Wind Loading: Uncertainties and Honesty Suggest Simplification

Alain H. Peyrot, PhD, PE[1]

[1] Emeritus Professor of Civil Engineering, Univ. of Wisconsin, Madison, WI, USA
Past President (1992 to 2008), Power Line Systems, Madison,WI, USA
apeyrot@powline.com

ABSTRACT

This paper discusses some of the recent wind loading procedures for the design of transmission lines (NESC, ASCE, IEC, CENELEC, etc.). It provides some detailed background behind their formulas for gust factors, span factors and gust response factors. It discusses the uncertainties inherent in each one of the parameters and assumptions behind the formulas: wind storm type, reference wind, terrain roughness, profiles, gustiness, time and spatial correlations, dynamic response, wind direction, drag coefficients, etc. It provides some examples to illustrate the current lack of consensus and to identify the important design parameters. The paper concludes that the complexity of most current wind design procedures is not justified. Instead, it provides the rationale for simplifying the entire wind design process and it offers specific recommendations for achieving that goal.

1. INTRODUCTION

The theoretical bases of the extreme wind provisions of some of the current generation of codes, standards or guides dealing with overhead transmission lines (NESC, 2007; CENELEC, 2001; IEC, 2003; UK NNA, 2004; ASCE Manual 74, 2008; ASCE Manual 113, 2008; etc.) were developed in the 1960's and early 1970's by a very small number of individuals with very sophisticated mathematical skills (Davenport, 1961 and later; Harris, 1963; Manuzio et al, 1964; Castanheta, 1970; Cojan, 1973; Armitt et al, 1975). We will refer to all the above-mentioned documents as "codes" even when they do not have that status. The mathematical bases of the wind models and associated structural responses borrowed from the random vibrations field (Crandall, 1963) and communication theory. For three decades, these provisions were published in draft documents and guides, but were not parts of legally binding documents. Therefore, they were mostly ignored. However, once the NESC in the US adopted Rule 250C in 2002, once the CENELEC document was finalized in 2001 and the IEC issued its Standard in 2003, it was no longer possible to ignore these provisions and they were finally tested by practicing engineers. This actual implementation revealed many problems and generated quite a few debates. In fact, the Europeans could never agree on a single approach and the CENELEC standard has an escape clause that allows each country to basically do what it wishes (i.e. publish a National Normative

Aspects, or NNA) as long as the general nomenclature is respected. This is what most of them do.
This writer is in a unique position to comment on the recent wind provisions as: 1) he has conducted academic research and taught graduate level and professional classes in wind engineering, reliability-based design and transmission line design, 2) he has been a member of some of the CIGRE (CIGRE, 1990) and ASCE (ASCE Manual 74, 1984, 1991, 2008) committees that developed the recent wind provisions, 3) he has worked as a transmission line consultant for many years and investigated line failures, and 4) he has participated in the implementation of the wind provisions of many international codes and standards in his company's design software (PLS-CADD & TOWER) that is used in over 100 countries. In doing so he has sometimes uncovered provisions that were simply untested, incomplete and confusing. Being confronted with so many ways of approaching the common problem of designing safe lines economically and practically, has led this writer to reflect on what we are doing, what is really justified and to suggest some simplifications. This is the rationale for writing this paper and hopefully convincing future generations that: 1) there is nothing sacred behind the current code procedures, 2) the complexity of the problem is not amenable to its description by fancy equations, and 3) simpler formulations should be considered.

Refining the wind loading equations has been part of the larger goal of improving design by what is now commonly known as Reliability Based Design (RBD). There was even some hope in the early days of RBD development, that given a sufficient amount of research and data collection, our industry would someday be able to quantify the probability of failure of a line (for example statistically determine the extent and number of failures of a line over a period of time). However, in part because of the uncertainties described in this paper, other uncertainties related to the size of the storms and the corresponding number of exposed structures, we will never be able to achieve this lofty goal. In spite of this shortcoming (inability to quantify the probability of failure), RBD is still a very valuable guide to develop consistent design procedures covering various combinations of loading events and materials (CIGRE, 1990 and 2006; ASCE Manual 111, 2006; Ghannoum, 2002; Mozer et al, 1984; Peyrot et al, 1984). Some concepts of RBD and simplicity are not incompatible.

2. TYPES OF WIND STORMS

For the proper understanding of the various engineering approaches to the determination of wind loads on transmission lines, it is imperative to have some knowledge of the various types of wind storms that may be damaging to our lines. Recommended reading on this subject are CIGRE Brochures 256 (2004), 344 (2008) and 350 (2008). Here we will limit our discussion to winds with gusts in the range of 40 to 60 m/s (about 90 to 140 mph) that cover at least two spans. Tornadic winds, while extremely violent, generally cover less than two spans and their peak velocity values are generally not considered in the statistics that form the basis of the "Basic/ Reference Wind" maps produced around the world. While tornadic winds can be considered in design, for example as explained in ASCE Manual 74 (1991 and later), they will not be discussed in this paper.

2.1 Extra Tropical Cyclones/ Winter Storms

These storms are characterized by circulating winds around very low pressure zones: they are also referred to as cyclonic systems with very large diameters, say between 500 and 3000 km. They commonly occur in North America and Europe in winter. They are the most studied storms with a well-defined "boundary layer" behavior and have been the basis of the "Academic Winds" discussed in Section 2.5. While they are generally regarded as the storms that cause most transmission line failures in Western Europe, this is not the case in most other regions of the world.

For such winds, the motion of the air at heights above the boundary layer (at heights higher than the Gradient Heights generally considered to be higher than 250 m) is essentially parallel to the isobars and its velocity is referred to as the Gradient, Geostrophic or Synoptic Wind.

2.2 Tropical Cyclones

These are the well-known hurricanes/ typhoons/ cyclones that affect coastal areas during warm seasons. Very high winds exist near the low pressure center of the storm (the eye). Tropical cyclones can be very damaging to lines due to their extent, duration and propensity to carry debris.

2.3 Local Storms

These include a large variety of storms from the classical convective cell thunderstorms to the squall line winds and downbursts (macroburst and microburst) that occur near advancing cold fronts. To distinguish them from those due to Extra-Tropical or Tropical Cyclone winds, such winds have also been referred to as High Intensity Winds or HIW. While local storms are smaller in size than winter storms, they are more frequent, and according to several authors (Dempsey et al,1996; de Oliveira, 2006), including the individual who produced the non-hurricane portion of the latest NESC and ASCE wind maps used in the US (Peterka, 1998 and 2005), and according to CIGRE Brochures 256 and 350 (CIGRE, 2004 and 2008), they are the cause of most wind failures in the USA, the central Canada, South America, South Africa and Australia.

2.4 Katabatic and Downslope Winds

Katabatic winds develop on the leeward side of mountains or ridges when the air approaching on the windward side is colder and flows downhill into warmer valleys due to its higher density. Downslope winds can also be caused by dry warm air forced down mountain sides by strong winds aloft. The various "Foen" winds of Europe and the "Chinook" wind in the USA are well-known examples of downslope wind.

2.5 Academic winds

Given the wide variety of storm types and the fact that no two storms are alike (and that their characteristics may even be affected by global climate change), it is understandable that it will never be possible, even statistically, to accurately predict future occurrences of winds, including their spatial and temporal variations. However, some attempts have been made and we will refer to them as the "Academic Winds" to emphasize the big differences between real future storms and the elegant idealized equations that attempt to characterize them.

The academic winds that have been proposed for transmission lines can only be justified for winter storms and maybe tropical storms (Sections 2.1 and 2.2). An academic wind assumes that, for an averaging time period, Tav, of at least 10 minutes, and sometimes 1 hour, the wind velocity at height "z" above the ground is the sum of a mean value $V_m[z]$ (a single random variable) plus a zero-mean fluctuating value $V_f[z,t]$ (the turbulence) as shown in Fig. 1. The fastest wind value is the peak gust $V_g[z]$. Actually, given the precision of everything related to wind, the peak gust or the often quoted 2-sec or 3-sec gusts are equivalent and will simply be referred to as the "peak gust" or simply "gust" in this paper. The gust is the maximum wind that a structure or a very short span experiences. While Fig. 1 depicts the variation over time of the wind velocity at one point, it could also be interpreted as the spatial variation of the wind velocity along the centerline of a transmission line at one instant of time.

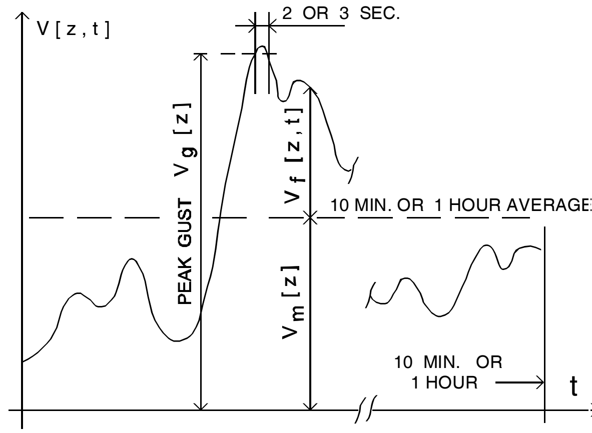

Figure 1 Wind over averaging period

2.5.1 The mean value

In the context of a winter storm, it can be argued that if you are far enough above the ground (above the gradient height), there is very little turbulence and the wind velocity is not affected by the retarding effect of the roughness of the ground (i.e., when z is higher than the gradient height, $V_f[z,t]$ is equal to zero and $V_m[z]$ is constant and equal to the gradient speed). Based on experimental observations and considering the need for simplification, an academic wind assumes that, below the gradient height, the average wind velocity decreases following a well defined profile. Below the gradient height, the rougher the surface of the ground (upstream from the location where we are interested in the wind speed), the slower the average wind speed Vm[z] will be. Fig. 2 shows typical academic wind profiles, completely defined by the assumed reference wind speed V_{Ref} at 10 m above the ground and the ground roughness.

Profile equations proposed by three major international codes for Open Country or Reference category (called Category C by ASCE, Category B by IEC, and Category 2 by CENELEC) are:

ASCE/ NESC	$Vm[z] / Vm[10] = 1.42(z/275)^{1/9.5}$	Eq. 1
IEC	$Vm[z] / Vm[10] = (z/10)^{0.16}$	Eq. 2
CENELEC	$Vm[z] / Vm[10] = 0.19 \text{ Log } [z/0.05]$	Eq. 3

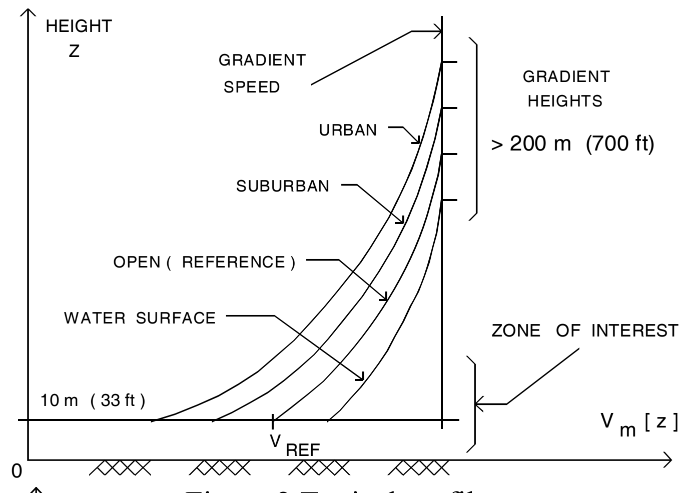

Figure 2 Typical profiles

At 40 m above ground, the above equations give increases in velocity of 1.16, 1.25 and 1.27, respectively (actually, Eq. 1 is applied to the gust even though its source implies an established wind over several minutes). Given that pressures are proportional to the squares of velocities, the pressure increase at 40 m over that at 10 m are 35%, 56% and 61% respectively. Nobody can really say that one of the prescribed increases is better than the other, as in a real storm the values are quite variable. One of the few full scale tests conducted on power lines (Houle et al., 1991) during Winter Storms conditions, show a very high coefficient of variation (from 30 to 42%) for the exponent of the power laws in Eq. 1 and 2. These observations, coupled with the fact that terrain categories are somewhat arbitrary and difficult to assign in real situations, suggest that there is significant uncertainty in the assumed shapes of the profiles. In fact, as described in CIGRE Brochure 350 (CIGRE, 2008), even the general shapes of the profiles shown in Fig. 2 are totally inappropriate for local storms.

While the profiles described by Eqs. 1 to 3 are supposed to be valid for the mean velocity and extend to very high elevations (the gradient height), actual profiles of interest are generally limited to less than 60 m (except for river crossing towers).

2.5.2 The fluctuating value

The fluctuating value $V_f[z,t]$ is assumed to be a stationary random process with invariant statistical properties over the averaging time period T_{av}. This assumes that during at least 10 minutes the storm properties do not change (stationarity). One can immediately see that this assumption is invalid for all local storms. Due to the retarding effect from the roughness of the ground, the turbulence increases from the gradient height down to the ground level, and is greater for rougher terrains (i.e. increasing from terrain Category D to Category A according to the ASCE classification, from Category A to Category D according to IEC classification and from Category 1 to Category 5 according to CENELEC classification). The turbulence has to be characterized by an auto-covariance and a cross-covariance function. The auto-covariance function C[z,lag] is a measure of the correlation between V_f[z,t] at time t and its value V_f[z,t+lag] at a later time t + lag. The cross-covariance function C[v,w,lag] is a measure of the correlation between the velocity v at one point in space (say one point on a transmission structure or along a span) and the velocity w at a second point (on the same structure or the same span, but a given distance away from the first point). The Fourier transforms of the auto-covariance function and the cross-covariance function are the Power Density Spectrum S(z,f) and the Cross-Spectrum S(z,f,distance) of the gust, respectively, where f is frequency.

The most used Power Density Spectra of the fluctuating wind referred to over and over again for the calculation of wind loads in the overhead line community (Cojan, 1973, Castanheta, 1970, Armitt et al., 1975; ASCE Manual 74, 1984, 1991 and 2008; NESC, 2007) are those suggested by Davenport (Davenport, 1961 and later). The Davenport spectra are “empirical” equations based on the average of measurements performed in strong winds, over terrains of different roughness, at different heights. The basic form is:

$$f\,S[f] = a\,x/(1+x^2)^{4/3} \qquad \text{Eq. 4}$$

where x = 1200 f divided by the mean velocity at 10 m and “a” is a constant that includes the mean wind value at 10 m above the ground and a surface drag coefficient. Eq. 4 assumes the spectrum to be independent of height, which is really not the case.

Davenport later proposed another formula that included the height above the ground (Davenport, 1979):

$$f\,S[f] = b\,(f\,z)^{-2/3} \qquad \text{Eq. 5}$$

where b is another constant.

Regarding assumed cross-spectra, there are very complicated equations and concepts: we will spare you the details.

2.5.2.1 The Gust Factor

One important thing to know about the academic winds, is that their wind gusts are not related to measured values such as those that form the basis of the 3-sec gust ASCE map (ASCE, 2006), but are statistical estimates related to their mean. The relationship between the wind gust and the corresponding 10 min average (or other averages) is called the Gust Factor, GF, which is NOT to be confused with the Gust Response Factor, GRF discussed later in this paper. Fig. 3 shows IEC estimates of the ratio of the fastest wind averaged over a time period “t” to the 10 min. average (IEC, 2003). This is obviously extremely approximate and results in straight lines on a log-scale paper between 2 sec. and 10 min., but it is being proposed to determine Gust Factors. ASCE Manual 74 uses a curve developed by Durst (Durst, 1960) to estimate GF's. CENELEC uses the following equation for the Gust Factor at height z to relate the peak gust $V_g[z]$ to the 10-min average for Open Country:

$$GF[z] = 1 + 2.28/ \mathrm{Log}[z/0.05] \qquad \text{Eq. 6}$$

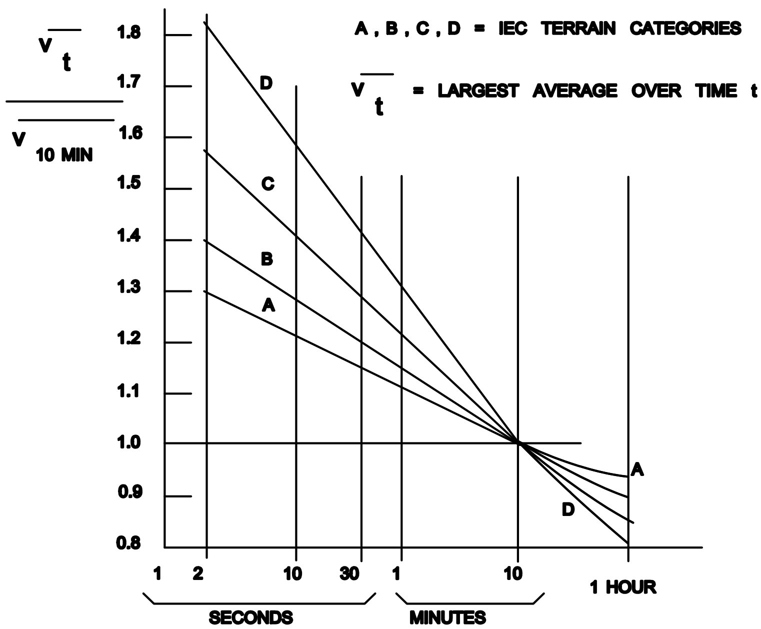

Figure 3 IEC velocity ratios for various averaging times

For Open Country, the Gust Factor related to the 10 minute average is about 1.40 according to IEC (from Fig. 3), 1.43 according to Durst and 1.43 according to CENELEC. The fact that these three numbers are close to each other does not imply that for real storms there is a good relationship between a gust and the 10-min average: it simply means that all three documents are based on similar 1960'smodels. Actually, GF values for real storms vary wildly and one can assume that any proposed equation or graph for GF's can only be approximate. According to IEC, the Open Country GF related to the hourly average is 1.4/0.88 = 1.59

2.5.3 Can we trust the academic wind equations ?

This author developed some teaching material for an advanced graduate course on the dynamics of structures subjected to random loads such as wind and earthquakes and he used a well-developed textbook on the subject (Ghiocel et al, 1975). Based on this experience, it was concluded that some advanced knowledge of calculus, statistics and random processes, and several class hours were needed simply to understand the concepts and limitations behind the academic winds, their power spectra and cross-spectra, and the corresponding structural responses. The complexity of the formulations has intimidated many engineers and prevented them from questioning their validity.
In summary, the academic winds assume the following:

At the somewhat arbitrary gradient height there is a hypothetical wind with no turbulence that remains constant over at least 10 minutes. However, due to the roughness of the ground, the mean wind is slowed-down by friction against the ground and turbulence is generated by that friction. The amount of turbulence depends on the roughness of the ground that is somewhat arbitrarily defined for 3 to 5 categories: smooth water surfaces such as coastal areas (ASCE Category D), slightly rough surfaces such as open country (ASCE Category C), rougher surfaces such as suburban areas (ASCE Category B), etc. Estimated gusts associated with academic winds are very approximate statistical outgrowth of the turbulence and are not a measured quantity.

Unfortunately, because of the impossibility of characterizing the almost infinite ways in which real winds can vary during a storm, their reduction to “academic winds” as described above (1960 and 1970 vintage models) has gained credibility due to: 1) the absence of alternates, 2) the lack of understanding of the limiting assumptions, 3) the elegance of the formulations, and 4) the status acquired by their appearance in print. Our industry was grasping for something to quantify a basically unknowable phenomenon. Therefore, like with other parts of the wind loading equations that are discussed later in this paper, the “academic winds” spread like unchecked viruses in many codes, are often accepted as “facts”, and are unnecessarily complicating our design processes.

3. EFFECT OF TOPOGRAPHY

It is well known that the presence of hills and valleys locally affects the wind velocity. The wind blowing over a hill ridge and perpendicular to that ridge has a higher velocity and different turbulence characteristics than the wind undisturbed by the hill. There are proposed solutions to that problem (Armitt et al., 1975; ASCE Manual 74, 2008), but it is generally not taken into account in transmission line design, probably because of the wide variation of topographic features along a line route and the need for standardization. However this is an important concept for building design where the topography is well known, the structure is custom designed and the need for precision is higher.

4. FROM WIND VELOCITY TO STRUCTURAL RESPONSE

If a transmission structure or a span were just a small rigid body at a distance z above the ground, the maximum wind force on it would simply be:

$$F = q_g[z]\ C_d\ A \qquad \text{Eq. 7}$$

where $q_g[z]$ is the stagnation pressure (also called the dynamic wind pressure) equal to 0.5 x mass density of air x $V_g[z]^2$ (where $V_g[z]$ is the gust velocity at height z), C_d is the drag coefficient of the body, and A is its exposed area (perpendicular to the wind).

4.2.1 Span Factor and Gust Response Factor

Unfortunately our transmission structures and our spans may not be small in size and, as some have suggested, they may have some kind of resonant dynamic response to the wind. We will first deal with the "resonant response" concern and then with the "size effect".

By resonant dynamic response, we are not talking about the pseudo static response of a line component that simply follows slowly increasing or decreasing winds, or the aeolian vibration (perpendicular to the wind velocity) that some spans and structural components experience under laminar wind, or the galloping of spans. We are talking about the possibility of some "along-wind" resonance, i.e. a resonant response in the direction of the wind due to the fact that the wind spectrum could have significant energy close to some natural frequencies of the system. Fortunately for us, along-wind resonance of transmission lines is NOT a concern. The majority of the wind loads come from the spans and when a span is subjected to extreme wind (blown out with non-uniform wind along its length) there is no conceivable mechanism or identifiable pendulum-type natural frequency that could be excited dynamically by the wind. As to some dynamic response of latticed towers, they have high natural frequencies at which the wind does not have energy. This is unlike some tall buildings that have much lower frequencies. Steel and concrete poles, if wires were not attached to them, could have some resonant along-wind response. However, with the wires attached, it is not possible for these structures to vibrate as they would do alone, both because of the restraint from the wires and the damping that they provide. Therefore, any attempt to include a possible resonant response factor in our transmission line design practice should be resisted vigorously as unfounded. This is similar to the attempt made by building designers to force our industry to include an earthquake loading case for transmission lines. Transmission lines, as shown during real earthquakes or theoretically, do not respond significantly to earthquakes as substation or building structures do.

Now let us discuss the "size effect", also referred to as aerodynamic admittance. First consider a 40 m/s gust. In 3 seconds it will have covered a space of 120 m. Therefore, such a gust will envelope an entire transmission structure (except probably a river crossing tower) at once and also a very short span (certainly the span of a distribution line). Therefore, there should not be any size effect concern for most of our transmission structures if the velocity of the gust is the reference design value. However, for spans, it is very likely that when a gust hits a portion of a span, the rest of it is subjected to lesser wind velocities. This is because wind velocities are not fully correlated over longer spans. Therefore, this "size effect" can be taken advantage of by allowing a reduction of the unit wire design loads for long spans. This is usually handled by adding a Span Factor (SF) to Eq. 7 as shown in Eq. 8:

$$F = q_g[z]\ SF\ C_d\ A \qquad \text{Eq. 8}$$

Codes that use Span Factors as shown in Eq. 8 normally neglect the "size effect" on structures and neglect all dynamic effects. Only the lack of correlation of the wind velocities along the span are considered.

Another approach to the handling of the “size effect” and the possible “resonant effect” is the Gust Response Factor (GRF) approach. The GRF approach consists of specifying a force F, which, if applied statically, would cause the system to reach its expected peak response. With the GRF approach, Eq. 8 is replaced by Eq. 9 where the stagnation pressure $q_m[z]$ is now equal to 0.5 x mass density of air x $V_m[z]^2$ (where $V_m[z)]$ is the 10-min average, or longer, wind velocity):

$$F = q_m[z]\ GRF\ C_d\ A \qquad \text{Eq. 9}$$

Codes that use Gust Response Factors traditionally consider both the “size effect” and the “resonant effect”, not only on spans but also on structures. The ASCE Standard 7-05 (ASCE, 2006) for buildings includes the resonant effect in its gust factors, but this standard is certainly not applicable to transmission lines.

There is sometimes some confusion in the naming of the factors: for example, CENELEC (CENELEC, 2001) in Art. 4.2.2.3 calls Gust Response Factor what is really the square of the Gust Factor (Gust Factor = ratio of peak wind to 10-min average) and it calls “Structural Resonance Factor” what is really as Span Factor. We also know of big mistakes that have been made when using 10-min mean wind values as input to equations such as Eq. 8 or gust values as input to equations such as Eq. 9. Depending on the formulation, SF and GRF may be a function of the height above ground “z”.

The derivation of the Gust Response Factor proposed by Davenport (Davenport, 1978 & 1979) that eventually found its way into the ASCE/ NESC equations is very elaborate and based on many assumptions, one of which is quite arbitrary. This arbitrary assumption simply says that the peak response of a span or a structure is equal to its response to the mean wind value (10-min average) plus a certain number (statistical factor) of standard deviations of the response. The statistical factor is suggested to be a number between 3.5 and 4. The standard deviation is calculated as the area under the power spectrum of the response. There are also approximations in the mathematics. For any wind that does not follow exactly the stationarity assumption and the power spectrum of the academic wind (as so many winds do), one has to question the validity of the proposed GRF.

Because Eq. 9 starts with the mean wind, the Davenport approach should end up with GRF’s for small structures and short spans that are larger than 1. However, because the NESC/ASCE procedures are not using an average reference wind but a gust reference wind, $q_g[z]$ replaced qm[z] in Eq. 9 and the GRF’s were decreased by the square of the gust factor.

4.2.2 Wind direction

Whether one uses a design equation such as Eq. 8 or Eq. 9 for determining the wind force on a span, it is almost universally assumed, as a worst case hypothesis, that the wind blows perpendicular to the spans. However, if the wind were to blow at a certain incidence angle from the normal to a span, the force would decrease by a factor that is equal to the square of the cosine of the incidence angle. For example, if the wind is at 45 degrees, the force goes down by a factor of 2. For any incidence larger than 18 degrees, the force will go down by more than 10%, i.e. for winds that are equally likely to come from any direction, there is an 80% chance that the force will be 10% smaller (and sometimes much less) than assumed by a design equation that applies the wind normal to the span. Since most design wind maps are statistics of wind velocities that do not consider wind direction, assuming that these wind will occur perpendicular to the spans as most codes require is quite conservative. For directional winds such as Downslope Winds, some lines will be much more vulnerable than others.

The wind direction effect is one of the major uncertainties when trying to estimate wire loads on a probabilistic basis. Some building and communication tower codes even include a "wind directionality factor" to account for the reduced probability of maximum wind coming from any direction.

4.2.3 Center of pressure

There is the question about what value of "z" should be used in Eqs. 8 and 9. For structures, the NESC suggests using a single value at 2/3 the height of the structure and then using the resulting pressure over the entire height. Other codes expect the designer to break down the structure into sections at different heights and to compute a different pressure at each height based on the center of gravity of the area of the section. For spans, some codes require the use of the center of pressure for the conductor, which is approximated as 1/3 of the sag below the attachment points. One issue is whether to have a different "z" for each wire or to use some kind of average. Some codes suggest using the average attachment height of all wires or that of the highest wire. Calculated loads are certainly sensitive to these assumptions.

4.2.4 Wire tensions

Wire tensions affect the transverse loads on all angle structures and all dead ends. They also affect the vertical loads on all structures with non-horizontal adjacent spans. So, the tension is not really due to the effect of the wind on the two spans adjacent to a structure, but the uncorrelated wind on all the spans in the tension sections (from dead end to dead end) where the structure is located. If this tension section includes many spans, no one will ever know what length to use for the calculation of the SF or the GRF, or even if the equations are valid for such a calculation.

4.2.5 Drag Coefficient

ASCE Manual 74 (1984 and 1991) had quite a bit of information on published values of drag coefficients for cables, structural members and some assemblies.

While these ASCE documents and IEC suggest a drag coefficient of 1 for all wires, international practice regarding this varies widely. Houle et al (1991) and ASCE Manual 74 show that, even for a given Reynolds number, there a wide scatter of conductor drag coefficients.

For poles, drag coefficients close to 1 are generally recommended.

Regarding latticed towers, the NESC simply requires a combined drag coefficient of 3.2 (1.6 for the members of the front face plus 1.6 for the members on the back face) to be applied to the exposed area of the front face and a constant design pressure calculated at 2/3 the height of the tower. However for rectangular cross section towers, the IEC, CENELEC and ASCE all use the same formula for the combination of drag coefficient and exposed area. The formula is:

$$(1+0.2\ \mathrm{Sin}^2[2a])\ (C_t\ A_t \cos^2[a] + C_l\ A_l \sin^2[a]) \qquad \text{Eq. 10}$$

where "a" is the direction of the wind relative to the tower transverse axis, A_t is the area of the tower face exposed to pure transverse wind, A_l is the area of the tower face exposed to pure longitudinal wind and C_t, C_l are drag coefficients based on the solidity ratios of the faces (these coefficients can vary from 4 to slightly less than 2). This is an example of a formula that spread like a virus from one code to the other as there is no real knowledge of what happens to a full size tower in a real storm. The formula is of no help to the designers of towers that have non-rectangular cross sections such as shown in Fig. 4 and is almost impossible to automate in a tower design computer program. The limitation of the formula was determined experimentally (De Oliveira et al, 2006).

The limited use of Eq. 10 prompted the latest ASCE Manual 74 (2008) to include a universal "wind on members" procedure as an alternate to the "wind on face" approach of Eq. 10. This alternate procedure conservatively ignores shielding (which is impossible to know for any configuration different from a perfectly rectangular section) and determines the wind force on each member independently, based on the relative orientation of the wind and the axis of that member. In the comparisons

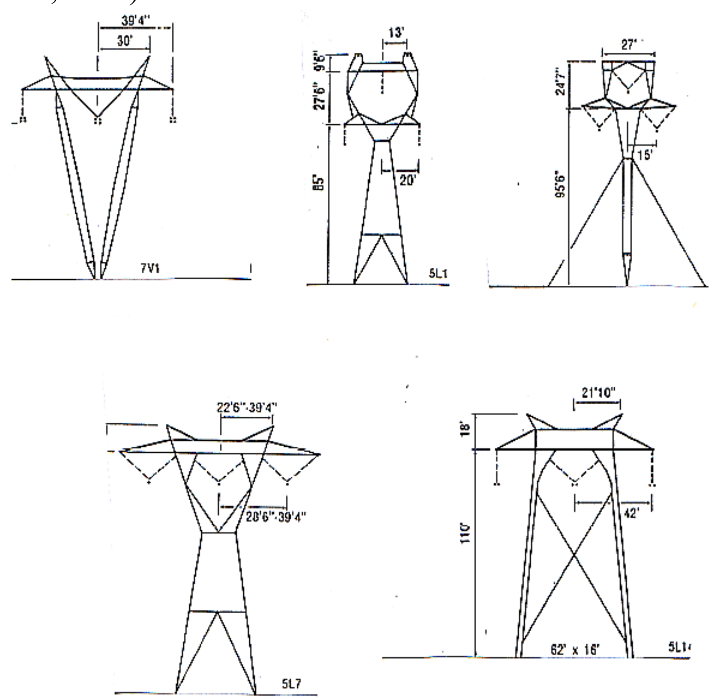

Figure 4 Typical tower configurations

of Section 6.2, we will refer to the ASCE 2008 method that uses the face as "ASCE 2008 F" and that that handles each member separately as "ASCE 2008 M".

In addition to its lack of universality, the formula of Eq. 10 requires that a tower be divided into sections, each section having its own solidity ratio and height above ground. The combination of having to micro-manage the solidity ratio together with the section height is another complexity (fortunately not required by the NESC) which cannot be justified by the uncertainties involved.

Micro-managing the wind load on a tower can be pushed to extremes as demonstrated by the UK-NNA (2004). This specification considers the probability that the maximum conductor loads are not fully synchronized with the maximum load on the tower itself, and therefore the two are not simply additive. Since the portion of the resulting force in a member due to the loads from the wires as opposed to wind loads directly on the structure varies depending on the type of member and its location (for example main leg member vs. diagonal), there is a factor that adjusts wind loads based on the relative sensitivity of the members to wire and tower loads.

4.2.6 Wind on insulators

Some codes require that wind on insulators be calculated, including the effect of gustiness and increase of wind velocity with height. This is probably the epitome of un-necessary complexity as no-one knows the shielding effect of the structures on insulators on one side of a structure and the exposed area/ drag coefficient of an inclined insulator under extreme wind. Given the small contribution of the insulator load to the total structure load, this is certainly something that can be neglected as rightly done in the US.

5. CODES, STANDARDS AND GUIDES SELECTED FOR COMPARISONS

Three design methods that have received international attention are those of the ASCE/ NESC, CENELEC, and IEC. We will add to the list a fourth method, the UK NNA method, which is one of the many variants of CENELEC, but is based on a one hour average academic wind. All are loosely based on academic wind models developed more than 30 years ago by the few individuals cited in Section 1. Very few engineers on the current code committees even know the assumptions and complexity of the original mathematics. However, over the last 30 years, the three design methods have evolved somewhat independently due to the political and adaptation processes that eventually led to their official adoption since 2000.

All four methods start with the selection of a basic 50-year Reference Wind measured at the Reference Height of 10 m. Reference Winds are normally available from national wind maps. However, the type of Reference Winds and their averaging time are totally different. In IEC document, it is a 10-minute average wind. In CENELEC (depending on the NNA or method chosen), it can be a 2-sec gust, a 10-min average or even a 1-hour average as used in the UK NNA. In ASCE/ NESC, it used to be a fastest mile (which is close to a 1-min average), but it is now a 3-sec gust. Since gusts are the velocities of importance in design, collecting and using “gust” data as the starting point (Reference Wind) is certainly the right thing to do, rather than counting on some lose statistical relationships between a gust or a peak structural response and the corresponding 10-min or 1-hour reference values. So we can certainly say that the adoption by the ASCE/ NESC of a 3-sec Reference Wind is a major move in the right direction. One of the common-sense features of the ASCE wind map is that measured gust data for the non-hurricane zones of the US were assembled from a number of stations in state-sized areas to decrease sampling error (Peterka, 1998). Then, based on the insufficient variations over the Eastern 3/4 of the lower 48 States to justify contours, a single zone of 40 m/s (90 mph) was adopted. This laudable simplification is in contrast to some unbelievable micro-zoning maps included in some other national codes. But it is understood that this simplification (harmonizing all winds into larger zones) has its inherent inaccuracies and has been challenged (Simiu, 2003).

One thing is clear though with the ASCE/ NESC maps: they do mix together data from large scale wind storms and local storms, as well as those from tropical cyclones, but they exclude winds from tornadoes. They are wind estimates at a point. However, the 50-year reference wind that would be needed for a better reliability estimation of a line should not include maximum winds at a point but maximum winds over the space of the line: but those data are rarely available.

All four methods use wind profiles (effect of height), and some combination of gust factors with span factors, or gust response factors, all of which are essentially only valid for the “academic winds”. In fact, the ASCE/ NESC procedure, which is using the best basis for the Reference Wind (3-sec), is still borrowing internally the mathematics of an academic wind. Now, let's think for a minute about what it is doing. It starts with a 3-sec gust which is related to measured data, but the academic wind theory says that we need an established wind over at least 10-minute for the theory behind the profiles and the GRF’s to be valid. So the ASCE/ NESC procedure takes a good design value (the 3-second gust) and it reduces that value internally by a somewhat arbitrary number to get the corresponding 10-min wind so that it can salvage the academic wind basis of the profile and the vintage GRF. It makes very little sense to say that a measured gust of say 40 m/s is really the peak wind of an assumed 10-min wind of 28 m/s or an assumed 1-hour wind of 25 m/s.

This writer understands the huge amount of sincere efforts behind the development of the code procedures mentioned in this section. The negative comments throughout this paper are only meant to emphasize the underlying uncertainties, the unnecessary complexity and the lack of consensus.

For the design of buildings or other isolated structures as described in the ASCE Standard 7-05 (ASCE, 2006) one can possibly justify some refinements in the description of the nearby terrain roughness and topography, as well as the dynamic and shape characteristics of these structures. But transmission lines are totally different systems from buildings that are normally custom designed for a site. Buildings have volumes for which there are very important issues of local “force” coefficients that affect local pressures around the building and possible “dynamic” behavior. Transmission lines and their supports, on the other hand, have "line like" components with insignificant along-wind resonant dynamics and where local "force" coefficients are irrelevant, but instead "drag" coefficients are needed. Transmission lines also benefit from some level of standardization and they can cover a wide range of terrain characteristics: their most important loads are span loads for which ground roughness, span length and wind incidence introduce considerable uncertainties. Therefore, design rules for transmission lines should be unique and not be imposed by regulating bodies or academic-types that deal with building structures.

6. SOME COMPARISONS

Very simple examples are presented in this section to demonstrate the combined effect of height and span length on wire loads and the combined effect of height and drag coefficients on structure loads with the goal of appreciating the sensitivity of these loads to some code assumptions and the corresponding lack of consensus.

6.1 Wire loads

The wire load examples all consider a Drake conductor sagged at 20% of ultimate after creep for various combinations of attachment heights, span lengths and code methods. For the five attachment heights considered, the span lengths were maximized so that they would have an 8 m clearance above ground at 115 deg C after creep as shown in Fig. 5 (a vertical to horizontal scale ratio of 10/1 was selected for the display of the PLS-CADD models in Fig. 5 for clarity of presentation).

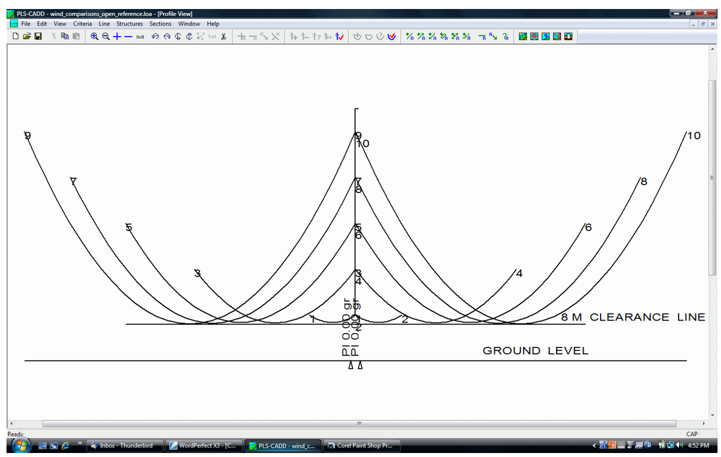

Figure 5 Span examples

As a reference for our wire load examples, a 1 m long rod having the same diameter as the Drake conductor, located 10 m above the ground and subjected to a gust of 40 m/s would be subjected to a force of .613 x 40 x 40 x 0.0281 = 27.6 N/m. In the rest of this section, we will report the load per unit length of that same Drake conductor in the models of Fig. 5 when subjected to combinations of heights, span lengths and code assumptions as the rounded ratio (Wire Load Ratio) of the calculated code load divided by the Reference Load of 27.6 N/m. The ratio is rounded to two digits, as any other precision for our demonstration purpose would be meaningless.

For comparison with the old traditional US design methods, the ratios are also calculated for NESC Rule 250B (including the 2.5 load factor for wind and the ice thickness, if any). For example, the ratio is 0.93 (25.6 N/m divided by 27.6) for the

Heavy Loading District, 0.71 for the Medium District and 1.10 for the Light District.

For the various codes considered, the Reference Winds at 10 m above the ground were taken as follows:

NESC 2007	40 m/s gust (same as ASCE Terrain Category C)
ASCE 2008	40 m/s gust for ASCE Terrain Category C
IEC 2003	40/1.40 = 28.6 m/s 10-min av. for IEC Terrain Category B (Remember, 1.40 from Section 2.5.2.1 is somewhat arbitrary)
CENELEC 2001	40 m/s gust for CENELEC Terrain Category II
UK NNA 2004	40/1.59 = 25 m/s 1-hr av. for UK NNA Terrain Category 3 (Remember, 1.59 from Section 2.5.2.1 is somewhat arbitrary)

For the Reference Winds described above, Table 1 shows the Wire Load Ratios for the Open Terrain categories of various codes. For completeness, these categories are described below (the wording of the documents is purposely included to draw attention to the sensitivity of the answers to some imprecise definitions):

NESC 2007	No terrain category considered
ASCE 2008	Cat. C: Flat open country, farms, and grasslands.
IEC 2003	Cat. B: Open country with very few obstacles, for example airports or cultivated fields with few trees or buildings
CENELEC 2001	Cat. II: Farmland with boundary hedges, occasional small farm structures, houses or trees
UK NNA 2004:	Cat. III: Basic open terrain, typical UK farmland, nearly flat or gently undulating countryside, fields with crops, fences and low hedges or isolated trees.

In order to show how loads change when going from one loosely defined terrain category to the next rougher one, Table 2 shows Wire Load Ratios similar to those in Table 1 for the categories defined below:

NESC 2007	No terrain category considered
ASCE 2008	Cat. B: Urban or suburban areas, well wooded areas, or terrain with numerous closely spaced obstructions having the size of a single-family dwelling or larger.
IEC 2003	Cat. C: Terrain with numerous small obstacles or low height (hedges, trees and buildings)
CENELEC 2001	Cat. III: Suburban or industrial areas and permanent forests
UK NNA 2004:	Cat. IV: Farmland with frequent high hedges, occasional small farm structures, houses or trees.

Table 1 Wire Load Ratios for Drake conductor in Open Country

	Combinations of attachment heights and span lengths (m)				
	10-100	20-350	30-500	40-620	50-720
NESC 2007	0.80	0.78	0.80	0.83	0.85
ASCE 2008	0.80	0.78	0.80	0.83	0.85
IEC 2003	0.94	1.03	1.05	1.06	1.07
CENELEC 2001	0.92	0.99	1.05	1.10	1.13
UK NNA 2004	0.85	0.81	0.84	0.86	0.88

Old NESC 250B Heavy:	0.93
Old NESC 250B Medium:	0.71
Old NESC 250B Light:	1.10

Table 2 Wire Load Ratios for Drake conductor for next rougher terrain category above Open Country

	Combinations of attachment heights and span lengths (m)				
	10-100	20-350	30-500	40-620	50-720
NESC 2007			Not used		
ASCE 2008	0.64	0.62	0.65	0.68	0.70
IEC 2003	0.76	0.85	0.88	0.89	0.90
CENELEC 2001	0.85	0.72	0.77	0.82	0.85
UK NNA 2004	0.69	0.75	0.78	0.81	0.83

One could be tempted to make comparisons between corresponding numbers in different rows within each of the two tables above, and say for example that NESC (2007) is not conservative because it sometimes gives conductor loads that are more than 20% lower than those from the IEC. However, such conclusions are meaningless as one calculation starts with a gust wind and the other with a 10-min average, and there is no fixed relationship between the two. But some useful conclusions can be drawn from the above tables as discussed below.

Looking at all the numbers across one row of Table 1 (i.e. over a very wide range of spans), one will notice that the maximum variation is only 6.6% for the NESC and ASCE, 13.5 % for IEC, 22.8 % for CENELEC, and 3.2 % for UK NNA. These percent numbers would be even smaller if we had accounted for the lower center of pressure for long spans, which some codes require to be 1/3 of the sag below the attachment points. This “relative insensitivity” of the wire loads to the combined height and span length parameters was also mentioned in a proposed change of the NESC (Kluge, 2005) and can be used as an argument for one of the proposed simplification in Section 7.3.4. Because long spans tend to be higher, the increase of the wind velocity with height is tempered by the lack of correlation of the gusts along the span (the size effect), thus the "relative insensitivity" (a term we will use later in this paper) of the wire loads.

But looking at numbers in identical locations in Table 1 and in Table 2 will show much larger differences. For example, the fact that the number of trees or buildings may vary from "a few" to "numerous" will lower the IEC span loads by over 17%, which is more than the 14% variation due to the full range of heights and span lengths considered. This is a very significant contribution to the uncertainty of the calculated numbers when one considers that a line will most likely traverse terrains where the roughness and topography of the upwind surface varies.

6.2 Structure loads

To illustrate some of the complexities, variations and uncertainties of some code tower wind loads, we purposely selected a very tall tower (86 m) as shown in Fig. 6. That tower has a fairly simple geometry, so that Eq. 10 can be used on the 7 main sections of the tower whose important properties are summarized in Table 3. The arms at each of the three levels were modeled as three separate sections with transverse wind areas of 4.5, 4.5 and 5.8 m^2, respectively.

Table 3 Tower Sections

Sect. #	Average Height (m)	Face Area (m^2)	Solidity Ratio (%)
1	81	6.2	37.1
2	70	9.4	39.6
3	57	8.2	23.7
4	46	6.1	14.4
5	35	9.1	9.4
6	21	8.9	8.3
7	8	10.4	6.1

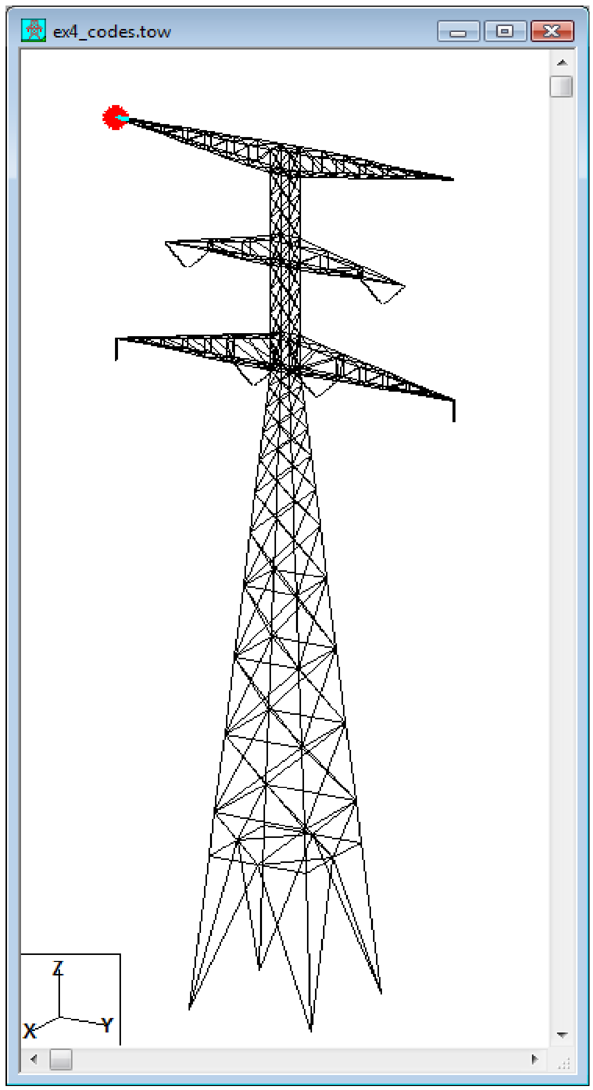

Figure 6 Tower example

We used the TOWER program to analyze the tower without conductor loads but with a transverse wind blowing on the tower, assuming in all cases a 40 m/s Reference Gust Wind at 10 m above the ground giving a Reference Pressure of .613 x 40 x 40 = 981 Pa. Assuming that the Reference Wind is applied over the entire height of the tower, i.e. that there is no escalation with height, two Reference cases were considered. For the “Reference F” case, the reference pressure of 981 Pa was applied to the entire face of the tower assuming a drag coefficient of 3.2 for that face (similar to that required by the NESC): this is a “wind-on-face” approach. For the “Reference M” case, the reference wind of 40 m/s was applied on each individual member of the tower, assuming a drag coefficient of 1.6 for each member and letting the computer determine the wind load on each member based on the relative orientation of the wind velocity and the member axis: this is a “wind on members” approach.

For each of the two Reference Cases and for some codes, the overturning moments at the tower base were calculated as shown in Table 4. The overturning moment is the simplest and most important measure of the wind load and its demand on the tower. All the calculations in Table 4 assumed an Open Country (as was done for the wires in Table 1). Since ASCE 2008 allows two alternate methods (a “wind-on-face” approach and a “wind-on-members” approach), the two methods are included in the table and are designated as “ASCE 2008 F” and “ASCE 2008 M”, respectively. All other code methods use the “wind on face” approach. Where appropriate, Table 4 also includes the pressures and drag coefficients for 3 of the 7 sections.

Table 4 Tower Base Moment, Pressures and Drag Coefficients for selected sections (Open Country)

	Base Moment (kN-m)	Sect. 1 Press. (Pa)	Sect. 1 CD	Sect. 3 Press. (Pa)	Sect. 3 CD	Sect. 6 Press. (Pa)	Sect. 6 CD
Reference F	10,300	981	3.2	981	3.2	981	3.2
NESC 2007	11,700	1110	3.2	1110	3.2	1110	3.2
ASCE 2008 F	9,200	1190	2.2	1110	2.9	901	3.7
IEC 2003	10,500	1310	2.3	1300	2.8	1090	3.5
CENELEC 2001	13,000	1700	2.3	1580	2.8	1260	3.5
UK NNA 2004	11,300	1560	2.3	1390	2.8	1000	3.5
ASCE 2008 M	11,800	Velocity at height of member considered CD = 1.6 for all members ASCE GRF = 0.78 for 86 m tower included					
Reference M	10,500	40 m/s assumed at all member locations CD = 1.6 for all members No GRF included					

Following are some comments regarding the numbers in Table 4.

NESC 2007 includes a 0.78 tower GRF (a surprisingly small number implying a significant lack of correlation of wind velocities along the height of the 86 m tower), it assumes a constant design pressure as that calculated at 2/3 of the total height of the structure and it uses a constant drag coefficient.

ASCE 2008 F includes a 0.78 tower GRF, it increases the pressure with height and it varies the drag coefficient based on the solidity ratio of the face of the section.

The IEC, CENELEC and UK NNA formulae do not specifically isolate a GRF for the tower (even though the TOWER CENELEC calculations include the recommended "structural resonance factor" of 1.05), but they require varying pressures with height and varying drag coefficients.

The large differences in base moments when comparing the IEC, CENELEC and UK NNA (there are much greater variations with other European codes that we have not summarized in this example) were surprising given the supposed cross-pollination between these specifications. This is just one example of the lack of consensus which we have observed all over Europe.

Comparing the base moment for "Reference M" (which ignores the increase of wind velocity with height and the complexity of the "wind on face" approach and associated solidity ratio issues) to the moments from the other codes, suggests that the conservativeness of the "wind on members" approach somehow makes up for ignoring the other factors.

Although not shown here, there would be a substantial drop in the tower loads if the terrain category was changed to the next rougher category.

As with the wire loads, there is some "relative insensitivity" of the final tower load (measured by the base moment) to the tower height and other parameters. This is because the increase of velocity with height may be tempered by a built-in code "size effect" and section drag coefficients that are substantially smaller in the high portion of a tower than near its base. Another contributor to the insensitivity is the fact that short towers are often used on top of a hill where the wind velocity may be higher as opposed to taller towers used at lower elevations. Given that: 1) some "relative insensitivity" is observed for some code procedures, 2) tower wind loads are much smaller than the sum of the wire loads, 3) tower wind loads are not fully synchronized with the wire loads (lack of spatial correlation), and 4) large uncertainties related to the use of academic winds and terrain categories, one should wonder if the minutia of dividing a tower into sections is necessary.

7. NEED FOR SIMPLIFICATION

It has been suggested that, given that most lines are now designed by computers where complex code formulas are automated (PLS-CADD, TOWER, Etc.), complexity is not an issue. This writer totally disagrees with this argument for the two reasons discussed below.

7.1 Reason 1 - Honesty

Given that damaging winds can come from a wide variety of storms and given all the uncertainties discussed previously, it is basically dishonest to pretend that our wind designs will be better (better balance between costs and reliability) if one fine-tunes the contribution of each of the many factors affecting the problem with some questionable equations. The multitude of the current factors that have to be accounted for increases the chance of errors and provides additional litigation opportunities to ignorant parties that will focus on minute irrelevant details rather than understanding what is important. As engineers, we should always favor common sense.

7.2 Reason 2 - Simplify the life of the designer.

One extremely useful concept often used in line design considers that a family of supports, for given supported wires and code criteria, has some allowable wind and weight spans. This concept is immediately invalidated by making wind loads dependent on structure height, conductor heights and span lengths. Another useful concept is that of designing a standard family of supports with the same top geometry but with different heights. For example, the upper portion of a tower and its shortest body is common to the entire family, with body and leg extensions taking care of the need for varying heights. Other issues that can plague the engineer when the wind loads vary with height and span length are the corresponding calculations of the wire tensions. Therefore, there are very good practical arguments for eliminating the dependence of wind loads on height and span length for most common design situations.

7.3 Suggested simplifications

7.3.1 Utilize the 3-sec gust as the Reference Wind

This simplification does not apply to the US where NESC/ ASCE already use gust as the reference wind speed. CENELEC also allows as an option the use of the gust. Since gust winds are those that destroy lines, starting the design process with the gust wind (Reference Wind) eliminates the large inherent uncertainty in some codes of going from mean wind velocity to gust through the Gust Factor or going from mean structure response to peak value through the Gust Response Factor.

The pressure caused by the Reference Wind is the Reference Pressure q_{Ref}. Some codes specify that pressure as the starting point of their wind calculations instead of the corresponding gust velocity.

7.3.2 Eliminate terrain categories

Since terrain categories have such an uncertain effect on wind gusts and are not amenable to clear definitions along a line, they should be eliminated. The reference Open Country category is the only one to keep. This is currently done by the NESC. A coastal or lake increase factor of about 20% might be appropriate for such exposures (current ASCE Category D or IEC Category A).

7.3.3 Abandon the concept of Gust Response Factor

As its name indicates, a Gust Response Factor is the ratio of a peak structure response divided by the average response due to the mean wind. As mentioned in Section 4.2.1, the GRF normally accounts for a possible resonant "dynamic effect" and a "size effect" (lack of correlation of wind gusts at distant points). But the "dynamic effect" is not a factor in transmission lines. The "size effect" is significant for wires, but not for their supporting structures. Therefore, the only contribution from the GRF should be a reduction of wire loads for longer spans, which is exactly what the concept of a Span Factor does.

Because: 1) there is no average structure response to apply a GRF to if one starts with a reference wind which is a gust, 2) there is no identifiable "resonant" response in transmission lines, 3) there is no significant "size effect" on transmission structures, and 4) the only components that can benefit from the "size effect" are the wires, the concept of GRF is inappropriate and should be replaced by a simpler Span Factor where there is a need to reduce unit loads on very long spans. This turns out to be the alternate "empirical approach" of CENELEC.

7.3.4 Eliminate height and span length as variables for the majority of designs

For all structures with maximum height below a cutoff value (to be determined but certainly above 50 m) use the Reference Pressure q_{Ref} over the entire height of the structure and use a reduced pressure (suggested to be around 0.90 q_{Ref}), for all wires attached to the structures. The reduction factor accounts for the low probability of having winds perfectly perpendicular to the spans and having the span loads perfectly synchronized with the structure loads. Using a constant design value accounts for some of the "relative insensitivities" discussed in Sections 6.1 and 6.2.

For unusual situations (river crossing or very long spans), an increase of wind velocity with height and a Span Factor may be considered.

An obvious and even more justified extension of this recommendation would be to simply use the Reference Pressure q_{Ref} on all structures and wires within a substation (ASCE Manual 113, 2008).

7.3.5 Use the same wire wind load for the determination of the lateral wire load transmitted to the supporting structure and for the determination of the wire tensions

This was discussed in Section 4.2.4, and if followed, this simplification will generally result in conservative values of mechanical tensions. However, since tension affects the loads on angle and dead-end structures and since these structures are normally designed to a higher reliability level than regular tangent structures, conservative tensions are desirable.

7.3.6 Offer an alternative to the solidity ratio-based drag coefficients for towers

As discussed in Section 4.2.5, an alternate "wind on members" option is necessary to handle towers that are not appropriate for a "wind on face" approach. This is already included in the latest ASCE Manual 74 (ASCE, 2008).

One could conclude after reading this section that we are going backward. But, after many years of hoping for a better handle on wind loads through research and detailed procedures, we are forced to admit that: 1) we are confronted by an immense problem that is not amenable to precise quantification, and 2) some of the simpler ways of the past, calibrated by new knowledge, are more appropriate.

8. SUMMARY

The complexity of the current generation of procedures for the design of wind loads on transmission lines (and substation structures) is not justified by the uncertainties inherent in each parameter or equation that make up the design process: wind storm type, reference wind, terrain roughness, profiles, gustiness and underlying assumptions regarding time and spatial correlations, gust response factor, wind direction, drag coefficients, etc. Scientifically, one cannot prove that a line designed according to NESC, ASCE, IEC, CENELEC, or any of its NNA's, has a better balance of reliability vs. economy than one designed according to some simpler procedures such as those suggested. Therefore, rather than pursuing the illusion that better designs can be achieved through the precision of complex formulas that depend on a multitude of unknowable parameters, let's favor common sense, engineering honesty and design simplicity over pseudo science. It is hoped that after three decades of complicating our design equations we will come to our senses and work on simplifying them.

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