compressive stress as well as the stress ratios are listed in Table 1. The stress ratio is defined as the ratio of the calculated stress to the design stress of steel members. The design stress of 16Mn and A3 type steel is 310MPa and 215MPa, respectively. The design wind velocity of the transmission line is 23.4 m/s, which is higher than the maximum wind velocity recorded by the meteorological department locating near the disaster points. The maximum wind velocity record is 20.7 m/s, which is equivalent to the wind intensity of eight Beaufort scale. Under the wind load corresponding to design wind velocity, the calculated stresses of the main members are not exceeding the design stress. Therefore the transmission may not be destroyed under the normal wind. The acting of the localized thunderstorm will probably be the main reason for the damage of the transmission tower. Then the bearing capacity analysis of No.18 tangent tower under downburst was carried out.

By adopting the Vicroy model with $V_{\max}$ =80 m/s and $z_{\max}$ =70 m, the wind velocities at center height z^* for different wind pressure segments of No.18 tangent tower are listed in Table 2. According to the regulations of ASCE Guidelines for Electrical Transmission Line Structural Loading, the time interval of the maximum velocity $V_{\max}$ of downburst is 3 seconds, and it is expressed by V_3. In order to be consistent with the regulations of normal wind in China standards, the time interval of downburst wind velocity can be changed to 10 minutes equal to 600 seconds, and the wind velocity can be written as V_{600} (Huang and Wang 2008). Then the wind loads acting on the transmission tower were calculated by China standards.

(a) (b)

Fig. 4 Collapsed state of the No.18 tower: (a) local view and (b) general view

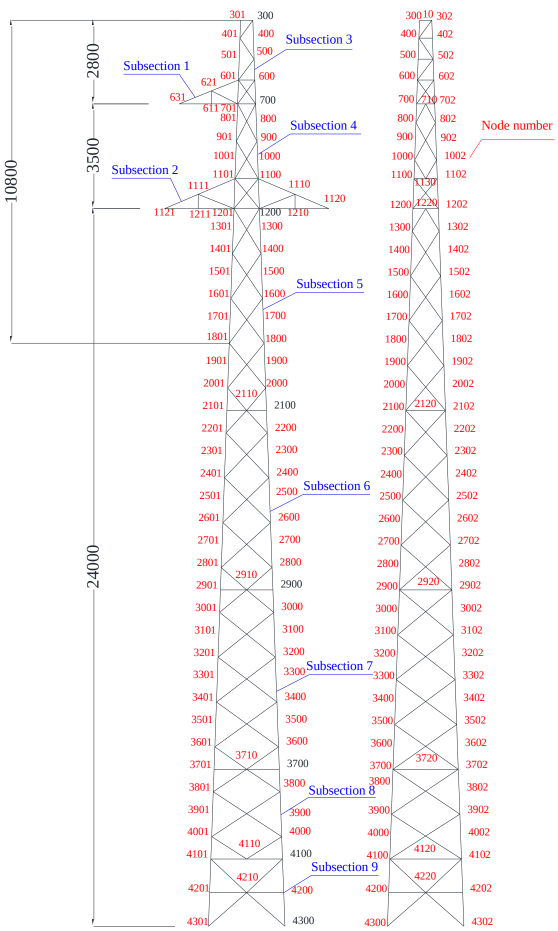


Fig. 5 Profile and wind pressure subsections of the tangent tower (Unit:mm)

Table 1 Axial forces, stresses and stress ratios of the main members of tower body and tower legs (normal wind)

Node number	Section type	Compression force/kN	Tension force/kN	Compressive stress/MPa	Stress ratio
3600-3700	L75×5-16Mn	−111.14	98.98	−252.7	0.86
3700-3800	L75×5-16Mn	−112.34	98.65	−255.5	0.87
3800-3900	L75×5-16Mn	−116.25	102.40	−264.3	0.90
3900-4000	L75×5-16Mn	−119.74	105.71	−272.3	0.92
4000-4100	L75×5-16Mn	−123.38	109.20	−280.6	0.95
4100-4200	L75×5-16Mn	−105.14	94.32	−230.9	0.78
4200-4300	L75×5-16Mn	−105.14	94.32	−230.9	0.78

Table 2 Wind velocity at the centric points of each wind pressure subsection (m/s)

z^* /m	27.9	24.5	28.9	25.8	20.6	14.3	8.3	3.8	1.1
V_3	64.9	61.0	65.9	62.5	55.6	44.0	29.1	14.7	4.7
V_{600}	45.6	42.9	46.3	43.9	39.1	30.9	20.5	10.3	3.3

The wind velocity of 25 m/s at 10 meters height with a time interval of 10 minutes was selected as the reference wind velocity. The ratios of the wind pressure height coefficients μ_{zd} under downburst to the wind pressure height coefficients μ_z under normal wind were computed and presented in Fig. 6. For different wind pressure segments of the tower body, the ratios of the wind loads are equal to the ratios of the wind pressure height coefficients. In current standards, the wind velocity at the height lower than 10m is valued as the wind velocity at 10 meters height. Therefore, as to the wind pressure segments with a centre height lower than 10 m, the wind loads under downburst are lower than those of the normal wind.

The calculated axial forces, the compressive stresses as well as the stress ratios of No.18 tangent tower under downburst are listed in Table 3. Under the wind load of downburst, all the calculated stresses of the main members are exceeding the design stress. The controlled load case is 45-degree wind. The maximum exceeding percentages of the stresses are 26%. The stress ratios of the main member 3900-4000 and 4000-4100 are higher than other members. These two main members will be firstly destroyed under downburst. Comparing to the collapsed state shown in Fig. 4, the vertical distance from the damaged point to the tower foots is about 3~4 meters, it can be seen that the destroyed locations from finite element analysis are consistent with the real state.

The ratios of the main member stresses under downburst to the stresses under normal wind are listed in Table 4. The ratios are in the range of 1.56 to 2.78 when the centre height is higher than 10 meters. As discussed above, the control load cases of the main members at the tower body are 60-degree wind and 45-degree wind for normal wind and downburst, respectively. Under two types of wind load cases, the longitudinal loads, the transversal loads, the shear forces as well as the moments for different wind pressure segments were calculated separately. The combined vectors of shear forces and the vector moments at different heights are presented in Fig. 7. It can be seen that when the height is higher than the point which is 10 m from the tower top in vertical direction, the combined forces from the wind loads of conductors, ground wires and tower body under normal wind are a little higher than the combined forces from the wind loads tower body under downburst. When the height is lower than this point, the combined forces under downburst are higher than the normal wind.

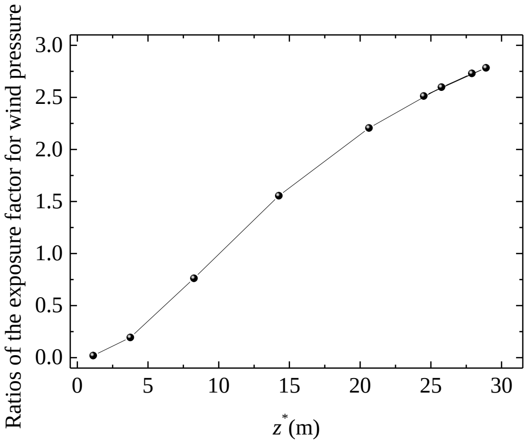


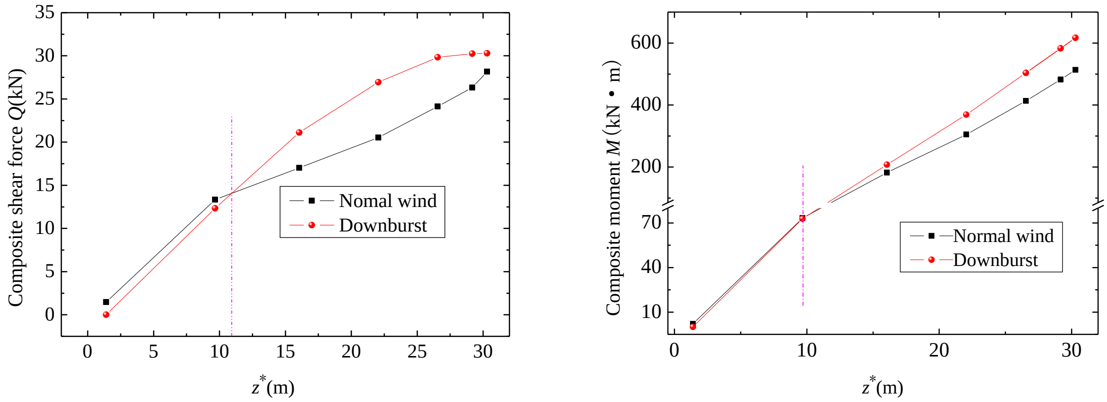
Fig. 6 Ratios of the wind pressure height coefficients

Table 3 Axial forces, stresses and stress ratios of the main members at tower body and tower legs (downburst)

Node number	Section type	Compression force/kN	Tension force/kN	Compressive stress/MPa	Stress ratio
3600-3700	L75×5-16Mn	−148.36	139.43	−337.4	1.15
3700-3800	L75×5-16Mn	−151.61	141.29	−344.7	1.17
3800-3900	L75×5-16Mn	−155.05	144.47	−352.6	1.20
3900-4000	L75×5-16Mn	−159.89	149.35	−363.6	1.23
4000-4100	L75×5-16Mn	−163.10	152.31	−370.9	1.26
4100-4200	L75×5-16Mn	−139.49	131.04	−306.4	1.04
4200-4300	L75×5-16Mn	−139.49	131.04	−306.4	1.04

Table 4 Comparison on the ratios of main member stresses

Node number	Section type	Ratios of member stresses
500-600	L56×5-A3	0.75
800-900	L56×5-A3	0.65
1500-1600	L63×5-A3	1.31
2400-2500	L70×5-A3	1.29
3200-3300	L70×5-16Mn	1.33
3600-3700	L75×5-16Mn	1.34
3700-3800	L75×5-16Mn	1.35
3800-3900	L75×5-16Mn	1.33
3900-4000	L75×5-16Mn	1.34
4000-4100	L75×5-16Mn	1.33
4100-4200	L75×5-16Mn	1.33
4200-4300	L75×5-16Mn	1.33



(a) (b)

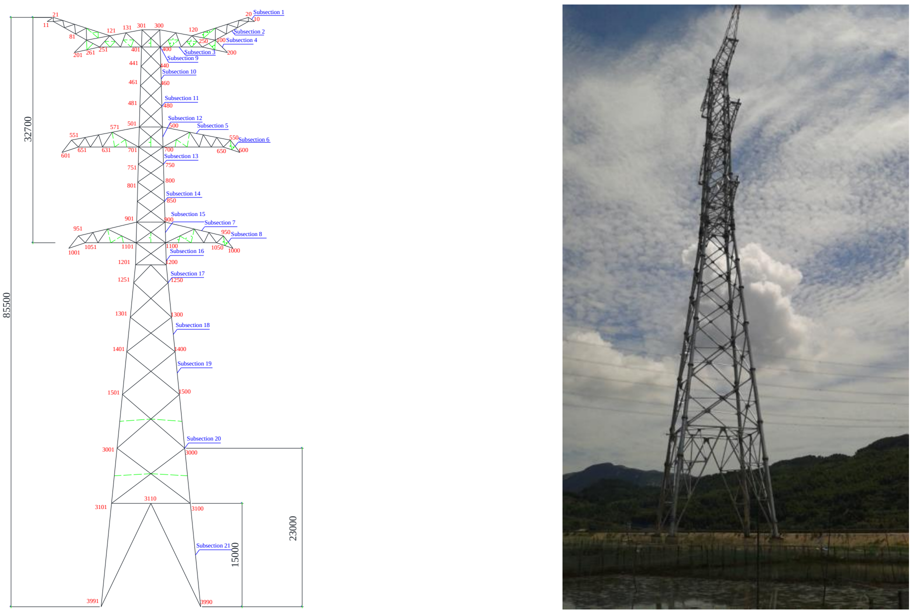
Fig. 7 Distribution of the shear force and the moment: (a) Shear force and (b) Moment

In the linear elastic range, the stresses of tower members are proportional to the wind loads acting on tower body. In Table 4, under two types of wind loads, the ratios of the main member stresses located above the cross arm are 0.75 and 0.65. It demonstrates that the stresses under normal wind are higher than those values under downburst. The ratios of the main member stresses located under the cross arm are in the range of 1.33 to 1.35. It demonstrates that the stresses under downburst are higher than those values under normal wind. However, the stress ratios are much lower than the wind load ratios, which are from 1.56 to 2.78 in Fig. 6. The main reason is that the wind loads from conductors and ground wires are ignored for downburst case. Especially for the normal wind case, the wind loads from conductors and ground wires occupy 36.8% of the total wind loads acting on the transmission tower.

4.2 Case II: 500 kV transmission tower in littoral areas

The second analysis object is the No.52 tangent transmission tower of 500 kV Ningde-LiLi transmission line. The design wind velocity at the height of 10 m is 42 m/s. The conductor type of this transmission line is JL/LB20A-720/50. The ground wire type is JLB35-150. The design wind span length and vertical span length is 650 m and 1100 m, respectively. The height of the No.52 tangent transmission tower is 85.5 m. The profile and wind pressure subsections of the tangent tower are presented in Fig. 8. The design span lengths of the conductors and the ground wires are used in this analysis.

The calculated method of the wind load acting on transmission tower in Section 4.1 was applied. The analyzed transmission tower is located in littoral areas with typhoon occurrences and the design wind velocity is very high (42 m/s). Therefore, in the altitude range not higher than the total tower height, the wind velocity under downburst is near or lower than the normal wind. The ratios of the wind pressure height coefficients μ_{zd} under downburst to the wind pressure height coefficients μ_z under normal wind were computed and presented in Fig. 9. For different wind pressure segments of the tower body, the ratios of the wind loads are equal to the ratios of the wind pressure height coefficients.



(a) (b)

Fig. 8 Wind pressure subsections and photo of the river crossing tower (Unit: mm): (a) Wind pressure subsections and (b) photo

Under the wind load of normal wind, the main members at tower body and tower legs are controlled by 60-degree load case, while the diagonal members at tower body are controlled by 90-degree load case. The calculated axial forces, the compressive stress as well as the stress ratios are listed in Table 5. The calculated stresses of the main members and the diagonal members are not exceeding the design stresses. The maximum stress ratios of the main members and the diagonal members are 0.84 and 0.94, respectively.

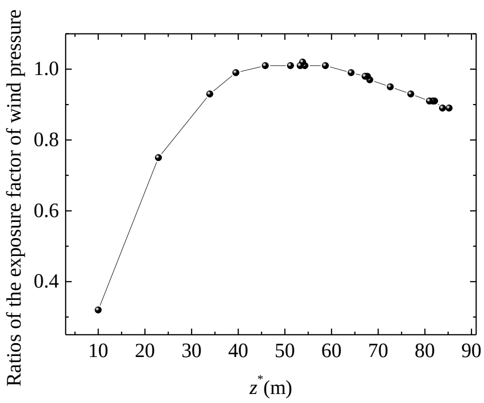


Fig. 9 Ratios of the wind pressure height coefficients

Table 5 Axial forces, stresses and stress ratios of the main members of tower body and tower legs (normal wind)

Node number	Section type	Compression force/kN	Tension force/kN	Compressive stress/MPa	Stress ratio
440-460	◎219×6-Q345	−410.60	358.53	−116.5	0.38
460-481	L125×10-Q345	−844.92	656.15	−239.7	0.77
500-700	◎325×8-Q420	−313.96	266.45	−287.9	0.93
800-850	◎426×10-Q420	−2047.62	1776.11	−279.6	0.74
800-852	L125×10-Q345	−3762.53	3402.86	−302.5	0.80
900-1101	L180×16-Q345	−287.17	297.04	−290.7	0.94
1250-1300	◎529×14-Q420	−386.29	391.23	−282.8	0.91
3100-3990	◎610×16-Q420	−6548.83	5914.02	−317.7	0.84

By adopting the Vicroy model with V_{max} =80 m/s and z_{max} =70 m, the wind velocities at centre height z^* for different wind pressure segments of No.52 tangent tower are listed in Table 6. Under the wind load of downburst, the calculated axial forces, the compressive stress as well as the stress ratios the main members are listed in Table 7. The control load case is 45-degree wind. The design stress of Q345 and Q420 type steel is 310MPa and 380MPa, respectively. Under downburst, the wind loads acting on tower body is near or lower than the normal wind. Furthermore, the wind loads from conductors and ground wires are ignored. However, for 90-degree wind case, the wind loads from conductors and ground wires occupy 54.7% of the total wind loads acting on the transmission tower. Therefore, the calculated stresses are lower than design stresses and the maximum stress ratio is only 0.68.

Table 6 Wind velocity at the centric points of each wind pressure subsection (m/s)

z^* /m	85.2	83.8	82.1	81.7	67.7	67.2	53.8	53.3	81.0	77.0	72.6	68.2	64.2	58.7	54.3	51.2	45.8	39.5	33.9	22.9	10.0
V_3	79.4	79.5	79.6	79.6	80.1	80.1	78.7	78.6	79.7	79.9	80.1	80.1	80.0	79.5	78.8	78.2	76.6	73.8	70.2	58.9	33.9
V_{600}	55.8	55.9	56.0	56.0	56.3	56.3	55.4	55.3	56.0	56.2	56.3	56.3	56.2	55.9	55.4	55.0	53.9	51.9	49.4	41.4	23.8

Table 7 Axial forces, stresses and stress ratios of the main members of tower body and tower legs (downburst)

Node number	Section type	Compression force/kN	Tension force/kN	Compressive stress/MPa	Stress ratio
440-460	◎219×6-Q345	−410.60	358.53	−116.5	0.38
460-481	L125×10-Q345	−87.23	82.74	−80.0	0.26
500-700	◎325×8-Q420	−1061.54	969.62	−144.9	0.38
800-850	◎426×10-Q420	−2042.01	1872.27	−164.2	0.43
800-852	L125×10-Q345	−209.23	209.56	−211.8	0.68
900-1101	L180×16-Q345	−193.60	190.85	−141.7	0.46
1250-1300	◎529×14-Q420	−3678.09	3370.45	−178.5	0.47
3100-3990	◎610×16-Q420	−4687.34	3737.37	−169.1	0.45

Table 8 Comparison on the ratios of main member stresses

Node number	Section type	Ratios of member stresses
440-460	◎219×6-Q345	0.49
460-481	L125×10-Q345	0.28
500-700	◎325×8-Q420	0.52
800-850	◎426×10-Q420	0.54
800-852	L125×10-Q345	0.73
900-1101	L180×16-Q345	0.50
1250-1300	◎529×14-Q420	0.56
3100-3990	◎610×16-Q420	0.62

Under two types of wind loads, the ratios of the main member stresses are listed in Table 8. For the transmission line in littoral areas, the design wind velocity is relatively high and valued as 42 m/s. the wind loads acting on tower body under downburst are near or lower than the normal wind. Furthermore, the wind loads from the conductors and ground wires are ignored. Therefore, the ratios of the main member stresses under downburst and normal wind are from 0.28 to 0.73.

5. Conclusions

Until now, there are no special standards for the design of transmission lines in downburst regions. The velocity of downburst in the vertical profile is different from the normal wind especially in the main height range of transmission towers. In this paper, the downburst wind field model by Chen and Letchford was applied, and based on the design regulations on the transmission lines in HIW areas by ASCE, the calculating method of wind loads acting on transmission towers was proposed. Structural analysis of two typical transmission towers in inland areas and littoral areas were carried out separately. The wind loads as well as the member stresses under downburst and normal wind were compared. Three main conclusions are listed as follows.

- Finite element models of two typical transmission towers were established by elastic beam and link elements. The wind loads as well as the dead loads of conductors and insulators are simplified and applied on the suspension points by concentrated form. The failure state of tower members can be judged by the calculated stress ratios.
- By considering the scale features of downbursts and the economic design principle of transmission lines, the moving process of the downburst should be ignored. The time history factor can be valued as 1.0 in the design of transmission lines. The downburst loading applied to the conductors and the ground wires is neglected. Only the wind loads acting on tower body are considered.
- Under the wind load of downburst, all the calculated stresses of the main members are controlled by 45-degree wind case. For the inland areas with low deign wind velocity, though the structural height is not in the highest wind velocity zone of downburst, the wind load under downburst is much higher than under normal wind. The main members above the transverse separator of the legs will be firstly destroyed. The destroyed locations from finite element analysis are consistent with the real state. For the littoral areas with high deign wind velocity,

the wind load under downburst is lower than under normal wind. Transmission towers are not controlled by the wind loads from downbursts in design process.

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