# Wind load and gust effects on slender steel arches

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ABSTRACT: The frequency of wind gusts appears to be much lower than common values of the natural frequencies of steel bridges. Simulation of several time-dependent spectra of wind gust loads related to measurements demonstrates that resonance effects are rare in steel arch bridges. The simulations are based on saw tooth and half sine wave patterns. This is consistent with results from the relevant Eurocode, showing that the dynamic factor from this code is too low to introduce critical behaviour with respect to arch stability. Nevertheless the magnitude of effective wind speed from this code appears to be surprisingly high.

## 1 INTRODUCTION

### *1.1 Wind and wind gusts*

Wind loads depend on the basic wind speed, the type of location, the height above the reference level and dynamic characteristics, both of the structure and of the wind itself. The first parameters determine the static wind pressure on a given structure, whereas the influence of wind speed variations and the response of the structure are included in dynamic factors applied to the basic pressure. These factors are determined by the structural response and the natural frequencies, structural damping and the influence of wind gusts. The value of the factors may become fairly high, especially for large span and narrow bridges, due to the lateral mode shapes of such structures and the low fundamental frequency.

It is generally accepted that the kinetic energy caused by short-term wind effects reaches a maximum value for a period of 1.2 minutes. On the other hand, the period of the most heavy wind gusts is closer to 10 or 12 seconds (Bierbooms 1988). This discrepancy may result from confusion between maximum of energy and the response of flexible structures to short-term loading variations.

A new method of assessment of wind loading on structures has been proposed as Data-Assisted design (Diniz and Simiu 2005). It allows taking into account the inherent randomness in the estimation of peak values of wind speed. This probabilistic method may constitute a major contribution to the assessment of the behaviour of flexible structures exposed to wind gusts. However, it still considers the use of load factors to be applied to conventional design wind load.

A different approach might be tried, by using measured values of instantaneous wind speed and directly compare these to the effects on slender structures. This may result in better understanding on the combined effect of variable wind and traffic on slender bridges, especially in relation to fatigue resistance.

### 1.2 *Wind and traffic*

The results of measured values of wind gust periods mentioned in section 2, clearly demonstrate that the dynamic effect of wind are unlikely to cause resonance with structural vibrations of steel bridges. Obviously, both types of loading relate to separate frequencies and are unlikely to interact. Apart from the wind action on the vehicle itself, the direction of both types of loading is different. Traffic loading is mainly vertical, whereas the most important vector of wind action can vary to any direction, although wind direction may be preferential at a given location. Regarding structural behaviour, horizontal wind load generally results in maximum effects and should be considered as the relevant design condition

## 2 WIND GUSTS

### 2.1 *Measurements*

Whether data assisted design or analysis based on effects of wind gusts be applied, the sampling of wind parameters becomes necessary. A test setup consisting of anemometers and digital high-frequency registration was installed at Zeebrugge along the Northsea coast. The equipment allows sampling of wind speed and direction at a frequency of 10 Hz. The sampling is digitalized and completely automatic, if downloaded regularly. Fig 1 shows the setup, placed on an outward sea pier, built especially for monitoring purpose at 6 m above high tide water level.

Figure 1 : Measuring setup on pier.

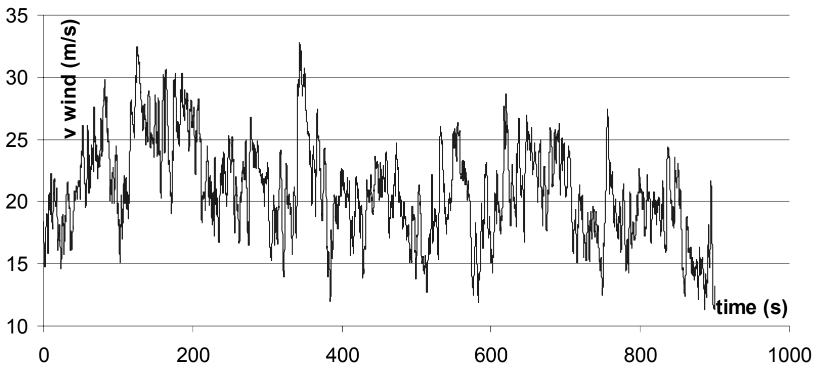


Figure 2 : Wind speed during storm of February 8th.

Fig 1 shows a typical wind spectrum as measured at the setup, by a graph of a 15 minutes storm on February 8$^{th}$ 2004 between 7.05 and 7.20 AM. We may notice a maximum wind speed of 32.82 m/s recorded once during sampling at a frequency of 10 Hz. During the four years of observation from 2003 to 2006 no heavier storm was found than the one of fig. 2. Obviously, the period of 15 minutes is too long to detect any gust period and statistical interpretation becomes necessary.

### *2.2 Statistical Interpretation*

The data from fig 2 can be interpreted in various ways. The maximum wind speed can be defined as the 97.5% probability value. If all recorded values are considered $v_{max\ 97.5}$ equals 28.49 m/s. However, this cannot be considered as the maximum wind speed during gusts. Another approach may consist of locating 2 periods of about 1 minute of heavy wind as can be seen in fig 2. If these are considered the value of $v_{max\ 97.5}$ equals 31.91 m/s. A third way of determining maximum speed during gusts may consist of determining $v_{max\ 97.5}$ from the 10 maximum peak values during the 15 minutes interval, equalling 33.48 m/s.

A second characteristic for describing numerically wind gusts is the duration or period. The time interval covered by the graph of fig. 2 is too wide to actually find any gust period. Hence, more detailed parts of the recordings are to be considered as in fig. 3, covering 100 seconds.

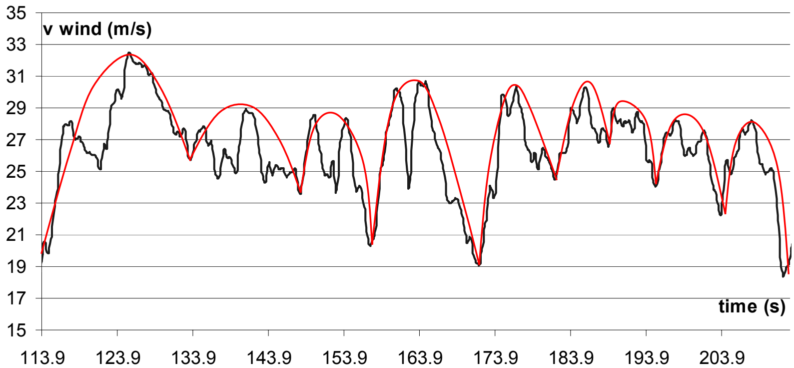


Figure 3 : Wind speed interval of 100 seconds.

Although there is no obvious pattern of the variation of wind speed with time, the lines drawn on top of the actual graph might be considered as a fair approximation. The exact shape of these lines can be discussed further. Nevertheless, determining periods of the individual wind gusts should be possible by counting the number of peak values for a given interval. Depending on the number of peak values being considered, the period values of table 1 are obtained.

Table 1 : Gust periods.

| Number of peaks | 33 | 110 | 264 | 355 | 503 | 622 | 715 | 833 | 934 |
|---|---|---|---|---|---|---|---|---|---|
| Period T (s) | | 7.7 | 15.4 | 9.1 | 14.8 | 11.9 | 9.3 | 11.8 | 10.1 |

The mean value of the gust period found from these data equals 11.2 seconds. In addition, the lines drawn in fig 3 max are identified to half-sine waves or as saw tooth functions. Both types may be considered as sufficiently accurate to describe actual loading by wind gusts. However, this also requires determining the characteristic maximum and minimum values for each cycle. The relation between these values must be considered for each interval and may not be derived from the extreme limits for the whole storm.

After consideration of the 100 second interval of fig 3, the characteristic 97.5% probability values of $v_{max\ 97.5}$ = 31.8 m/s and $v_{min\ 97.5}$ = 20.6 m/s have been found. The values apply for 6 m height above sea level.

## 3 SLENDER STEEL ARCH BRIDGES

### 3.1 Railway bridges

During the past years about 11 large and medium span steel railway bridges have been built in Belgium, mainly for new high-speed railway lines. Since deformation is an important criterion for these bridges, the hangers, connecting the steel arches with the lower chords have a triangular arrangement, since deflections are effectively smaller than for classical vertical arrangement.

The largest of these bridges is found in Schaarbeek, near Brussels. It allows the crossing of the high-speed line from Brussels to Cologne with domestic lines. The total span of this bridge equals 136.2 m the arch rise being 21.6 m. The bridge floor consists of steel orthotropic plated decking, the tracks being placed on ballast.

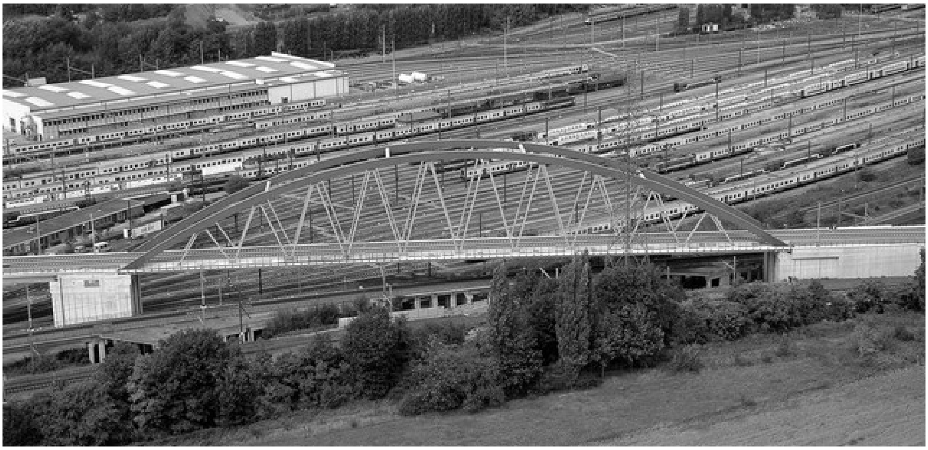

Figure 4 : Steel tied arch bridge Schaarbeek.

Similar bridges were built near Lontzen, Leuven, Antwerp and Halle. A particular case are the twin bridges built across the Ringvaart canal near Ghent, because these are very narrow bridges, of 5.4 m width for single track and spanning 120 m, the arch rise equalling 20 m. The Ringvaart bridges are particularly sensitive to loading by wind gusts. Fig. 4 clearly shows the light upper wind bracing of these bridges, which is distinctly more aesthetical than other complicated arrangements.

### 3.2 Effect of altitude

Before being able to use the half-sine wave of saw tooth pattern of wind spectra on numerical models of arch bridges, the wind speed values have to be adapted to the actual height of the structure. This can be processed by applying the exposure factor $c_e(z_e)$ from Eurocode 1991-1-4:2005, given by

$$c_e(z) = k_r^2\, ln^2(z/z_0)\; (1 + 7\, I_v(z)) \;=\; k_r^2\, ln^2(z/z_0)\; \left( 1 \;+\; \frac{7\, k_l}{c_0(z)\; ln\,(z\,/\,z_0)} \right) \qquad (1)$$

The exposure factor (1) includes the turbulence intensity $I_v$ at height z and short-term velocity fluctuations, as well as the terrain factor $k_r$. Application of this factor to experimental wind speed values, inevitably includes part of the dynamic effects of wind, because of the fluctuations. However, it seems the only way to deal with the variation of wind velocity with the altitude. In addition the turbulence is accounted for by the factor $k_l$, which can be taken as 1, to obtain the exposure factor for a steady state. The reference height $z_0$ for coastal regions equals 0.003 m, whereas the orography factor $c_0(z)$ also equals 1, except for sloping grounds. For the coastal region of the North sea and the measuring height of 6 m, the factor $c_e(z)$ =2.7 which actually means that the maximum wind speed at sea level equals 19.5 m/s.

*3.3 Application of spectra*

Using expression (1) for transforming the measured velocities, allows applying the half-sine wave and seeing tooth spectra to numerical models of the aforementioned steel arch bridges. For this, modal analysis and transient dynamic response have been used. The time step was lower than 0.5% of the gust period. In (Van Bogaert and Jutila 2006) the case of the Ringvaart bridges was already examined.

Considering the steel tied arch bridge at Schaarbeek fig. 5 shows the horizontal displacements and accelerations at the arch top. Because of the effect of altitude, the values of $v_{max\ 97.5}$ = 94.7 m/s and $v_{min\ 97.5}$ = 61.3 m/s have been used.

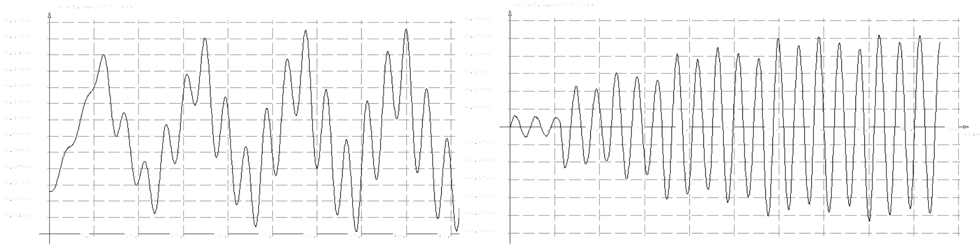

Figure 5 : Arch top displacements (left) and accelerations (right).

The excitation spectrum can be noticed in the displacement chart during the first cycles. The subsequent cycles no longer correspond to a saw tooth pattern, whereas the accelerations are built up gradually and the response shows a vibration having a period of 1.75 s, which does not correspond to any particular natural mode, the lowest frequency of the arches being 0.437 Hz and of the deck 1.402 Hz. The arch top displacements are limited to 104 mm, whereas the accelerations can reach 0.26 m/s². Consequently there is no resonance between the wind gust effect and the steel arches.

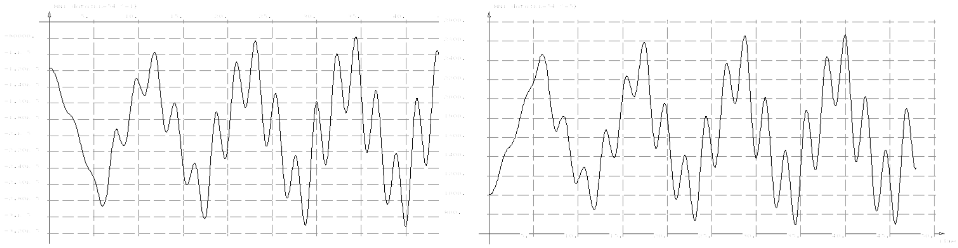

Figure 6 : Arch top compression force (left) and out-of-plane bending moment (right).

Looking at the arch compression force and bending moments, the results of fig. 6 are obtained. These correspond to the pattern of displacements rather than to the accelerations. This is due to the absence of resonance of the structure to the wind gusts.

Identical conclusions are found when using half-sine pattern in stead of the saw tooth scheme. The horizontal displacements for half-sine wave pattern are 4.5 % larger whereas the accelerations show no differences.

As the wind data may vary for other locations, the main parameters of wind gusts may be varied. Obviously, the amplitudes of wind cycles seem important. However $v_{max\ 97.5}$ and $v_{min\ 97.5}$ are strictly linked and modification of one value automatically implies a change of the other one. If the set of values is modified to $v_{max}$ = 61.4 m/s and $v_{min}$ = 72.2 m/s, the lateral arch displacements lower by 8.3%, far less than the decrease of the wind pressure by 28%. The arch compression force is lowering in the same manner. This demonstrates that the response of the steel arch does not vary linearly with the actual wind pressure. The normal compression force and the out-of-plane bending moment show identical variation as the displacements.

### 3.4 *Influence of gust period*

An important factor is the gust period, since resonance is linked directly by this quantity. The initial wind spectrum is used with varying periods of 1, 2.8, 5.6, 11.2, 15 and 22.4 seconds. The maximum horizontal displacements are listed in table 2.

Table 2 : Maximum horizontal displacements for various gust periods.

| Period T (s) | 1 | 2.8 | 5.6 | 11.2 | 15 | 22.4 |
|---|---|---|---|---|---|---|
| Displacement (mm) | 100 | 120 | 107 | 104 | 109 | 104 |

The period of 2.8 s distinctly shows larger amplitude of the arch top displacement. It corresponds to half the natural frequency of the deck and a fraction of 0.82-times the natural frequency of the arches. The variation with time of the displacements and accelerations is shown in fig. 7. The type of this graph differs fundamentally from fig. 5, demonstrating the resonance effect. Similar results were obtained for the arch compression force and the out-of-plane bending moment.

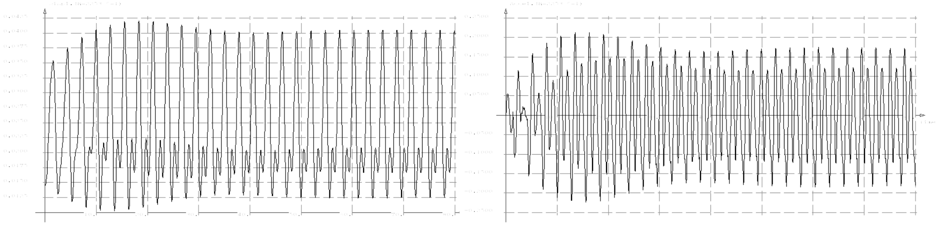

Figure 7 : Arch top displacements (left) and accelerations (right) for period of 2.8 s

The resonance does not occur at a period of 1 s or 0.5 Hz frequency, or 1.144-times the free vibration frequency of the arches. Hence, the resonance appears to be due to excitation of the deck. However, as the resonance point is somewhat lowered by damping, spectra having frequencies just below the free vibration value are likely to return the behaviour of fig. 7. In this particular case the conclusion might be that the resonance is due to excitation close to the arch system free vibration, although spectra close to half the deck free vibration mode should also be considered.

It should be stressed that the wind gust periods as considered in table 2 were never found during the observations along the North sea coast and are well out of the range of the values from table 1 and have almost inexistent occurrence probability.

## 4 ACTUAL DESIGN WIND SPEED AND EUROCODE

### 4.1 *Interpreting Eurocode factors*

As demonstrated in section 3.2 the relevant Eurocode 1991-1-4 renders expressions, which unfortunately combines the effect of altitude and short-term velocity fluctuations. The latter have been used directly in the aforementioned simulations. These results should be compared to the approach of Eurocode.

Apart from the altitude factor $c_e(z)$ and the force coefficient $c_f$ which is determined by the type of cross-section of the element, wind pressures should be multiplied by the structural factor $c_s\, c_d$ consisting of the size factor and the dynamic factor.

The size factor $c_s$ takes into account the reduction effect of the wind action to the non-simultaneity of occurrence of the peak wind pressures on the surface. The size factor is given by

$$c_s = \frac{1 + 7\, I_v(z_s)\, B}{1 + 7\, I_v(z_s)} \tag{2}$$

The height $z_s$ is the reference height for determining the structural factor, which is readily taken as the height z itself. The factor $B^2$ is called background factor, taking into account the lack of full correlation of the pressure on the structure surface. Eurocode 1991-1-4 introduces the size factor as inseparable from the dynamic factor. In fact, through the background factor, the concept of the size factor also uses the turbulent length scale *L(z)*, representing the average gust size for natural winds.

In order to clarify some of these quantities for the bridge of fig. 4, the length scale *L(z)* = 103.38 m, and the background factor $B^2 = 0.458$. This means that the simulations, using a single spectrum may well overestimate the effect of wind gusts. Consequently, the size factor $c_s$ = 0.763 lowers the effect of the wind with about 25%. While applying saw tooth or half-sine wave spectra, the factor $c_s$ should be considered.

The structural factor is obtained by multiplying $c_s$ by the dynamic factor $c_d$ which takes into account the increasing effect from vibrations due to turbulence in resonance with the structure. This factor is obtained from

$$c_d = \frac{1 + 2\,kp\,I_v(z_s)\sqrt{B^2 + R^2}}{1 + 7\,I_v(z_s)\,B} \tag{3}$$

The dynamic factor is in fact the quantity resulting from possible resonance and should be consistent with the values found in 3.3. In the present case of the Schaarbeek bridge a value of 1.558 has been found.

### *4.2 Effective wind velocities*

Taking into account the various results from Eurocode and the actual wind spectrum being used, the maximum wind velocities can be compared. Obviously this must be interpreted cautiously, since the former are design values, whereas the latter are taken from observations, although they were transformed to the actual height of the structure and required multiplication by the force factor $c_f$ depending o the properties of the cross-section.

For the Schaarbeek bridge of fig. 4 the maximum wind pressure on the arches, according to Eurocode equals 1961.7 N/m². Taking into account a force factor of 1.3 and the size factor of 0.763, the corresponding wind velocity equals 56 m/s or 202 km/h. If the terrain category is transformed to coastal regions, by using the exposure factor $c_e$, the design wind speed at 25.8 m height reaches 60.71 m/s or 218.6 kmh

The measured value of maximum wind speed equals 31.8 m/s. At 25.8 m height this becomes 54.9 m/s or 197.6 km/h. Obviously the Eurocode value overestimates wind velocity by 11% or results in increasing the wind pressure effect on the structure by 22%.

Similar calculations were carried out for the much smaller arch bridge Ijzer Lane near Antwerp. This bridge has a span of 56 m, the fundamental frequency of the arches being 1.047 Hz. The wind pressure according to Eurocode equals 1739.4 N/m². From this, the design wind speed at 16.6 m height and transformed to coastal region becomes 59.1 m/s or 212.8 km/h. If the measured maximum wind speed at 16.6 m height equals 52.2 m/s or 188.1 km/h, Eurocode may overestimate wind velocity by 13% or wind pressure by 28%.

Although the simulations of 3.3 are entirely based on observations during a single storm and more heavy weather conditions might possibly occur during the structure's life time, it should be remembered that $v_{max}$ is a 97.5% statistical value. This might imply that Eurocode design wind velocities are rather large.

## 5 CONCLUSIONS

Observation of instantaneous wind speed during storms along the North sea coast has shown that the period of appearance of wind gusts is close to 11 seconds. Statistical interpretation of the measured wind velocities enables to derive 97.5% probability statistical values of the maximum and minimum wind velocity during cyclic wind gust loading.

The gust spectra have subsequently been transformed to the actual height of recently built steel tied arch bridges. Modal analysis and transient response calculations have shown that resonance of steel arches with the determined cyclic wind loading do not result in resonance effects. In fact, the arch compression force and out of plane bending moments vary according to the displacements rather than to accelerations. This can be explained by the fact that the free vibration frequencies of the structures being considered are fundamentally different from the frequency of appearing wind gusts.

The observed wind velocities have been compared to the design values from Eurocode 1991-1-4. This shows that the effective speed from the latter design method may be rather high and certainly is on the very safe side.

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