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SEISMIC DESIGN OF UTILITY-SCALE WIND FARM TOWERS IN THE AMERICAS: A COMPARISON OF US, MEXICAN, AND CHILEAN CODE REQUIREMENTS

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Abstract

With its beginnings in Europe, the commercial utility-scale wind industry has been evolving in North America for approximately thirty years, yet only recently have there been attempts to reconcile building code requirements with industry practices for the seismic design of the wind turbine support structures, the most predominant of which is the modern fabricated tubular tower. One such attempt was a joint effort of the American Society of Civil Engineers (ASCE) and the American Wind Energy Association (AWEA) that resulted in the publication in 2011 of a tower design guideline titled "Recommended Practice for Compliance of Large Land-based Wind Turbine Support Structures," also known as "ASCE/AWEA RP2011." Although perhaps just a first step, RP2011 represented a milestone in the level of design maturity for wind towers and in the level of coordination with the US building code, i.e., the International Building Code (IBC) and ASCE 7.

Now in recent years, utility-scale wind project development has expanded and continues to gain momentum in Central and South America. Based on experiences on these projects, there is a similar situation to the earlier days in the US wind industry where the local building codes are not specialized to address the idiosyncrasies of wind tower design either in general or specifically in seismic design.

Considering the modern wind tower, this paper will compare and contrast US seismic design practices (based on ASCE/AWEA RP2011, the IBC, and ASCE 7) with those of Mexico (the CFE Manual de Diseño de Obras Civiles, Diseño por Sismo, MDOC-DS) and those of Chile (Norma Chilena NCh2369 and the Endesa ETG 1.015 Specification).

The comparison intends to accomplish the following: introduce wind industry design practices; provide insight into the idiosyncratic issues of wind tower seismic design; show differences in how the modal response spectrum analysis (MRSA) procedure is applied; provide an overview of the seismic design challenges considered and addressed by ASCE/AWEA RP2011; and suggest how recommended practices and lessons learned from experiences in the US wind industry might be beneficially considered and coordinated with the Mexican and Chilean seismic design practices for wind towers.

Keywords: Wind Turbine Tower; Chile Wind Tower Design; Wind Tower Seismic Design

1. Introduction

The utility-scale wind energy industry, from its origins in Europe to the ongoing developments in North America and the current expansion into South America, has continued to evolve over the last three decades. This paper discusses the current state of practice for the seismic design of the wind turbine generator system (WTGS) support structure, i.e., the wind farm tower, by comparing the requirements of selected US, Mexican, and Chilean design standards.

1.1. Rapid evolution of WTGS

In the US, WTGS have increased in scale from 65 kW turbines with hub heights (i.e., height to center of rotor) of approximately 25 m to modern turbines ranging from 1.5 MW to over 3 MW with hub heights routinely from 80 m to 100 m. This rapid evolution is shown in Fig. 1.

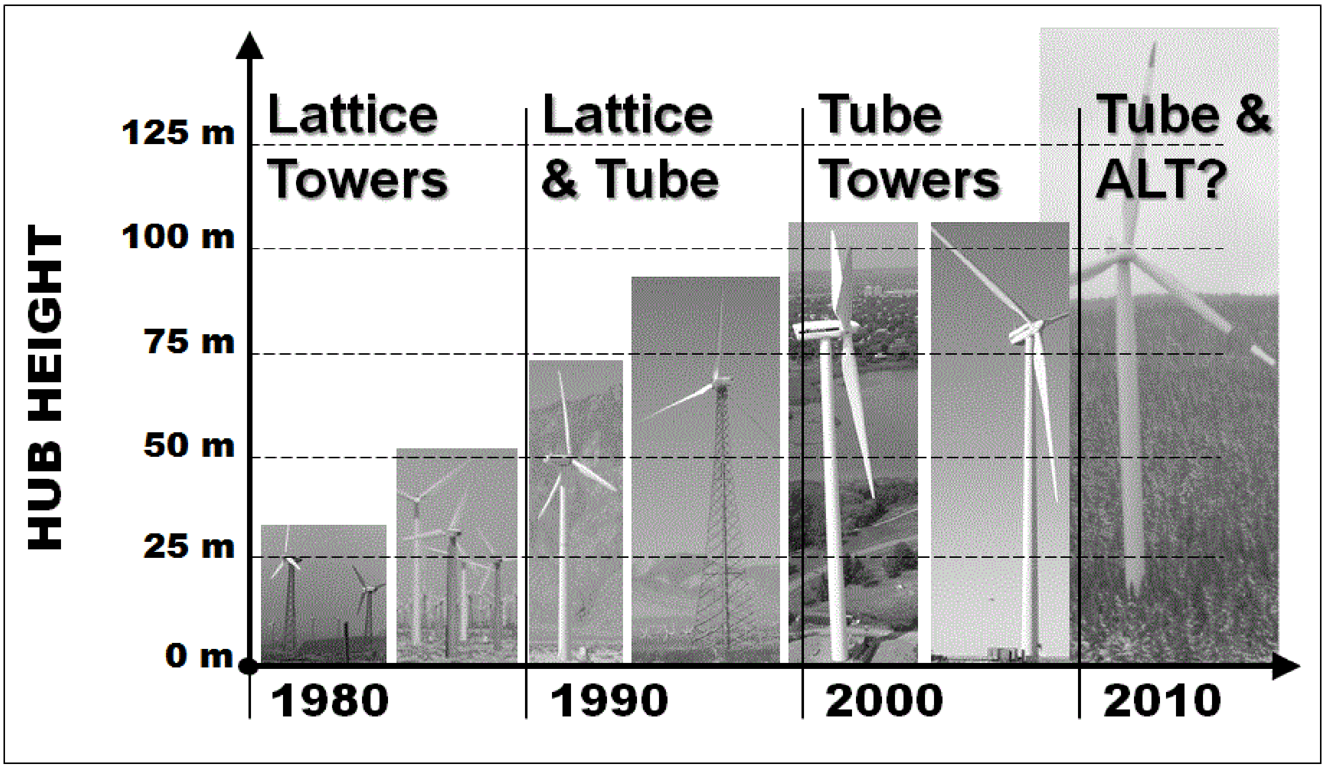

Fig. 1 – The evolution of WTGS showing tower type and height. From Reference [1].

As can be seen in Fig. 1, the WTGS support structure types range from steel lattice (truss) towers to circular cross section tubular towers. Currently, there is interest in alternative tower systems such as concrete tubular towers, hybrid concrete and steel towers, and other proprietary tall tower systems as indicated in Reference [1]. However, in the last decade and a half, the steel tubular tower has been the most widely used WTGS support structure, with the 80 m hub height tower being the current workhorse of the wind industry. For this reason, this paper addresses the seismic design of only this predominant tower type.

In US building code terminology, the WTGS tower is a "nonbuilding structure that is not similar to a building." The wind farm tower structure type is not explicitly captured in the building code seismic coefficient tables (as will be discussed later), but it would be similar in construction to an unlined welded steel stack (chimney) consisting of a large-diameter steel fabricated tube of circular cross section.

1.2. Idiosyncratic design practice

Wind tower design has many aspects that are idiosyncratic to the wind industry. As described in Reference [2], building codes do not specifically address wind farm towers, so the industry relies mainly on European commercial certification agency standards. While building code extreme wind or seismic loads govern the design of most conventional industrial structures, the wind tower's primary shell may sometimes be governed by less common and unfamiliar loads: fatigue loading; abnormal wind loads associated with turbine electrical faults; combinations of annual gusts and turbine faults; earthquake in combination with concurrent turbine aerodynamic operational loads; earthquake in combination with concurrent emergency stop loads; and non-strength-related frequency separation criteria to avoid resonance.

While these are not all seismic conditions, they are mentioned to illustrate the idiosyncratic nature of wind farm tower design, and also to give a preview of the seismic load cases that consider earthquake with concurrent turbine operational or fault loading.

2. Seismic design

What follows are discussions of key seismic design issues along with comparisons among building code seismic standards in the US, Mexico, and Chile: the International Building Code (IBC) and its reference load standard ASCE 7—References [3] and [4], respectively; the CFE MDOC-DS—Reference [5]; and NCh2369 and Endesa ETG 1.015 standards—Reference [6] and [7], respectively.

It is universal that the various building codes do not specifically address WTGS support structures. Moreover, to the authors' collective knowledge, there are no dedicated WTGS support structure reference standards in the subject nations. However, in the US in 2011, a joint effort of the American Society of Civil Engineers (ASCE) and the American Wind Energy Association (AWEA) produced a guideline titled *Recommended Practice for Compliance of Large Land-based Wind Turbine Support Structures*, Reference [8], known as "ASCE/AWEA RP2011." One of the purposes of this guideline was to reconcile and bridge the idiosyncratic wind industry design practices with the requirements for US building code compliance. Among several design issues, code-compliant earthquake design of towers was addressed.

2.1. Permitted analysis procedure

In the US, the ASCE 7 standard conditionally allows the following analysis procedures: equivalent lateral force (ELF) procedure, modal response spectrum analysis (MRSA), and linear and nonlinear response history analysis (RHA) (a.k.a. "time history analysis"). These permitted procedures appear to be generally universal as they also appear in the Mexican CFE MDOC-DS and the Chilean NCh2369 and Endesa ETG 1.015 specifications.

As a practical matter, this paper limits the discussion to the MRSA, for there are good reasons to do so. It should first be observed that in regions of low seismicity where wind governs design, the seismic analysis procedure selected is often irrelevant since wind design forces far exceed the small seismic design force. In regions of moderate to high seismicity where seismic design forces may govern design, the relative conservatism among the seismic procedures becomes important. Since WTGS towers can often number in the tens to hundreds in a single wind farm project, conservatism in design forces and the resulting increase in steel tower weight is multiplied for each tower and foundation made thicker, heavier, deeper, etc. The economic and schedule impacts associated with towers custom designed for additional conservatism can jeopardize the viability of a given project.

In such cases, the ELF procedure can result in seismic design forces significantly higher than for MRSA. In some standards, there are triggers that prevent the use of equivalent static force procedures, making dynamic analysis procedures mandatory. Due to the higher design forces, the ELF procedure, although conservative, is not recommended. In fact, experience has shown that the resulting forces from the ELF procedure may exceed the capacity of otherwise practical tower designs.

The RHA procedure is seldom utilized in the wind industry for several reasons: it is complex and requires geotechnical expertise for ground motion selection; it requires specialized structural expertise; it requires the software capability; in the end the results may still have to be approximately manually combined with the concurrent turbine operational loads, diluting the accuracy of the combined results; it requires the turbine manufacturer to share proprietary secret information with an outside consultant, which is a sensitive issue in the secretive culture of the wind industry; and the US code requires that nonlinear RHA be peer reviewed at the permit applicant's expense, introducing nontrivial engineering expense and yet another outside consultant (peer reviewer) with whom to share proprietary secret information.

In the end, the MRSA procedure represents the best compromise for accuracy as the entry-level linear dynamic analysis method that avoids the conservatism of the ELF procedure and the complexity and mandatory peer review of the nonlinear RHA method. The following items compare features of the various implementations of the MRSA procedure.

2.1.1. Shape of the design response spectrum

Much can be discussed about the relative merits of the shapes of the design response spectra in each of the subject standards. However, for brevity, reader familiarity with general response spectra is assumed, and the discussion will focus on aspects more relevant to tubular towers.

- In the US, ASCE 7 specifies a response spectrum with the following shape, where domain is structure period T and range is spectral acceleration: first, the short period domain, i.e., the constant pseudo-acceleration region, consisting of a short period ramp up to the flat acceleration plateau; next, a long period domain, i.e., the constant pseudo-velocity region, descending as $(1/T)$; and finally, a very long period domain, i.e., the constant pseudo-displacement region, descending as $(1/T^2)$. The base shear obtained from the response spectrum is subject to minimum base shear values, which ultimately often govern the tower design, as discussed later.
- In Mexico, MDOC-DS specifies a response spectrum generally similar in shape to that of ASCE 7. In the long period domain, the spectral acceleration descends as $(1/T^r)$, where $0.5 \leq r \leq 1.0$. In the very long period domain, the spectral acceleration descends as a formula with $(1/T^2)$ and $(1/T^4)$ terms. The formulas allow a condition where the very long period region formula can replace that of the long period region. There is a minimum base shear discussed later.
- In Chile, NCh2369 specifies a response spectrum with the following shape: a short period domain consisting of a flat plateau (with no short period ramp); and then a long period domain, descending as $(1/T^n)$, where n equals 1.00, 1.33, 1.80, or 1.80 for soil type I, II, III, or IV, respectively. There is a minimum base shear discussed later.
- Also in Chile, ETG 1.015 specifies a response spectrum generally similar in shape to that of ASCE 7, except without the very long period descending branch. In the long period domain, the spectral acceleration descends as $(1/T^{2/3})$. There is no base shear minimum. A peculiar feature is a stated limit of T=2.5 seconds maximum. Beyond this limit, engineering judgment would have to apply as to the response spectrum shape.

Since the 80 m to 100 m tubular towers have fundamental periods of approximately 2.5 to 4 seconds, they typically are in the long period domain or possibly in the very long period domain. Where the descent is of the usual form $(1/T^x)$, it is important to observe that smaller values of x result in higher spectral acceleration than larger values of x. Therefore ETG 1.015 with a 2/3 exponent is more conservative than the other standards in this respect. Those with exponents of 1 to 2 would descend more rapidly but likely trigger the minimum base shear requirements.

The return period of the ground motion represented by the response spectra has not been discussed in the interest of brevity. However, it should be noted that core standards of the wind industry such as IEC 61400-1, Reference [9], and the Germanischer Lloyd (GL) Rules, Reference [10], require a 475-year return period. Insofar as this is a commercial certification requirement, this does not necessarily override the regulatory authority of the local building code. This is not a problem if the local requirement meets or exceeds the 475 year value. If not however, then the governing project seismic design requirements must be negotiated.

2.1.2. Number of modes

- ASCE 7 requires a sufficient number of modes for a minimum of 90% mass participation.
- MDOC-DS requires a minimum of 90% mass participation and consideration of all modes with a period of 0.4 seconds or greater.
- NCh2369 requires a sufficient number of modes for a minimum of 90% mass participation.
- ETG 1.015 specification requires consideration of all modes under 30 Hz.

2.1.3. Combining the response parameters

- ASCE 7 specifies the SRSS or CQC method.
- MDOC-DS specifies the SRSS or CQC method, but for steel chimneys and silos it also specifies a similar method to that of ETG 1.015 and Eq. (1) as shown below.
- NCh2369 specifies the CQC method.
- ETG 1.015 specifies an addition of half the sum of the absolute values plus half the sum of the SRSS values as shown in Eq. (1) below, taken from ETG 1.015 Eq. 8.

$$S = (½)\, \Sigma|S_j| + (½)\sqrt{(\Sigma S_j^2)} \quad (1)$$

It is important to consider that for wind towers, the results of this method can be approximately 25% to 40% larger than the result of SRSS combination alone. This increase in seismic design force is not trivial.

2.1.4. Scaling the design values of combined response

- ASCE 7 requires that the combined response for modal base shear shall be scaled such that it is not less than 85% of the static procedure base shear, i.e., the ELF procedure base shear. Further, the ELF base shear itself is subject to minimum values. It is important to note that in the US, long-period structures such as wind towers do not necessarily benefit from the steadily decreasing accelerations along the descending branch of the design response spectrum. In fact, in areas of high seismicity defined by mapped values of $S_1 \geq 0.6$ g, ASCE 7 Eq. 15.4-2 shown below as Eq. (2) almost always governs as the minimum base shear coefficient, C_s:

$$C_s = 0.8S_1/(R/I_e) \text{ where } R \text{ and } I_e \text{ are discussed in a later section} \quad (2)$$

  This minimum base shear coefficient represents a long-period acceleration plateau occurring in some near-source ground motions. According to Kircher, Reference [11], these spectra consistently deviate from the $1/T$ shape (where T is period) in the constant pseudo-velocity region. There appears to be no parallel to this equation in any of the other subject standards, yet this governs tower seismic design in the US.

- MDOC-DS, similar to ASCE 7’s 85% minimum scaling value, specifies a minimum value of 75% for steel chimneys and silos. A minimum base shear is specified as an acceleration parameter a_0 multiplied by the total seismic weight.
- NCh2369 specifies a scaling requirement to a minimum base shear that is defined as a coefficient of 0.25 times an importance factor I times a reference seismic zone acceleration A_o, which takes on values of 0.20 g, 0.30 g, and 0.40 g in seismic zones 1, 2, and 3, respectively. Values of importance factor I are described in the next section.

- ETG 1.015 has no scaling requirements or base shear minimums.

2.2. Structure occupancy or importance

The concept of structure use or occupancy for categorizing importance as a design parameter that affects the value of the seismic design force is common to all of the subject US, Mexican, and Chilean standards.

- ASCE 7 defines a structure's "Risk Category" to determine its "Importance Factor," I_e. It is a direct multiplier on the seismic design force. It ranges from a value 1.0 for structures of low to normal importance, 1.25 for more important structures with a higher failure consequence, and 1.50 for very important structures such as post-disaster essential facilities.
- MDOC-DS standard classifies a structure according to Group. Group B represents normal importance and represents the default spectral values. Group A represents important structures and requires that the Group B spectral ordinates be multiplied by a factor of 1.5. There is Group A+ for structures of great importance that require an extreme grade of security. No particular increase factor is specified, but a much higher 30,000-year return period ground motion is required.
- NCh2369 has structure classifications according to importance. Categories C1, C2, and C3 correspond to structures with high, normal, and low importance, respectively. The corresponding importance factor, *I*, has a value of 1.20, 1.00, or 0.80, respectively.
- ETG 1.015 classifies structures into Seismic Categories A, B, and C, corresponding to structures with high, normal, and low importance, respectively. The published response spectra ordinates are multiplied by 1.0, 0.8, or 0.6, respectively

There is an often encountered disagreement with respect to this design importance concept between US practice and other Mexican and Chilean practice. Common to all these standards is that power generating facilities are considered to be of high importance. In fact, in the US, ASCE 7 specifically considers power generating facilities to be of high importance. A problem arises because in the US, older editions of the building code categorized towers as a miscellaneous structure with a standard occupancy and normal importance. In fact, classification with higher importance applied only if the tower was "required for continued operation." Since wind farm towers were associated with intermittent (not continuous) power generation, assigning normal importance (instead of high importance) was standard design practice in the US wind industry during the early years. Recognizing this, RP2011 recommended the "normal" importance classification but observed that the local building official or Authority Having Jurisdiction (AHJ) would have ultimate say. That all being said, it has been past experience that some engineers in the subject nations assign high importance to wind farms as power generating facilities with no distinction between continuous and intermittent power generation. For example, the classification of high importance is reflected in applicable regulations of the electric power generation and distribution sector in Chile.

2.3. Structural damping

Most design response spectra in building codes are defined at a standard value of 5% damping. The subject codes are no exception. While 5% damping is considered to be appropriate for conventional building-type structures, many reference texts state the rule-of-thumb that bare steel industrial structures may be assumed to have 2% damping. Steel fabricated tubular towers similar to steel stacks (chimneys) have damping values approximately 1% or even less: 0.2% to 0.6% for unlined welded steel stacks according to ASME STS-1 *Steel Stacks*, Reference [12]. IEC 61400-1 considers a value of 1% damping as conservative. Field measurements on wind towers have confirmed this. Recognizing that bare steel industrial structures have less inherent damping, it is necessary to adjust (i.e., increase) the standard 5% damped spectral ordinates. The following are comparisons of recommended damping values for wind farm tubular towers and the resulting adjustment (i.e., increase) in the value of the 5% damped spectral ordinate for acceleration.

- ASCE 7 defines a 5% damped design response spectrum. There is no specific equation in ASCE 7 for the damping adjustment, so RP2011 recommends the following damping adjustment factor, *B*, shown in Fig. 2.

Damping (%)	B [1]
5%	1.00
4%	1.05
3%	1.13
2%	1.23
1%	1.40
[1]Factor calculated per ASCE 41-06 Section 1.6.1.5.1	

Fig. 2 – The spectral response damping adjustment factor, *B*. From Table 5-6 in Reference [8].

In the US, RP2011 recommends a 1% damping value for load combinations with earthquake load alone and the turbine at standstill, i.e., the turbine is producing no operational loading. For load combinations involving earthquake with concurrent turbine aerodynamic operational loads or fault (emergency stop), a 5% damping value is recommended. The aerodynamic damping from the operational turbine is thought to provide at least the equivalent of 5% structural damping. From Fig. 2, it can be seen that the spectral adjustment going from 5% to 1% damping is an increase by a factor of 1.4, which for design is not trivial.

- The MDOC-DS damping factor contained in the response spectrum definition is 1.0 at a default damping value of ζ_e=5%, as can be seen in Eq. (3):

$$\beta = (0.05/\zeta_e)^{\lambda} \text{ where typically } \lambda=0.45 \text{ or less} \quad (3)$$

MDOC-DS recommends a 2% damping value for steel chimneys and silos, and there is no specific value for wind farm tubular towers. Therefore, entering Eq. (3) with values of ζ_e =2% and λ=0.45 results in a spectral increase of β=1.51, which for design is not trivial.

- The NCh2369 damping factor is similarly defined, where a spectral adjustment factor is 1.0 at a default damping value of ξ=5% as can be seen in Eq. (4):

$$(0.05/\xi)^{0.4} \quad (4)$$

NCh2369 recommends a 2% damping value for steel chimneys and silos, and there is no specific value for wind farm tubular towers. Therefore, entering Eq. (4) with a value of ξ=2% results in a spectral increase factor of 1.44.

- ETG 1.015 provides a figure containing a family of explicitly graphed response spectra with ordinates adjusted according to damping. No specific equation is provided, but the results are generally consistent with the other standards. The adjustment from 5% to 2% damping is approximately a factor of 1.29. From 5% to 1% it is 1.51.

2.4. Response modification factor

Common to the subject standards are response modification factors (often denoted as "*R*") that reduce the response spectra ordinates down to a design level to account for system ductility and overstrength. As most practitioners know, these *R* factors may be as much art as science, and as a divisor to the response spectrum ordinates, the value chosen for *R* can make a very significant difference in the resulting values of design seismic force.

Of course, none of the subject standards prescribe a value specifically for wind farm tubular towers. In considering possible choices for R, it is important to observe that tubular towers have poor seismic resistance characteristics. For example, virtually all practical tubular tower designs have high D/t ratios (i.e., wall outside diameter over wall nominal thickness) and are therefore sensitive to local buckling, a limit state with little or no ductility. See Fig. 3 for an example of tower shell buckling. Moreover, the monopole, single-member system has no redundancy and no means for load redistribution. Further, the presence of a heavy wind turbine results in a top-heavy system that is more prone to secondary P-Δ effects and instability. Further still, the system has low inherent damping, which as previously discussed is accounted for by amplifying the response spectrum ordinates.

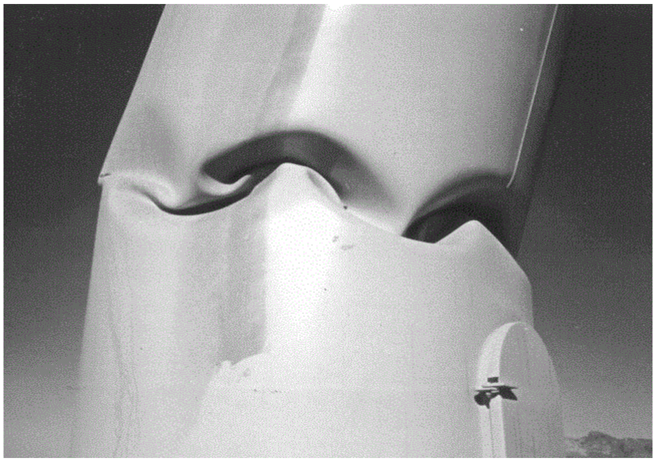

Fig. 3 – Local buckling of tower shell. From Reference [13], courtesy of ASE.

With these poor seismic resistance characteristics in mind, possible R values can be surveyed by considering plausible structural systems.

<u>Distributed mass welded steel chimneys or silos:</u>

- ASCE 7 specifies R=2 to R=3.
- MDOC-DS specifies two factors: Q to account for ductility and R_0 to account for overstrength. The standard specifies Q=3 and R_0 =1.25. However, these quantities are further subject to adjustment according to structure period, resulting in Q' and R values. Also, as discussed later, there is also a redundancy factor ρ that is a divisor to the spectral ordinates. While it is unclear whether this applies to nonbuilding structures, the most conservative value is ρ=0.8, which seems appropriate for a non-redundant single-member monopole mast. For simplicity assuming Q'=Q and R=R_0 , the total reduction factor would be ($Q'R\rho$)=3, which is generally consistent with the other standards.
- NCh2369 specifies R=3.
- ETG 1.015 specifies R=4.5.

Unfortunately, these values seem too large based on engineering judgment. They assume a distributed mass system that does not account for the top-heavy, destabilizing presence of the turbine. Further, these values do not appear to account for local buckling sensitivity.

<u>Inverted pendulum systems:</u>

- ASCE 7 specifies R=2.

- MDOC-DS specifies *R*=1.25, no ρ value, and *Q* according to the structural system comprising the inverted pendulum structure. Assuming *Q*=3 for a steel chimney, the total reduction factor would be (*QR*)=3.75.
- NCh2369 specifies *R*=3.
- ETG 1.015 specifies *R*=3.0.

Again unfortunately, these values seem too large based on intuition. While an improvement is that the top-heavy mass is accounted for, these values do not appear to account for local buckling sensitivity of the tower structure itself.

Other systems not listed:

- Assuming a MDOC-DS value of *R*=1.25, no ρ value, and *Q*=1, the total reduction factor would be (*QR*)=1.25.
- NCh2369 specifies *R*=1 for structures designed to remain elastic and *R*=2 for other structures not listed.
- ETG 1.015 specifies *R*=1 for structures formed from "fragile materials."
- ASCE 7 specifies *R*=1.25 for "all other self-supporting structures" but limits the structure height to 50 feet in the Seismic Design Categories D, E, and F, which are regions of moderate to high seismicity. Therefore, this *R* value is not viable for use due to low height limit. ASCE 7 recommends that vessel support skirts or pedestals sensitive to local buckling should be assessed with the term I_e/R set equal to 1.0 and the critical buckling resistance used with a factor of safety set to 1.0. This is interpreted to mean (in LRFD terminology, Reference [14]) that the "expected" strength multiplied by a capacity reduction factor ϕ=1.0 may serve as the design strength.

Additional guidance can be gained from general wind industry standards:

- IEC 61400-1 describes a conservative method that does not appear to use a response modification factor, i.e., equivalence to *R*=1.
- The GL Rules states that "in general, an elastic load-bearing behaviour shall be assumed for the structure," which is interpreted to mean *R*=1. It goes on to state that "if ductile behaviour is assumed, it shall be necessary to inspect the structure after earthquakes have occurred," which is interpreted to mean that some value of *R*>1.0 may be used if post-earthquake damage inspection is performed.

These values of 1.0 or slightly greater seem to be appropriate and occupy the most conservative range of values possible. Given the lack of specific guidance in ASCE 7, RP2011 recommends a value of *R*=1.5 and is careful to state that "the use of the suggested *R*=1.5 factor does not necessarily imply the expectation for ductile response or material overstrength, but accounts for other items …." The other items include but are not limited to possible conservatism in seismic loads, conservatism in tower design strength (capacity) calculations with respect to expected strength, soil-foundation-structure interaction, and perhaps some minor amount of ductility from anchor bolt stretch as observed in post-earthquake inspections of steel stack structures.

2.5. Load combinations

There are three main issues with respect to earthquake load combinations: (1) identifying the earthquake load factor that establishes the difference between the service (unfactored or allowable stress design) earthquake load and the strength design (factored) earthquake load; (2) understanding that in addition to load combinations with earthquake alone there are load combinations with earthquake and concurrent turbine aerodynamic operational loads; and (3) determining the appropriate method of combining the earthquake and operational loads.

With respect to the difference between service load and strength design load:

- ASCE 7 defines the earthquake load E as a strength design load, therefore it appears as 1.0E in the strength design load combinations. 1.0E contains an inherent 1.4 load factor, so to obtain the corresponding service load, remove the inherent 1.4 load factor by multiplying by a factor of 0.7, resulting in a service load of 0.7E.
- The MDOC-DS earthquake load is a strength design load. It is unclear what the inherent earthquake load factor is.
- NCh2369 specifies an earthquake load factor of 1.1 for steel structures.
- ETG 1.015 does not specify an earthquake load factor. The earthquake load is treated as a service load and multiplied by a 1.4 load factor to achieve a strength design load.

As it is often a point of confusion, it is important to note that it is the strength design (factored) earthquake load that is combined with the operational load.

With respect to the earthquake and operational load combination:

- ASCE 7 does not specify a load combination that includes earthquake concurrent with operational loads. As with any building code, there are load combinations with gravity loads and earthquake.
- MDOC-DS states that "these [seismic load] components will be combined with other effects that may occur under operating conditions, … and that might be critical for structural performance."
- NCh2369 explicitly specifies load combinations that include earthquake with operational loads.
- ETG 1.015 states that "seismic loads shall be combined with dynamic service loads and other loads originated by normal operation of the equipment …." Further, abnormal operational faults are to be considered: "Other eventual loads whose probability of occurrence with the earthquake is not negligible, as short circuit torques, … and so forth, shall also be considered as being simultaneous to the foregoing."

Due to the absence of guidance in ASCE 7, RP2011 specifies the following load combinations to supplement the load combinations with gravity and earthquake (alone):

$$U = (1.2 + 0.2S_{DS})D + 0.75(\rho Q_E + 1.0M) \tag{3}$$

$$U = (0.9 - 0.2S_{DS})D + 0.75(\rho Q_E + 1.0M) \tag{4}$$

where all notation is consistent with ASCE 7 and ρQ_E is the horizontal component of earthquake load and M is the concurrent operational load. The 0.75 factor is explained in the section that follows.

With respect to the method of combining the earthquake and operational load:

- ASCE 7 gives no guidance. RP2011 combines the earthquake and operational load by addition followed by a 0.75 factor. See the discussion below.
- MDOC-DS gives no guidance on how to combine the load effects.
- NCh2369 load combinations are often interpreted literally to imply combining of the earthquake and operational loads by absolute summation, but in its treatment of operational load factors, the standard suggests possible justification for reducing the operational load based on probability of its simultaneous occurrence with earthquake.

- ETG 1.015 gives no guidance but it is often interpreted to imply combining of the earthquake and operational loads by absolute summation.

It is certainly conservative to combine the earthquake and operational load components by absolute summation, which implies that the peak design earthquake load and peak operational load occur in the same direction at the same time. However, RP2011 assumes that peak design loads are correlated in neither direction nor time, making combination by square root sum of the squares (SRSS) an appropriate and rational method of statistical combination of uncorrelated quantities. The 0.75 factor on the absolute sum of the two loads closely approximates the value of the SRSS combination of the two loads for a wide range of relative differences in the magnitudes of the loads. Further, the 0.75 factor is consistent with some older code practices of using a 0.75 load factor on all transient loads when combining multiple transient loads.

It should be noted that the SRSS combination is sometimes misinterpreted to imply that the combined loads act at 90 degrees to each other. This is not the case because it is unlikely that the maximum values for these design loads would occur at the same time.

There is the design position that earthquakes in Chile tend to be of relatively long duration, resulting in a higher probability that the peak loads may align. For this reason, it is argued that the absolute sum addition of the earthquake and operational loads is more appropriate under these conditions. The counter argument is that once the tower top accelerations are large enough to trigger the emergency stop condition, the turbine operation comes to a rapid stop, and in the absence of operational load, the remaining earthquake ground motion is captured by the earthquake “alone” load combination, regardless of the ground motion duration. This is clearly a topic for further study.

It is worth mentioning another point often the subject of debate. The standards typically have some provision for the combination of orthogonal earthquake load components. All of the subject standards except ETG 1.015 mention the so-called 100%-30% method, where 100% of the design seismic force is applied parallel to a structure’s seismic force-resisting system (SFRS) and 30% of the design seismic force is applied simultaneously in the perpendicular direction. In past experience, some engineers apply this to tubular towers. An opposing view is that this is not applicable to a tubular tower as a single member system with the symmetry of a circular cross section, and ASCE 7 and NCh2369 describe applicable exemptions. The 100%-30% rule was meant to capture the load effects at intersections of SFRS such as at corner columns of intersecting building frame systems. This is also a topic for further study and clarification.

2.6. Redundancy factor

The single-member monopole mast has no redundancy, which precludes any beneficial load re-distribution in the event of overload.

- ASCE 7 has a redundancy factor, ρ, but it is equal to 1.0 for nonbuilding structures that are not similar to buildings. ρ is a multiplier that increases the earthquake’s horizontal load effect.
- MDOC-DS has a redundancy factor, ρ, that is a divisor that can increase or decrease the spectral ordinates. The lowest value (for the least structural redundancy) is 0.8, which would result in a 1/0.8=1.25 factor increase on the spectral ordinates.
- NCh2369 has no redundancy factor.
- ETG 1.015 has no redundancy factor.

3. Conclusions

In the context of wind farm tubular tower design, this paper has compared and contrasted the seismic design provisions of the US’s ASCE 7 and RP2011, Mexico’s CFE MDOC-DS, and Chile’s NCh2369 and Endesa ETG 1.015 specification. Similarities and differences were identified. The relative conservatism of assumptions and approaches of the design practices as applied to wind towers were shown.

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