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Fatigue of tubular steel lighting columns under wind load

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Abstract. Lighting and traffic signal columns are mainly stressed by excitation due to natural, gusty wind. Such columns typically have a door opening about 60 cm above ground level for the connection of the buried cable with the column's electric system. When the columns around this notch are inadequately designed, vibrations due to gusty winds will produce considerable stress amplitudes in this area, which lead to fatigue cracks. To give a realistic basis for a reliable and economic design of lighting and traffic signal columns, a number of experimental and theoretical investigations have been made. The proposed design concept allows the life of such columns to be assessed with a satisfactory degree of accuracy.

Key words: lighting column; traffic signal column; notch; steel; wind; gust; fatigue; dynamic; damping.

1. Introduction

Throughout the world, several lighting and traffic signal columns have collapsed in the past few years (Gilani and Whittaker 2000, Hamilton *et al*. 2000). In many cases the structures did not fail under extreme-event wind, and the fatigue crack growth was most likely caused by vibrations at lower wind speed excitation. Considerable numbers of these columns, which may vary significantly both in type and size, are required for urban and suburban areas. Fig. 1 shows a typical lighting column. In Germany, the number of new columns or replacements is assumed to run into hundreds of thousands every year. Although the individual lighting column may be a relatively low-priced item, the enormous number represents a considerable annual investment. The ability to assess the life of a lighting column can, therefore, be a highly effective planning tool, and, on the other hand, also meets the safety needs of the general public.

The structural system of lighting columns, a cantilever with one or more concentrated masses (lanterns), is pretty simple. Although most of these columns do not reach excessive heights, they show a complex dynamic behaviour because of their considerable slenderness, coupled with local masses. Additionally, the door opening considerably affects the distribution of the forces in the structure (Fig. 2). In this area, substantial portions of the bending cross section are not available. The torsional moment (resulting from wind acting on mast arm and lantern) produces warping torsion with additional normal stress peaks.

Pagnini and Solari (1998, 1999) illustrated the basic hypotheses and the main steps taken in

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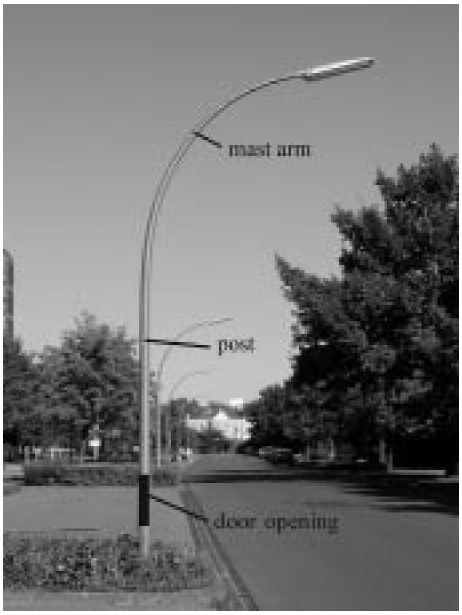
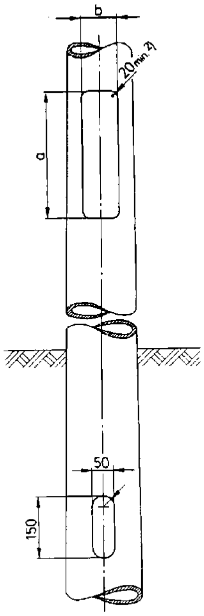


Fig. 1 Typical lighting column



a	*b*
200	75
300	85
400	100
500	100
500	120
600	115
600	130
680 1)	95 1)
680 1)	130 1)
900 1)	130 1)

Fig. 2 Door opening acc. to DIN EN 40 T2

developing the formulation, focusing on the role of aerodynamic terms and mechanical models of steel poles and monotubular towers. However, they only deal with masts having neither a mast arm nor a door opening (cf. Fig. 1). Determination of the dynamic response of the investigated masts is more difficult when considering these parameters. The available literature offers very little and often inadequate details on the design and, in particular, the fatigue behaviour around the door openings. The draft European Standard prEN 40-3-3:1999 for the design and verification of lighting columns contains an (informative) Annex A on fatigue requirements for steel lighting columns. However, it provides only simplified design aids for welded components, and no details on the fatigue behaviour of door openings.

For a safety and cost-effective design, and verification of lighting and traffic signal columns, theoretical and experimental research was conducted dealing with the following main influencing factors :

- Wind action
- Dynamic system response, and
- Structural behaviour and fatigue strength within the field of the door opening.

2. Measurements of wind and dynamic response

To study the effects of the natural wind on lighting columns, a mobile measuring system was installed. Full-scale measurements were conducted at typical urban locations which by approximation correspond to the building structures (urban area, industrial area etc.) classified by Badde and Plate (1994). The turbulence of the wind was measured by three sensors which were fitted on a vehicle-mounted telescopic mast (Fig. 3). This measuring equipment supplies rough details on the wind profile, which is of course affected by the shape of the vehicle, and can also be used to describe

Fig. 3 Wind measuring equipment


Fig. 4 Mast response measuring equipment

correlations between the wind speeds measured at different heights.

The column response was measured simultaneously. Nine columns were instrumented with strain gauges bonded to the outside of the main shaft, 70 cm above the door opening (Fig. 4). Conical steel columns of a circular cross-section, 6.0 to 8.0 m high, were investigated. Each column supported one lantern, mounted on a mast arm with a lateral outstand of 3.0 to 4.5 m. The strain gauges were not to supply local stress peaks around the notch, but the internal forces as a result of wind load. Calculations based on the finite-element method (FEM) can be used to infer from the local strains the stresses that are of relevance for the fatigue behaviour (chapter 6).

3. Wind loading

3.1. Mechanisms of excitation

The fatigue of lighting columns is caused by natural wind. There are different wind-induced excitation mechanisms which should be considered for the design of such structures :

- Vortex shedding,
- Self-excited vibrations like galloping, and
- Buffeting due to gusts.

Vortex-induced vibrations are known to occur in particular for structures with a circular cross-section. However, slender lighting columns have not yet been found to show significant vortex shedding. Wind tunnel tests have proved the restraint moments of conical cylinders to be dramatically reduced, even if their conicity is insignificant (Quadflieg 1975). Apart from that, a standard-height lighting column (< 20 m above ground level according to prEN 40-2:1999), in particular when installed in an urban area, does not seem to be affected by vortex-induced vibrations, because of the roughness of the ground and adjacent buildings produce turbulences which make stationary air flow for prolonged periods of time highly unlikely.

With lighting columns of hexagonal or octagonal cross-section, there is a certain probability of galloping vibrations (Pagnini and Solari 1998, 1999). These vibrations do not occur in connection with the circular structures considered here, as these are aeroelastically stable. When covered with ice, an originally aeroelastically stable cross-section may, however, become unstable. Galloping will only be observed in connection with slightly damped systems, while the damping effect for the column systems considered here is relatively high (chapter 4). For self-excited vibrations under atmospheric conditions, stationary air flow is needed, which has to persist for a minimum of 1 to 2 minutes. Such stationary flow conditions are unlikely in an urban area. This is another factor which much reduces the potential risk of galloping vibrations for lighting columns installed in such environments - independent of the cross-sectional shape.

Results of aerodynamic tests indicated that cantilevered sign and signal support structures are susceptible to large displacement amplitudes resulting from galloping (Hamilton *et al*. 2000). The problem occurs as a result of the aerodynamic characteristics of the attached signs and signals. However, the columns investigated (and the attached lanterns) did not show any significant vibrations caused by galloping.

Vortex shedding and galloping can be controlled by increasing the damping of the structure (Hamilton *et al*. 2000). The response to buffeting is, however, difficult to control. The magnitude of the variable stress ranges in the area of the door opening may be sufficiently large to produce fatigue cracks in the long term.

3.2. Evaluation of wind measurements

An action model as a basis for fatigue investigations has to start from the stochastic wind parameters in the form of correlation functions, power density spectra and coherence functions. The wind measurements were subjected to a detailed analysis for these parameters (Peil and Behrens 2000a). In this context, only wind measurements with stationary or nearly stationary properties can be used. The measurements, however, show that in particular in an urban environment at heights of typical lighting columns, these conditions will hardly ever be met, i.e., the estimated wind parameters contain a number of uncertainties. In the urban canopy layer the description of the complex flow behaviour is very difficult (see Badde and Plate 1994).

Fig. 5 shows measured spectra of the wind turbulence together with the spectrum given in ENV 1991-2-4:1995 :

$$\frac{f \cdot S_{uu}}{\sigma_u^2} = \frac{6.8 \cdot N_x}{(1 + 10.2 \cdot N_x)^{5/3}}, \quad \text{with} \quad N_x = \frac{L_i(z_{equ}) \cdot f}{\bar{u}(z_{equ})} \tag{1}$$

The ENV1991 spectrum contains the integral length scale L_i, the mean wind speed $\bar{u}$ and the equivalent height z_{equ}. For a description of this spectrum, the integral length scale $L_i = 80$ m was assumed to be the same for both locations. In the frequency range above 0.1 Hz, which is of relevance for the dynamic behaviour of these structures, the ENV1991 spectra describe the measured spectra with satisfactory accuracy. For these frequencies, many of the measured power density

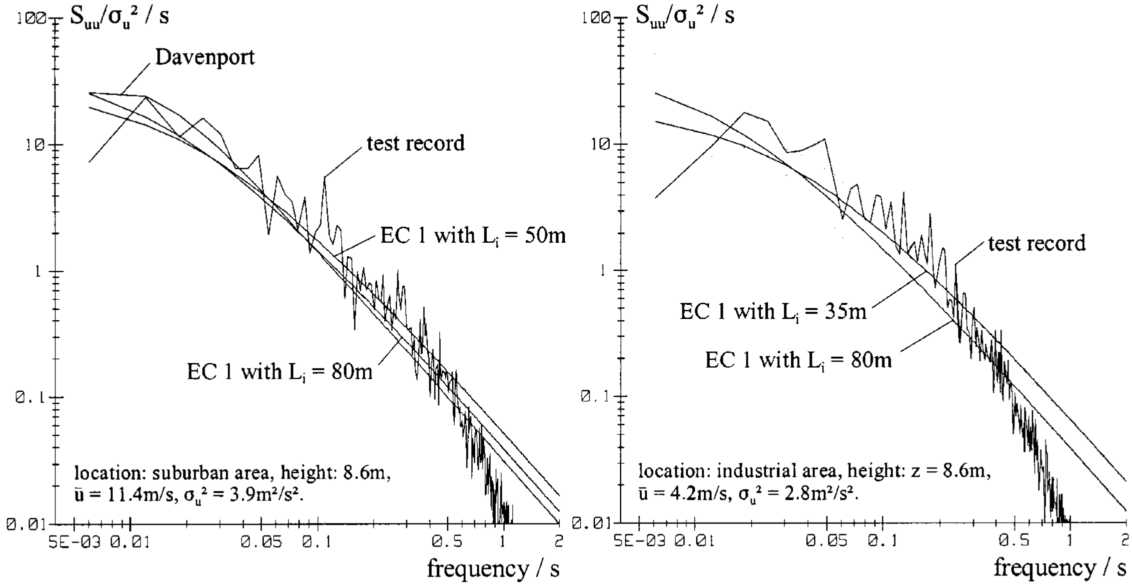


Fig. 5 Measured power density spectra of turbulence

spectra can be approximated fairly well in an analytical approach based on ENV1991-2-4. L_i has been used here as a parameter for approximation of the measured spectra (cf. Fig. 5). The results have to be seen as a practical approach for the frequency range considered. Further investigations concerning the power density spectrum and its behaviour at a lower frequency range can be found in Niemann (1997).

The auto-correlation function of the wind speed measured with the wind monitor in an industrial area at the tip of a column and the cross-correlation functions estimated for the readings of the wind monitor and the two anemometers (cf. Fig. 3) are shown in Fig. 6. Despite the short distances between the measuring points, there is only a slight correlation of the wind speeds at different heights. The maximum of the cross-correlation functions shifts from the centre, which is due to the shape of the eddies which hit higher anemometers earlier than those positioned at lower levels. Measurements made at low levels show that the integral length scales given in ENV 1991-2-4 for urban and industrial areas overestimate the measured values established on the basis of estimated correlation functions. The measurements confirmed, however, the integral length scales L_i in areas of a less rough topography (Table 1).

Table 1 Stochastic wind parameters estimated from full-scale measurements at heights < 10 m

Terrain topography	Integral length scale L_i	Decay factor C of the coherence function Eq. (2)
Suburban areas, industrial areas with structured development and 2–3 floors	50 m to 92 m	15 to 23
Industrial areas with scattered development and 3–4 floors, urban areas (average heights of the buildings < 12 m)	35 m to 60 m	19 to 28

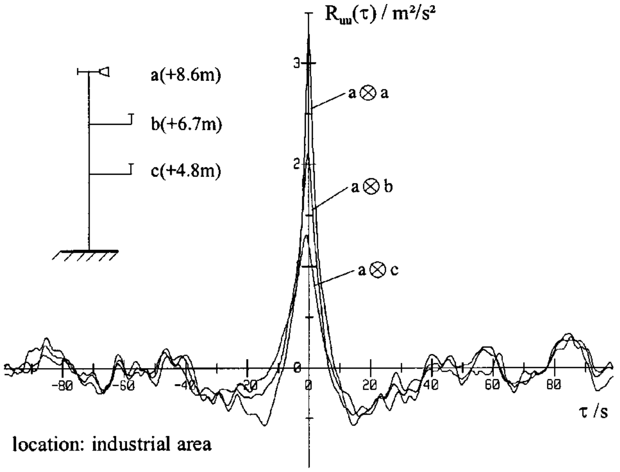


Fig. 6 Correlation function

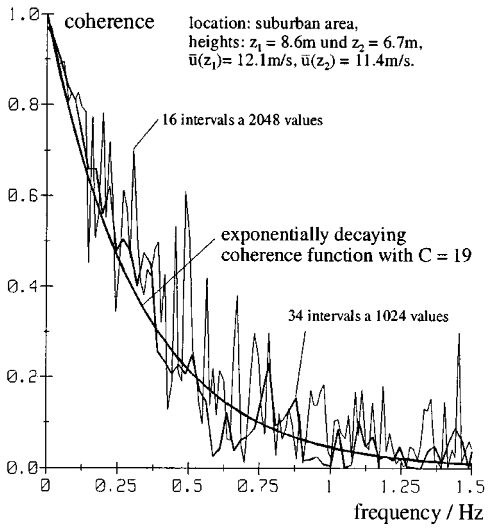


Fig. 7 Coherence function

The coherence function defines the degree of the statistical dependence of two spectral densities versus frequency. The following well-known empirical approach for this function is used here :

$$\gamma_{u_1 u_2}(f) = e^{-\frac{C \cdot f \cdot \Delta z}{(\bar{u}_1 + \bar{u}_2)/2}} \tag{2}$$

see e.g., Telljohann (1998). According to Fig. 7, the samples analysed here reveal that above 0.75 Hz, there is hardly any statistical relationship between wind speed turbulences at immediately adjacent measuring points (Δz = 1.9 m). In areas of rough topography, the eddies within the basic frequency range of the lighting columns (f > 1.0 Hz) do not contribute in any significant way to the correlation between wind speeds at the different measuring locations. The decay function Eq. (2) and the measured record can be approximated fairly well when selecting a decay factor of C = 19. The factors derived from the measurements are summarized in Table 1.

4. Damping

Damping is a largely unknown factor in modelling dynamic system behaviour of lighting columns. It is essential that a realistic value of damping is used in calculations. In connection with lighting columns, four different kinds of damping have to be distinguished :

- Structural damping (energy dissipated in material and joints),
- Damping of the structure interacting with soil,
- Damping due to energy dissipation by cables hanging inside the lighting column, and
- Aerodynamic damping.

Various full-scale experiments have been carried out during various wind speeds to identify the different contributions involved in dissipation. The structure was pulled with a cable positioned at the tip of the mast arm (Fig. 8). The column was then released and allowed to vibrate freely. For each sample structure, the tests were repeated, varying the induced displacement. The lighting

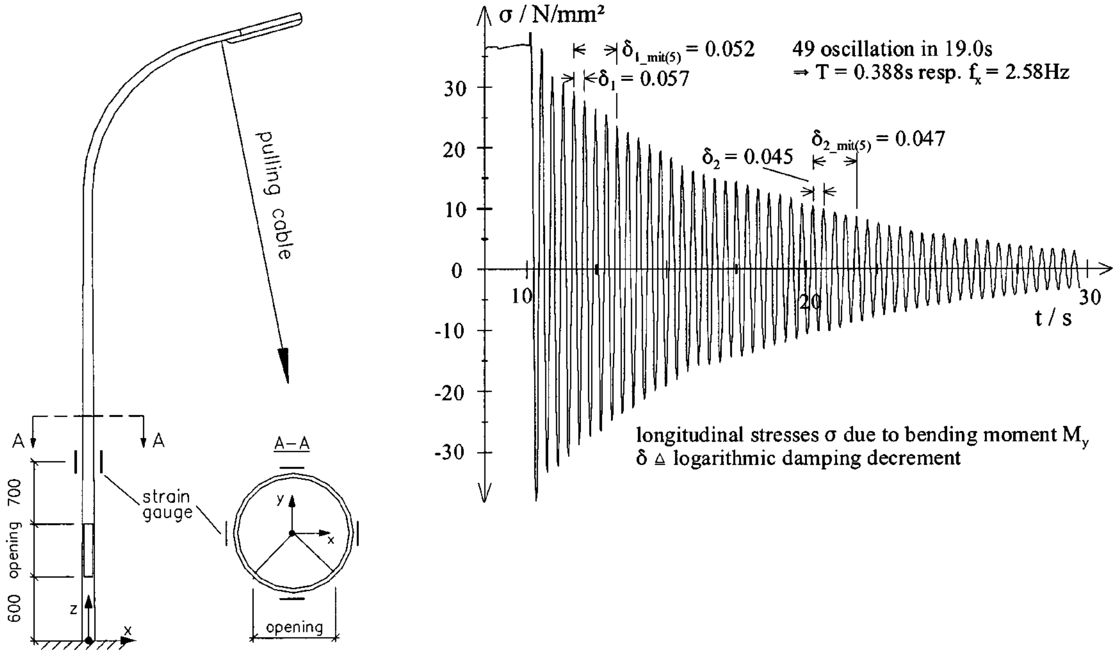


Fig. 8 Forced vibration test-decay of vibration in-plane

columns were excited in plane (with the post and the mast arm) and out of plane. Average values of damping were calculated from the free vibration response data.

Fig. 8 shows the decay of longitudinal stresses due to bending about the strong axis recorded during a period of negligible wind speed. In this case, the mast was excited in-plane. The longitudinal stresses were calculated from the strains measured with the strain gauges bonded to the column 70 cm above the door opening. Once the pulling cable is released, the system approximately vibrates at its basic natural frequency. For determination of the logarithmic damping decrements δ, five consecutive peak values at different amplitudes were subjected to exponential regression.

Table 2 presents the lower limits of the damping values measured in the quasi-calm state. One can see, damping depends on the oscillation amplitude. For the oscillation amplitude, the maximum normal stress at the base is given. In terms of material fatigue, only low and medium wind speeds are of relevance, since such conditions appear much more frequently than situations with very strong winds. Thus fatigue assessments should consider in a dynamic calculation the damping value given for small amplitude oscillations (cf. chapter 8). If the response under maximum wind load is required, an iterative process has to be conducted using the damping values which belong to the amplitudes of this response.

Table 2 Logarithmic damping decrement δ

Oscillation amplitude	Small	Medium	Large
Max. normal stress at the base	40 N/mm^2	80 N/mm^2	> 80 N/mm^2
Material damping, soil damping, “cable” damping, other parts of damping	0.015	0.030	0.050

For assessing the effects of the wind-speed related aerodynamic damping, the vibration tests were repeated at different wind speeds. As compared with the tests conducted in the quasi-calm state, the logarithmic damping decrement δ was found to increase by 0.018 to 0.034 at wind speeds of approx. 10 m/s (1 min-mean). This increase corresponds by approximation with the portion of aerodynamic damping δ_a. For the columns examined, this damping portion is relatively small, which is due to the fact that the area exposed to the wind when dealing with lighting columns is normally limited and that the vibration tests were conducted at fairly low wind speeds as compared to the design wind speed.

The test results shown in Table 2 correlate with the experiments described by Pagnini and Solari (1999), where reference is also made to a way of how aerodynamic damping can be considered in calculations. ENV 1991-2-4 provides a logarithmic mechanical damping decrement of $\delta_{mech} = 0.015$ to be used for slender welded steel structures. In all the cases examined here, the actual damping decrement is significantly higher.

5. Dynamic system response due to wind

Structures with separated and low eigenfrequencies show a narrow-band response even under a broad-band excitation. Fig. 9 presents measured response data for one of the lighting columns investigated. The time histories show the wind speed at the tip of the mast arm and the simultaneously measured bending moment about the x-axis (cf. Fig. 8).

Wind direction out of plane excites the natural modes of vibration with the frequencies $f_y = 2.50$ Hz (out of plane, combined torsional and bending vibration) and $f_x = 3.05$ Hz (in plane). In either of the planes considered, the column vibrations correspond to the basic natural modes which approximately vibrate in this plane. After each stress amplitude, the mean value is crossed. In such a narrow-band processes, the distribution of peak values is well described by a Rayleigh distribution (see Fig. 10).

Fig. 11 shows the power spectra of the column response presented in Fig. 9. In this case, frequencies above 5 Hz are of no relevance for the design and are hence not shown. Since the excited natural mode of vibration produces bending moments M_x as well as M_y in the column's post, the response spectrum shows distinct amplitude peaks with the associated frequencies $f_y = 2.50$ and $f_x = 3.05$ Hz (Peil and Behrens 2000a). The dynamic amplification due to resonance remains relatively small, which can explained by the high natural eigenfrequencies and the high overall damping values. In this case, the overall logarithmic damping decrement δ amounts to 0.105 (including aerodynamic damping).

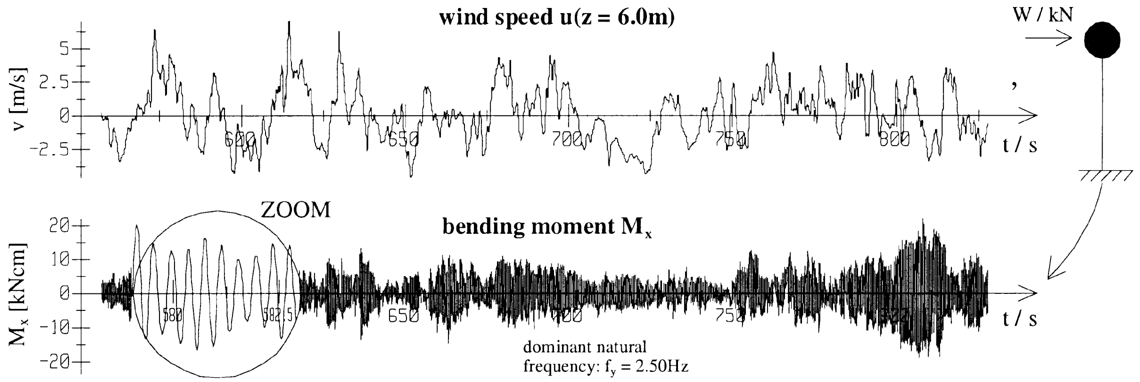


Fig. 9 Time histories of wind speed and column response (mean-value- and trend-adjusted)

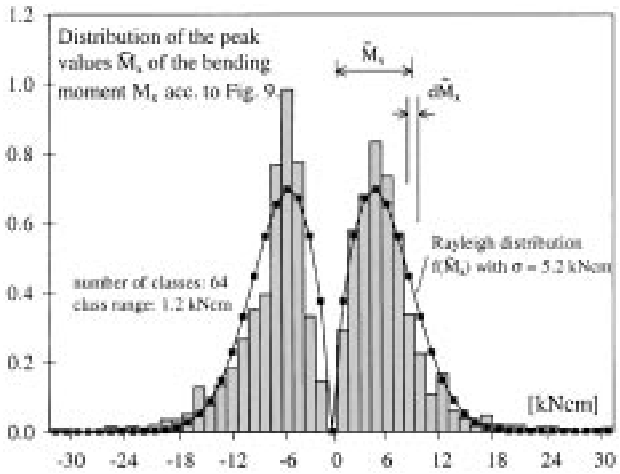


Fig. 10 Response amplitude distribution

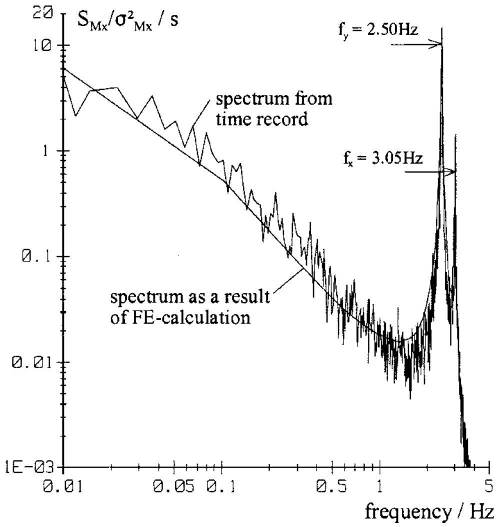


Fig. 11 Response power density spectrum

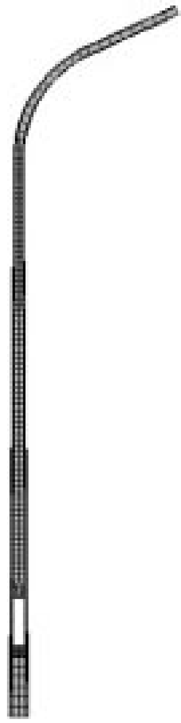


Fig. 12 FE model for dynamic calculation

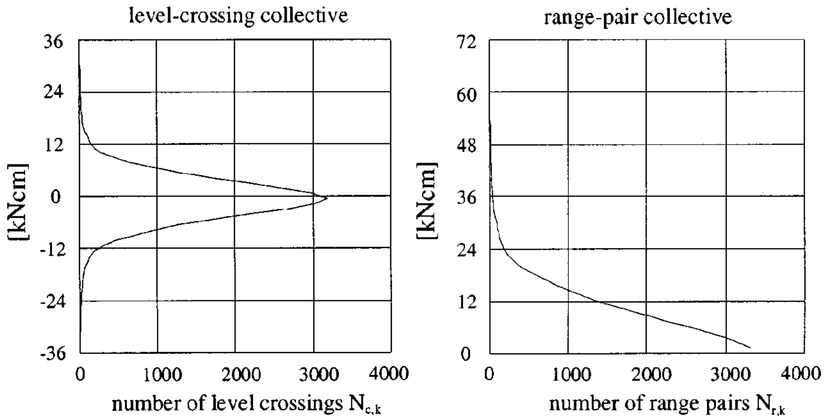


Fig. 13 Level crossing and range pair collective

The dynamic calculations were made on the basis of the random vibration theory. The statistic parameters of the excitation process are known (cf. chapter 3). The column geometries were modelled by means of shell elements. Fig. 12 shows the FE model of one of the examined columns. Damping was considered in the calculation on the basis of the vibration tests made (see chapter 4). Seven stochastic excitation forces were applied to represent the wind action along the column height. The cross-spectra of the wind turbulence were considered on the basis of the auto-spectra and the coherence function, which were determined starting from the wind measurements conducted. It should be noted that for the high natural frequencies of the columns analysed here, the correlation of the wind load is very small.

A comparison of measured and calculated column response shows that the agreement between measured and calculated natural frequencies and resonance amplitudes is very good (Fig. 11).

For fatigue assessment, the number N and the magnitude of stress amplitudes $\Delta\sigma$ must be known. From the measured time histories of stresses due to bending moment M_x (Fig. 9) the level-crossing collective and the range-pair collective are determined (Fig. 13). The collective obtained from the range-pair count represents the basis of the fatigue assessment.

6. Structural behaviour within the area of the door opening

Using the FE-method, Peil and Behrens (2000a) describe in detail the structural behaviour within the area of the opening under different load conditions. Only the part of the post including the door opening was considered in the elastic FE analysis. With the investigated lighting columns, which incorporate non-reinforced door openings, the most critical stress concentrations occur at the end of the longitudinal edges of the opening above and below the rounded corners (Fig. 14, Point A). Fig. 14 uses magnification to show the deformation behaviour of a typical column section subjected to various loads. The FE calculations were verified by strain-gauge measurements for the area of the opening. There was very good agreement between the results produced by FE calculations and the strains measured (Peil and Behrens 2000a).

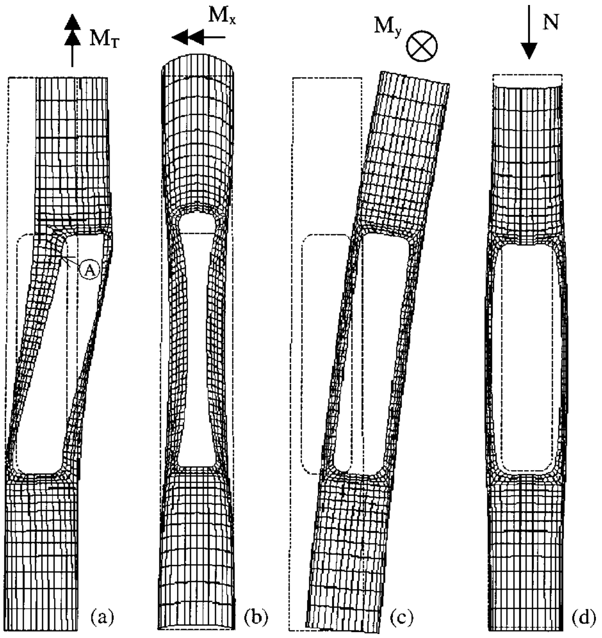


Fig. 14 Displacement plots due to torsional moment (a), bending moment M_x (b), bending moment M_y (c), axial force (d)

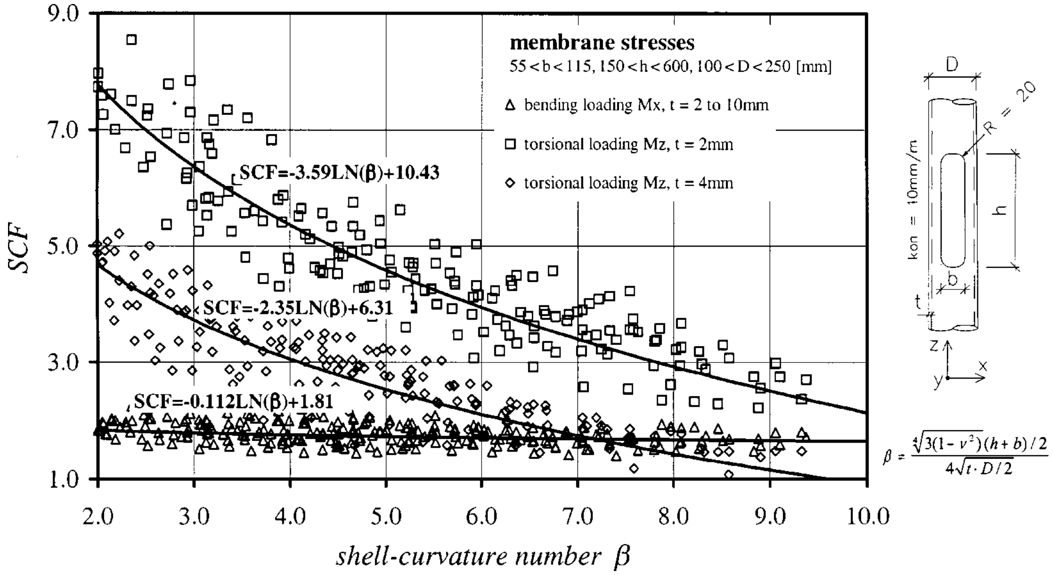


Fig. 15 SCFs for torsional and bending loading

To provide a basis for simple calculation methods, i.e., without making use of the FE method, the stress concentration factor (SCF) was determined for a large number of column and opening geometries. Fig. 15, for example, shows the trend curves of the SCFs for the torsional moment M_z and the bending moment M_x. The geometry of the columns was related to the shell-curvature number β. The SCFs for bending stresses are smaller than the factor for the torsional stresses. On the basis of these factors the maximum stresses at the point at which cracking is expected to occur

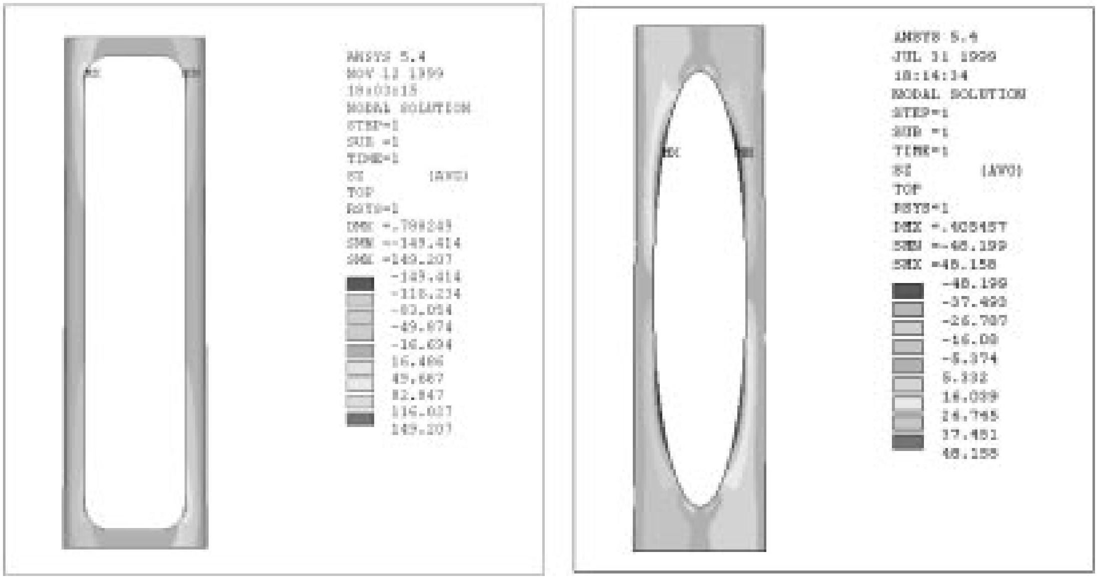


Fig. 16 Longitudinal stresses in [N/mm^2] due to unit torsional moment within the field of a rectangular and elliptical opening of a typical lighting column

can be determined from the geometry of the structure and the nature of the stress. The SCFs allow the local stress peaks resulting from the geometry of the opening to be adequately considered, not, however, the notch effects due to flaws or welds within the area of the opening. The SCFs have to be multiplied by the nominal stresses, which are defined as stresses due to the beam theory with a cross-section weakened by the opening (net cross-section). The stresses are not related to the solid cross-section, because the torsional SCF of a hollow section is infinite.

The geometry of the notch is one of the most dominant factors influencing fatigue life. To investigate the effect of the geometry of the door openings, parameter studies were performed to examine the effect of the corner radius, as well as the height and width of the opening. When consideration being given to production factors and accessibility, the shape of the opening was modified to arrive at favourable design with respect to the notch effect. An elliptical opening was found to be the ideal solution. Fig. 16 shows the longitudinal stresses due to torsion around two different opening shapes. Under torsional stress conditions, the notch effect of the column investigated for the elliptical opening is found to be reduced by a factor of about 3 ($\approx \sigma_{rect.} / \sigma_{ell.} = 149.2 / 48.2$) as compared with the conventionally used rectangular opening having only slightly rounded corners.

7. Fatigue behaviour within the field of the opening

The failure of a lighting column due to wind-induced fatigue is associated with a large number of

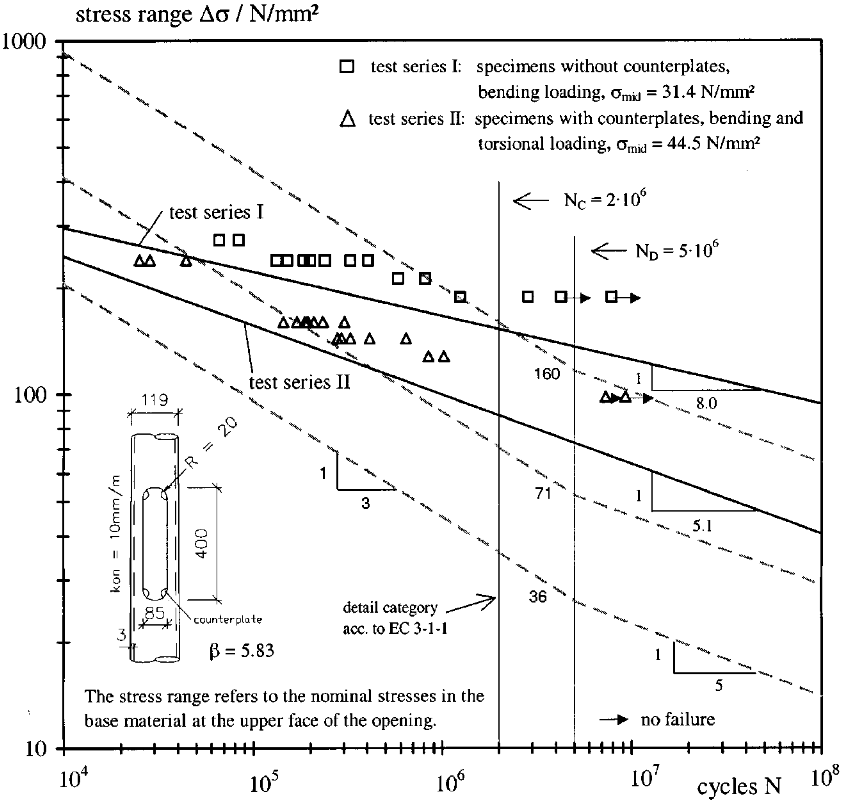


Fig. 17 S-N curve of a circular tube with an opening

loading cycles at strain levels less than yield strain. Although such columns are typically subjected to complex loads, constant-amplitude sinusoidal loading is usually used to characterise the fatigue life of components and connections. The fatigue life of a specimen at different stress ranges can be represented by an S-N curve (where S is the stress range which is equal to twice the stress amplitude in a given cycle, and N the number of cycles leading to failure). 59 fatigue tests were conducted on specimens showing the typical column and opening dimensions to determine the fatigue strength of lighting columns. Fig. 17 shows the fatigue strength curves for two series of tests made for columns of the same opening geometry, but with different local edge design. One type of the specimen has rounded corners, the other type was provided with additional dot-welded plates to keep the door in position.

According to ENV 1993-1-1:1992, the curves shown correspond to a 95% survival probability for $\log N$ with a confidence interval of 95%. The stress ranges refer to the nominal stresses in the base material in the immediate vicinity of the point where the material is expected to crack. As expected, the tests demonstrate that the fatigue strength for columns provided with a counterplate is less favourable than those for columns not fitted with such plates. For a detailed description of the tests and the test evaluation, see (Peil and Behrens 2000a).

The determination of the fatigue limit needs a very long testing time. These tests could not be performed. For the purpose of the fatigue assessment the determined finite life curve with the slope m could be extrapolated to the cut-off limit at $N_L = 10^8$ cycles. This assumption is very much on the safe side. Another possibility, following the rules of ENV 1993-1-1:1992, is to define the fatigue limit at $N_D = 5 \cong 10^6$ cycles. The fatigue curve from the fatigue limit to the cut-off limit can be assessed using a slope of $(2 \cdot m - 1)$.

8. Fatigue assessment

For fatigue assessment due to gusty wind, the distribution of the mean wind speeds has to be known, which prevails at the location where the considered lighting column is installed. Details on the wind distribution may be obtained either from meteorological institutions or from the European Wind Atlas (1990), which gives the frequency distribution of the wind speed in the form of a Weibull distribution. The adaptation of the recorded data for high wind speeds to the mathematical distribution is not significant when thinking in terms of material fatigue. The contribution of a small number of high wind speeds to the damage accumulation is by far smaller than that of the enormous number of small and medium wind speeds. For wind speeds that are of relevance for fatigue problems, the Weibull distribution should be approximated by considering at least 6 classes of wind speeds.

The mean system response to the different wind speeds follows from a structural static analysis. The rms of the response can be determined either on the basis of the random vibration theory, or by approximation, using the methods described in the relevant literature (e.g., Niemann 1990 or ENV 1991-2-4). The methods described in the literature generally refer to a simple cylindrical cantilever structure (without mast arm) in which the mass and stiffness distribution is constant. When used for fatigue assessment, the deviations resulting from these methods, however, can be expected to remain within acceptable limits. Fig. 18 shows the flow chart of the proposed fatigue assessment procedure.

A narrow-band process can be used to adequately describe the dynamic response of lighting columns. The peak value distribution of such a process reflects the Rayleigh distribution (Fig. 10). Assuming that the narrow-band process produces after every maximum a minimum of response, the number of sign changes is directly connected with the basic natural frequency f_e. This gives us the

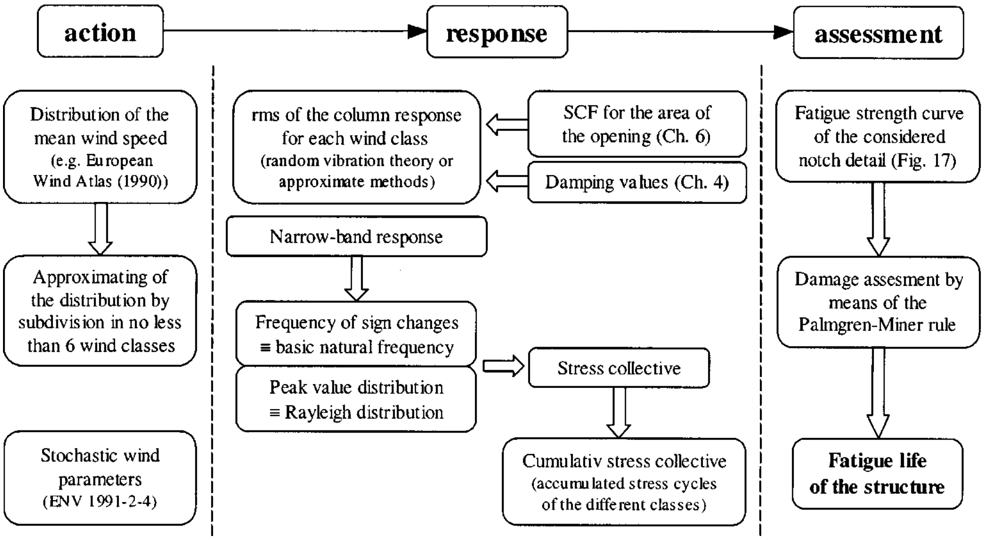


Fig. 18 Flow chart of the proposed fatigue assessment procedure

total number of stress cycles, and for each class of the considered response process, the number of stress cycles can be determined (Peil 1994); the stress collective is thus known. When summing up the stress cycles of the different classes, a cumulative stress collective is generated. Based on this stress collective and knowledge of the fatigue strength curve (Fig. 17) for the considered notch detail, the damage state of the life cycle of the structure can be determined by means of the Palmgren-Miner damage accumulation rule.

Fig. 19, for example, shows such cumulative stress collectives for two columns 8.0 and 20.0 m high, which differ in their basic natural frequencies. In the case considered here, the collective curves for columns with the low basic frequencies remain below the curves representing the more rigid columns with higher natural frequencies, which consequently are less likely to start oscillating in gusty winds. This is due to the fact that the system response is related to the maximum response that can be expected to occur once every 50 years, and that the number of stress range cycles for the columns with higher natural frequencies is higher by a factor of 7.

The dynamic column reaction is known to be decisively influenced by wind turbulences. At a height of 20.0 m, turbulences are clearly less marked than at a height of 8.0 m at the same location. Since the effects of actions in Fig. 19 are related to maximum loads, the collective curves for the 20.0 m columns remain despite the higher wind speeds below those for the 8.0 m column with the same natural frequency.

The local stresses at the notches depend very much on the wind direction. Thus wind-direction-related investigations should be performed in general. This must, of course, include the exact position of the lighting column in the street, which may change very much as a result of street direction. On the other hand, the numerical effort increases enormously if the wind direction is taken into account. Thus, a simple way of approximating the effect of wind direction is proposed: the wind direction field is split up into two main directions (±45° around each direction of flow at right angles with the cantilever). The stresses around the notch are determined as a result of wind action at right angles with the cantilever. This wind direction generally produces the highest stresses

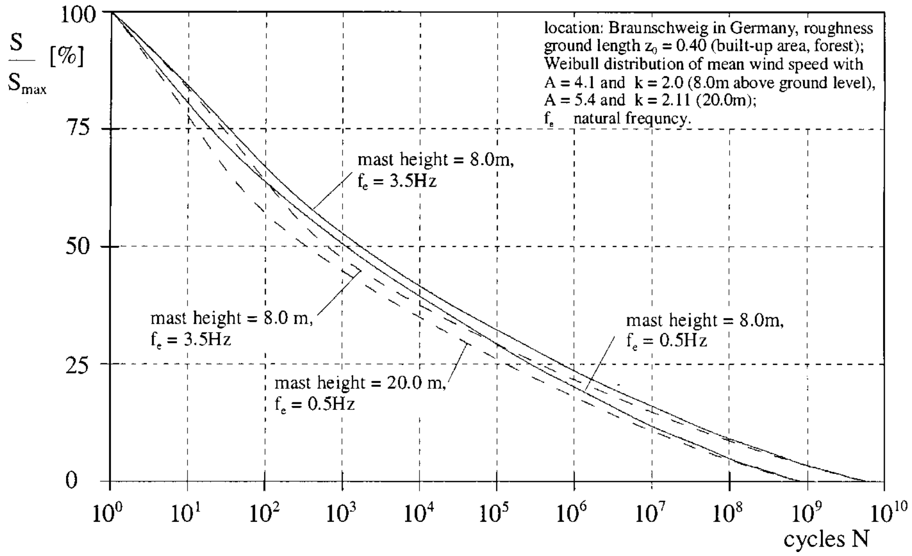


Fig. 19 Collectives of a response *S* due to gusty wind during a 50-year return period

in lighting columns. As mentioned above, only low and medium wind speeds are generally of relevance in terms of material fatigue. These wind speeds depend very little on the wind direction. Using this assumption, the number of stress cycles for one main direction can be approximated by dividing the total of the whole wind process by the number 2.

Starting from the load collectives in Fig. 19, calculation of the fatigue life was based on the assumption that the largest amplitude occurring within a period of 50 years will correspond to a nominal stress of 165 N/mm^2 ($\approx$70% yield strength steel S 235 JG). The uncertainties with respect to fatigue strengths and wind effects were considered in the fatigue life calculation in the form of partial safety factors (Peil and Behrens 2000a). Taking the fatigue strength curves in Fig. 17 as a basis, one arrives at a theoretical fatigue life of 10 years for the 8.0 m column with 0.5 Hz natural frequency and an opening provided with counterplates. For a column without any additional plates, this is more than 100 years. Starting from the same expected maximum load, the fatigue life of an 8.0 m column with a basic frequency of 3.5 Hz will be 4 years (incl. counterplates for the opening) and 71 years (without counterplates). It should be noted here that in view of the higher wind-gust-excited dynamic amplification, the effects of actions have to be expected to be by far greater for the column with a natural frequency of 0.5 Hz than for the column with the higher natural frequency, the structural design of the opening being identical in both cases.

9. Conclusions

The behaviour of slender steel lighting columns is investigated. Evaluations of wind measurements are compared with the ENV 1991-2-4. In addition, measurements of damping decrement values are performed under different conditions. The structural behaviour is studied using the FE-method. Fatigue tests were conducted on specimens showing the typical column and opening dimensions to determine the fatigue strength.

A concept has been developed for fatigue assessment of lighting and traffic signal columns (Fig.

18). This concept can be used to determine the cumulative stress collectives required for a simple design approach (Peil and Behrens 2000a). The design proposal is a reliable and efficient method, allowing the fatigue life of such columns to be assessed with satisfactory accuracy.

For long life cycles, it is recommended to use rounded corners with large radiuses to reduce the peak stresses in the area of the door opening. Respecting accessibility, an elliptical opening was found to be ideal. One of the results produced in the fatigue tests is that counterplates dot-welded in the rounded corners of an opening to support the door covers will much reduce the fatigue strength of a column.

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