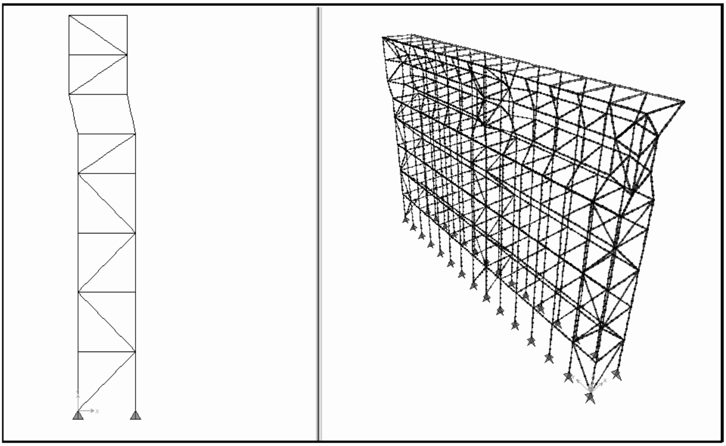

**Figure 9. The supporting structure strip with inadequate lateral bracings.**

**Assessment of P-Delta effects in the scaffolding piers.** P-Delta is a non-linear effect that occurs in every structure where elements are subjected to axial loads. As structures consist of more slender and less resistant to deformation elements, the necessity of considering the P-Delta effects increases. Since the magnitude of P-Delta effects is related to axial loads as well as slenderness of individual elements, it seems that investigation of this effect in the accident of Bojnourd cement project would be valuable. As mentioned, the use of steel bars as connectors with diameter smaller than the internal diameter of scaffold tubes can be considered as a weak point during the formwork collapse. As it can be seen in Figure10, the relative rotation and resulted out of plumb about 7 degrees between upper and lower elements in the joint spots due to existence of the void space can lead to a 360-milimeter horizontal displacement from vertical axe for every 3-meter-high vertical pier; this can cause increase of internal stresses in elements or P-Delta effects due to vertical and horizontal loads. Therefore, corresponding P-Delta effects in the braced structure elements even in absence of lateral loads can result in decrease of elements strength in compression and formation of buckling in several elements situated in the lower elevations of the supporting structure. Thus, in order to investigate the P-Delta effects, one of the scaffolding strips has been modeled under applying only the vertical loads and relating effect of the 360-mm displacement occurred between two adjacent 3-meter-high piers. It was illustrated that the lower elements of the piers which support a 3.2-tone axial load in the common state, experience an axial load about 4.5 tones in case of this eccentricity existence (see Figure 11). Since the comparison of the results of two analyses, with and without P-Delta, illustrates the magnitude of the P-Delta effects (Rutenberg, 1982), for the mentioned structure with these improper connections, the contributions from the P-Delta effects are highly amplified and can change the internal forces by 40 percent or more. Consequently, this increase in displacements and internal forces could be one of the main causes of the formwork collapse.

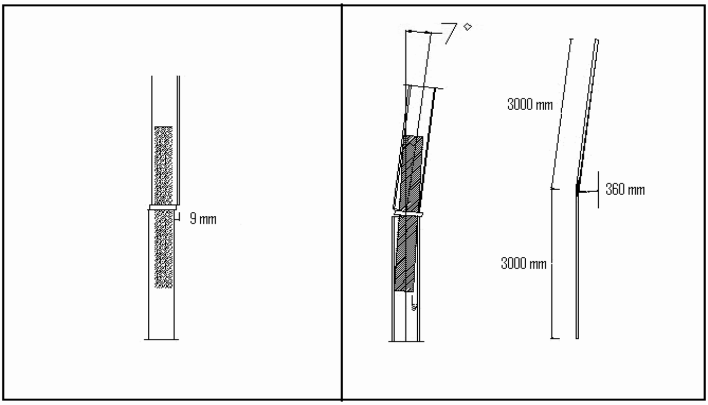


**Figure10. Relative rotation and resulted out of plumb between upper and lower elements in the joining location of piers.**

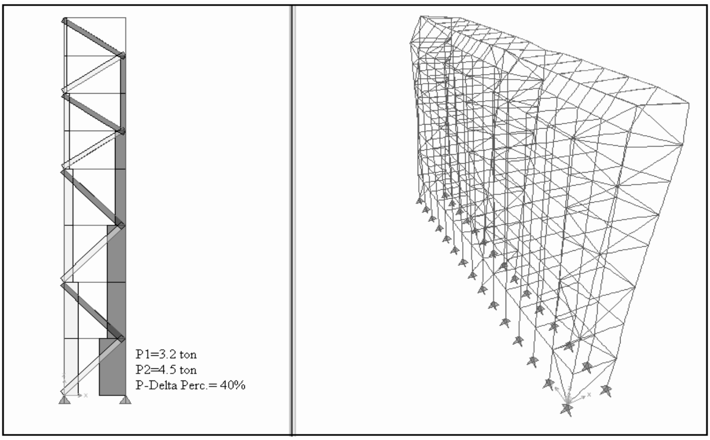


**Figure11. P-Delta effect on the internal compressive forces of a supporting pier.**

## SUMMARY AND CONCLUSIONS

The failure of forming system related to the concrete slab of the bypass clinker silo of Bojnourd cement factory was assessed. Based on the in-situ investigations mentioned in this study and numerical analyses performed on the formwork supporting structure, the main causes of the formwork collapse can be listed as below:

- The lateral bracing for vertical scaffolding piers particularly in the 3-meter-high lower piers was performed inadequately and also diagonal elements were not used properly.

- The connecting horizontal elements used between the separate piers did not have enough capacity to create an integrated lateral supporting system.
- The use of a 32-milimeter-diameter steel bar to join the scaffold tubes with 44 mm internal diameter led to a 360-milimeter horizontal displacement for every 3-meter vertical pier and creation of P-Delta effects in these elements.

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## Defects, Damage, and Repairs Subject to High-cycle Fatigue: Examples from Wind Farm Tower Design

by

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### Abstract

Wind farm towers are the steel lattice or tube towers serving as the support structures for large power-generating wind turbines. This paper presents a variety of wind farm tower problems encountered over two decades of practice in the wind energy industry. Wind towers are subject to turbine operational loads that cause very high-cycle fatigue loading and can give rise to unusual tower problems requiring special repairs. Problems unique to wind towers include: structural damage or collapse from turbine overspeed ("runaway" condition); tower resonance with turbine operational frequencies; turbine fires; and excessive localized member vibrations. Further, wind towers can suffer from construction-related problems: fabrication and weld defects; design deviations; accidental dents and gouges during transport; construction errors during tower erection; and wind-induced vortex shedding oscillations. Each tower problem is discussed in terms of its cause, evaluation method, mitigation strategies, and repair options with special consideration for high-cycle fatigue.

### Introduction

**Wind Tower Structural Systems.** Figure 1 shows typical lattice and tubular tower structures. The lattice tower legs and diagonal web members are steel angle (i.e., L-shape) members that are connected with high-strength bolts at their joints. The tubular tower is a fabricated tube consisting of "can" segments formed from flat steel plate. Figure 2 shows how cans are welded together to form longer tubular sections. The lattice towers were used predominantly in the early days (early 1980's) of the US wind industry. Lattice towers ranged from heights of 20 m to 60 m. However, in the last decade, the utility-scale wind industry has transitioned to almost exclusive use of tubular towers that routinely reach heights of 80 m.

**Codes, Standards, and Certification.** The commercial wind turbine industry evolved in Europe, and European wind turbine manufacturers continue to dominate the global market. Since the US has no specific wind turbine tower design standards, European wind industry standards are the basis for most wind tower design. Wind turbines and towers are certified by a handful of established European certification



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agencies, such as Germanischer Lloyd (GL) and Det Norsk Veritas (DNV). Certification is achieved by demonstrating design compliance with IEC standards, e.g., IEC (1999), or agency-specific standards, e.g., GL (2003). Achieving certification is the industry standard for establishing the commercial legitimacy of a turbine-tower system in a way that is analogous to a UL listing for an electronic device or to an ICC test report for a structural product.

Figure 1. Typical lattice towers (left) and tubular towers (right). (Photographs courtesy of MSKA.)

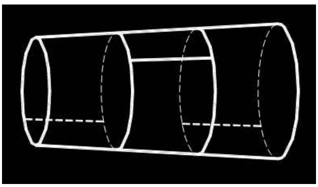

Figure 2. Illustration of a fabricated tube comprised of "can" segments welded together at circumferential weld seams. (Detail courtesy of MSKA.)

**High-cycle Fatigue.** A typical modern utility-scale wind tower with a 20- or 30-year design life is subject to "megacycle" fatigue, which is low-stress, high-cycle fatigue on the order of several hundred million cycles. It is not unusual for fatigue loading to govern over the usual Code wind or seismic forces. For these reasons, consideration of fatigue is imperative in the assessment of defects, damage, and repairs. Based on provisions of Eurocode 3, Section 9, (ENV, 2002) detailed examples of wind tower fatigue calculations using *S-N* curve methods can be found in Agbayani and Kyatham (2008 and 2009). Fatigue damage is calculated using Miner's Rule linear damage summation method (Miner, 1945).

## Overview of Damage and Defects in Wind Towers

In the discussions that follow, tower problems are grouped by structure type (lattice or tube). However, a few problems common to both tower structure types are discussed here.

**Turbine "Runaway."** Near or total tower collapse can be caused by turbine "runaway," which is a condition of uncontrolled turbine rotor overspeed. The typical scenario starts with a mechanical or electrical failure in the turbine's rotor braking systems. For example, the failure of a turbine's lightning protection system might leave other electrical systems susceptible to failure after lighting strike. Excessive blade rotation speed breaks off a turbine blade, resulting in both a blade projectile and a large magnitude eccentric rotor loading condition. By chance should the projectile miss the tower, the remaining blades can strike the tower due to the unexpected rotor deflections from the eccentric loading. Figure 3 shows a tower with local buckling damage due to a runaway turbine scenario.

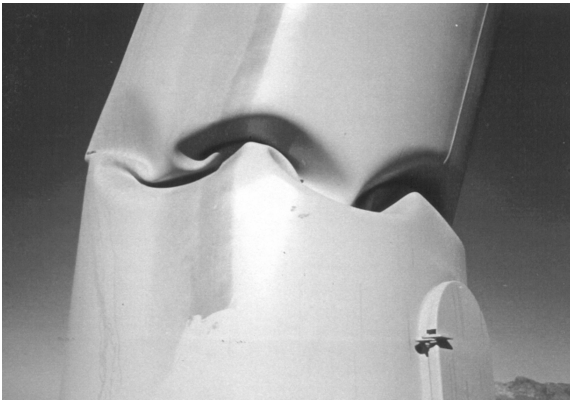

Figure 3. A tubular tower with local buckling failure after a turbine overspeed condition due to lightning strike. (Photograph courtesy of MSKA.)

**Tower Resonance.** Most modern wind turbines have one or more fixed operational frequencies. The system itself (consisting of the turbine, tower, and foundation) has natural frequencies corresponding to the system's natural modes of vibration. The fundamental mode is the simple back and forth lateral displacement of the tower as an inverted pendulum. Resonance characterized by the build-up of large displacements may occur if there is inadequate separation between a turbine operational frequency and a system natural frequency. Lack of frequency separation could stem from gross error in tower design or from more subtle problems such as the turbine being significantly out of specification for mass or operational speed. A resonance scenario usually involves large tower displacements shortly after start of turbine operation. When the turbine sensors detect excessive displacement, they trigger a fault condition known as an "emergency shutdown" to protect the tower from damaging oscillations. The result is lack of power generation. Possible retrofit measures to provide frequency separation include: shifting the system frequency by stiffening the tower or increasing the tower top mass; or changing the turbine operational frequency usually by reprogramming its controller to operate slower (unfortunately resulting in less power generation).

**Turbine Fires.** A turbine may catch fire due to an electrical or mechanical problem. An assessment is made to separate cosmetic damage from more serious damage that has affected the chemistry and mechanical properties of the steel. Affected portions of the tower are usually scrapped and totally replaced with new components.

## Damage and Defects in Lattice Towers

The bygone era of small lattice towers (early 1980's) has now transitioned into the era of large tubular towers. For this reason, the defects and damage in lattice towers are only briefly mentioned here for the sake of historical interest. Fleets of lattice towers have now fulfilled most of their 20- to 30-year design life, and many towers have outlived their original turbine. However, no structural system is perfect, and along the way various problems have been encountered with lattice towers.

**Lack of Field Fit-up.** With lattice towers comprised of hundreds of members, connection plates and bolts, the occasional lack of field fit-up is usually due to a fabrication error. Erection of a single initial prototype tower essentially ensures that any blunders in geometry contained in either the engineering plans or the fabricator shop/assembly drawings can be corrected before the mass production phase.

**Excessive Local Vibrations.** Lattice towers can be susceptible to local vibrations. In contrast to overall tower vibration, specific pairs of web members or specific groups of web members may undergo excessive out-of-plane vibrations. Continual vibration during turbine operation would point to local resonance with an operational frequency, a rotor imbalance, or gearbox problem. Sporadic local vibration may indicate resonance of a transient nature such as with a pass-through frequency during turbine start-up. Inevitably, loose bolts accompany excessive vibration. An effective remedy for a loose bolt is usually a new replacement bolt properly pre-tensioned and included with a nut-locking device (for redundancy).

**Fatigue Cracks in Welded Connections.** While lattice towers are comprised mostly of field-bolted connections, a few welded connections exist in lattice towers such as at the leg-to-top mounting ring connection. Fatigue cracks have been observed in tower top ring plates, usually at locations of stress concentrations or with high residual stresses.

**Other Damage.** Other problems are known to have befallen lattice towers: flood damage undermining or scouring the foundations; intentional vandalism, such as bolt removal, with intent to cause full or partial tower collapse; accidental vehicle impact; total tower collapse due to overwhelming extreme wind event, etc.

## Damage and Defects in Tubular Towers

**Tower Wall Dents.** Towers occasionally are dented by mishandling or accidents during fabrication, transport, or erection. See Figure 4. Excluding fatigue failure and connection failures, the most likely design limit state (or failure mode) for a tubular tower structure is local buckling of the tower shell. See Figure 3. For this reason, a dent in a tubular tower shell is considered a serious problem, because a dent is like the onset of a local buckle. With a dent, the very condition that the structural design attempts to avoid has already occurred. According to Troitsky (1990):

> About the worst thing that can happen to a tubular structure either during construction or operation is for it to develop a buckle. The buckle may be defined as a localized failure in the form of a wrinkle or indentation caused by over-stress or instability of the wall of tubular structure on the compression side of the structure subjected to bending. . . . Theoretical analyses . . . of the effect of imperfections on the buckling behavior of cylinders have clearly demonstrated that relatively small imperfection amplitudes can drastically reduce the critical load of the shell. . . . The results of the . . . analysis indicate that an initial imperfection amplitude equal to the shell thickness is sufficient to reduce the buckling load to only 20 percent of the corresponding value for the perfect cylinder.

For these reasons, dents in the tower wall should be limited in size to that allowed by defined fabrication tolerances. Dent tolerances are found in various European standards such as DIN 18800. Should a dent's size exceed tolerances, then a repair is necessary. Repair involves the careful pushing out of the dent until it is either removed or at least until the dent's size is reduced below maximum tolerances. Where a dent affects a main tower weld, the weld is subject to NDT inspection after the dent repair.

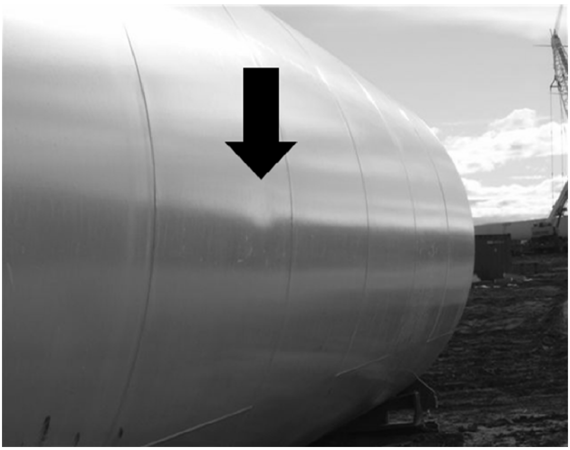

Figure 4. Dent in tower shell. (Photograph courtesy of MSKA.)

**Tower Wall Surface Defects.** Wall surface scratches, indentations, and gouges may result from accidents or mishandling. See Figure 5. Other examples of surface defects include: identification marks by physical indentation; miscellaneous streak indentations from unknown contact or rubbing; intentional fabrication marks; rolling marks, etc. While they may seem insignificant from the standpoint of tower ultimate strength, surface defects have the potential to be locations of fatigue crack initiation, so these otherwise minor defects are considered to be a serious problem when subject to high-cycle fatigue. At the very least, the small surface defects should be ground smooth to prevent stress-rising notch effects. Plate defect tolerances are usually based on engineering judgment informed by plate defect limits such as found in ASTM A6 or weld notch/undercut defect tolerances as found in AWS (2004). For deeper defects, base metal repair by welding is an option. However, in light of the high-cycle fatigue environment, repair by field welding should be considered a last resort and undertaken with great caution. The possibility exists that a low-quality repair by field welding may result in the weld material itself cracking or acting as an even larger notch. To prevent the "fix" from being worse than the problem, a repair procedure and welding specification should be developed in consultation with an expert welding engineer who can address weldability issues.

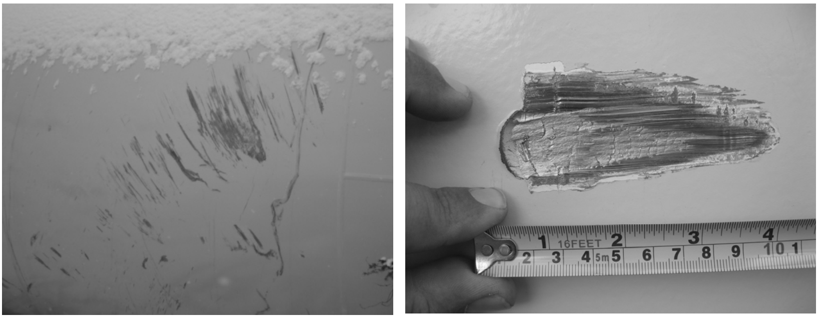


Figure 5. Scratches (left) and severe gouge (right) on tower wall. (Photographs courtesy of MSKA.)

**Tower Paint/Coating System Problems.** The tower paint/coating system protects the tower from corrosion. When the tower paint is damaged or found to be locally flawed (e.g., peeling, blistering, etc.) it is imperative to perform robust paint repairs compatible with the original 20- or 30-year-life coating system. Specialized paint specification is best left to knowledgeable coating specialists. Interestingly, while foul weather exposure is easy to imagine for a tower's exterior, less obvious is routine moisture exposure on the interior surfaces of a tubular tower due to condensation in the slightly warmer conditions within the tower caused by heat from electrical systems. Similarly, corrosion too has a subtle aspect. Obviously, extensive corrosion can reduce the strength of a structural component, but a less obvious consideration is the aggravating effect of corrosion on fatigue. Corrosion-fatigue is a complex phenomenon, but stated in simple terms: corrosion in the presence of fatigue increases susceptibility to cracking, hastens the onset of cracking, and reduces fatigue life. The surface roughness, pitting, cracking, notching effects, and the resulting chemical and material property changes in steel that result from corrosion are contributing factors to this behavior. In a high-cycle fatigue environment, maintaining the integrity of the paint is especially critical to prevent corrosion and to prevent premature fatigue damage.

**Tower Wall Can Joint Offsets.** The tower is a fabricated tube consisting of "can" segments welded together with circumferential weld seams (i.e., "girth" welds). See Figure 2. Where the plate edges at the can-to-can joints do not abut within the permissible tolerances for misalignment, repair or mitigation is necessary. The misalignments cause additional stresses due to joint eccentricity resulting in reduced fatigue life. The circumferential can joints must comply with constructional details that have a fatigue Detail Category (DC) classification. The DC usually has a misalignment tolerance built into its definition. The flawed detail can be assessed accounting for the misalignment. A satisfactory fatigue life may indicate that repair is not required. However, if the risk or engineering judgment requires a repair, a mitigation strategy is to provide fatigue life increase by applying post-weld improvement techniques such as weld profile improvement. The stress-riser effect of the joint offset can be removed by grinding the weld surface smooth with a flush slope transition across the joint weld surface. Fatigue life enhancements are described in Section 8.4 of AWS (2004) or Section 3.5.3 of IIW (2003).

**Tower Splice Flange "Toe-to-Toe" Contact.** Splice flanges are the bolted ring plates that join tower sections. A cross section through the tower flange is L-shaped somewhat like a person's foot with the flange having a "heel" and "toe." Figure 6 depicts the required "heel-to-heel" initial contact condition for splice flanges coming into contact. When two tower sections are joined together during the erection process, the high-strength splice bolts are pre-tensioned by application of an installation torque. Ideally, the splice flanges should close with perfect flat contact, but in reality, contact is not perfect. Of the imperfect contact conditions, initial heel contact is permissible, but initial toe contact is prohibited. The reason is fatigue. Clamping the flange closed starting from initial *toe* contact represents a pre-pried condition that leaves the splice bolts susceptible to fatigue. In contrast, clamping the flange closed starting from initial *heel* contact provides a load path through the heel contact pressure that protects the splice bolt from prying forces. The repair is flange contact surface grinding to provide either perfect flat contact or initial heel-to-heel contact.

**Tower Splice Flange Gaps.** When flanges make initial heel contact, a toe gap (lack of contact at the flange toes) naturally exists. See Figure 6. The toe gap disappears as the flanges are clamped together during bolt installation. Flange toe gaps result when the full effort of the bolt pre-tension installation fails to close the initial gaps. See Figure 7. This condition is a problem because the splice bolts at gaps are more susceptible to fatigue damage. Gaps prevent the desired load paths and contact stress distributions assumed in the flange analytical models used for design. Residual flange gaps are typically discovered after tower erection but just after the bolt installation process, so disconnecting the tower sections for flange grinding is not a practical option. Careful flange gap shimming using precision stainless steel shims is the typical repair. The width and extent of flange gaps are measured and mapped around the entire flange circle. The *GL Rules* (GL, 2003) requires that the shimming material match the elastic modulus of the steel flanges, so this prevents the use of