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Field measurements of wind-induced transmission tower foundation loads

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Abstract. This paper discusses some of the findings arising from long-term monitoring of the wind effects on a transmission tower located on an exposed site in South-West England. Site wind speeds have been measured, together with the foundation loads at the base of each of the four legs. The results show good correlation between the wind speeds and leg strains (loads) for a given wind direction, as expected, for wind speeds in excess of 10 m/s. Comparisons between the measured strains and those determined from the UK Code of Practice for lattice towers (BS8100), for the same wind speed and direction, show that the Code over-estimates most of the measured foundation loads by a moderate amount of about 14% at the higher wind speeds. This tends to confirm the validity of the Code for assessing design foundation loads. A finite element analysis model has been used to examine the dynamic behaviour of the tower and conductor system. This shows that, in the absence of the conductors, the tower alone has similar natural frequencies of approximately 2.2 Hz in the both the first (transversal) and second (longitudinal) modes, whilst for the complete system the conductor oscillations dominate, giving similar frequencies of approximately 0.1 Hz for both the first and second modes.

Key words: transmission tower; wind loading; meteorology; structural analysis.

1. Introduction

For prolonging the working life of existing electricity transmission towers, as well as enhancing the economic viability of new structures, it is important that improved assessments of the imposed loads are obtained. In particular, a better understanding of the wind-induced loads at the foundations of the structure is required. The present work is part of a project in which the wind-induced loads in the legs of transmission towers in the UK are being monitored. The towers chosen for the research are located in areas of extreme winds. The first tower to be monitored, and the subject of the present paper, is located at Winterbourne Abbas near Dorchester, Dorset, in South-West England. The structure is in a high, exposed location approximately 8 km from the coast. Measurement of field data has allowed comparisons with results from the UK Code of Practice for lattice towers, British Standards (1986a, b). The dynamic behaviour of the tower and conductor system has been investigated using a finite element model and the results are also reported in this paper.

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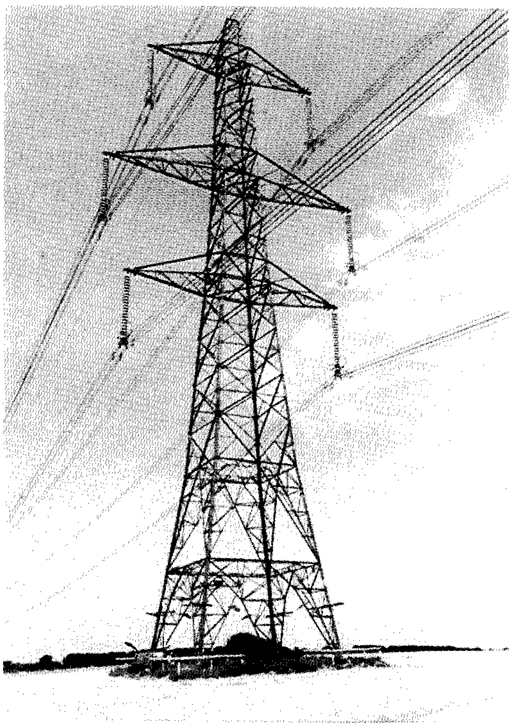

Fig. 1 View of the monitored tower looking from the South-East

2. Details of test tower and instrumentation

The tower under test is illustrated in Fig. 1, which shows the view of the site looking from the South-East, whilst the location is shown in Fig. 2. The altitude of the topography is 186 m above mean sea level and the structure is a 44 m tall CEGB type Blaw Knox L6 standard height tower, Lomas (1993), with a square base occupying an area of 9.1×9.1 m at the foundations. The mean height of the conductors above ground level is 30 m with an effective span between adjacent towers of 341 m. The four conductors in each bundle, as well as the earth wire, are 400 mm^2 section ACSR cables. The wind speed is monitored by a Vector Instrument A101ML high-resolution cup anemometer located at the top of the tower, with the wind direction being determined from a type W200P Vector Wind Vane. A Vector Instrument 107 thermistor probe is used to monitor the air temperature at a level approximately 2 m above the base.

The main interest with regards to the loading on the tower concerns the forces induced in the four legs at locations immediately above the foundations. This is because, under current practice, the foundations and supports in this region are designed using conservative estimates of the loads. The strains at the base of each leg are monitored using Measurements Group CEA W250A 120 ohm strain gauges. All of the strain gauge bridge networks, including precision resistors, are mounted on steel shims which are spot welded to the structure. Care was taken to ensure a similar temperature environment for all the components and the gauges were temperature compensated for steel. Weather protection for the devices is achieved by encapsulation using a layer of Teflon followed by a 5 mm layer of butyl rubber sealant and then a 5 mm layer of neoprene rubber for mechanical protection. Finally, a layer of aluminium

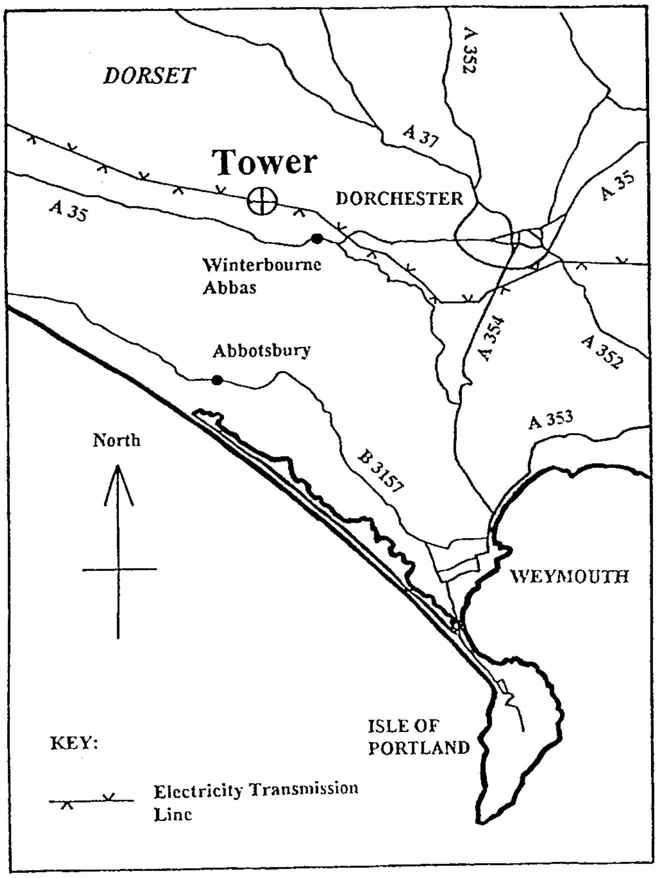

Fig. 2 Location of site in Dorset in the South of England

foil tape is used which is then coated with silicon sealant.

Simultaneous measurements of the wind speed and direction, the air temperature and the outputs from the gauges are taken using a Campbell Scientific CR10 twenty channel programmable data logger. All of the sensors are scanned at 10 minute intervals in order to develop a loading time-history for the tower. In addition, some shorter time histories have been obtained in which more rapid acquisition rates, from 3 to 10 secs per sample set have been used. The information gathered from this procedure is collected weekly over a Paknet data link to the University. The nature and extent of the instrumentation on the tower is limited by the need for the logging and remote data transmission equipment to be solar powered. The strain data have been processed by examining the time records and determining the zero readings from the occasions where the wind speed is zero. These zero readings have then been curve-fitted, with the resulting zero values being subtracted from the corresponding data points in the time history.

3. Results and discussion

A summary of some of the data obtained since the June 1995 start is given here, covering the basic meteorological data for the site, followed by an analysis of the leg loads on the tower.

3.1. Meteorological and foundation load data

The yearly wind rose for the site from July 1995 to February 1997, shown in Fig. 3, highlights the predominance of South-Westerly and North-Easterly winds in this region, as

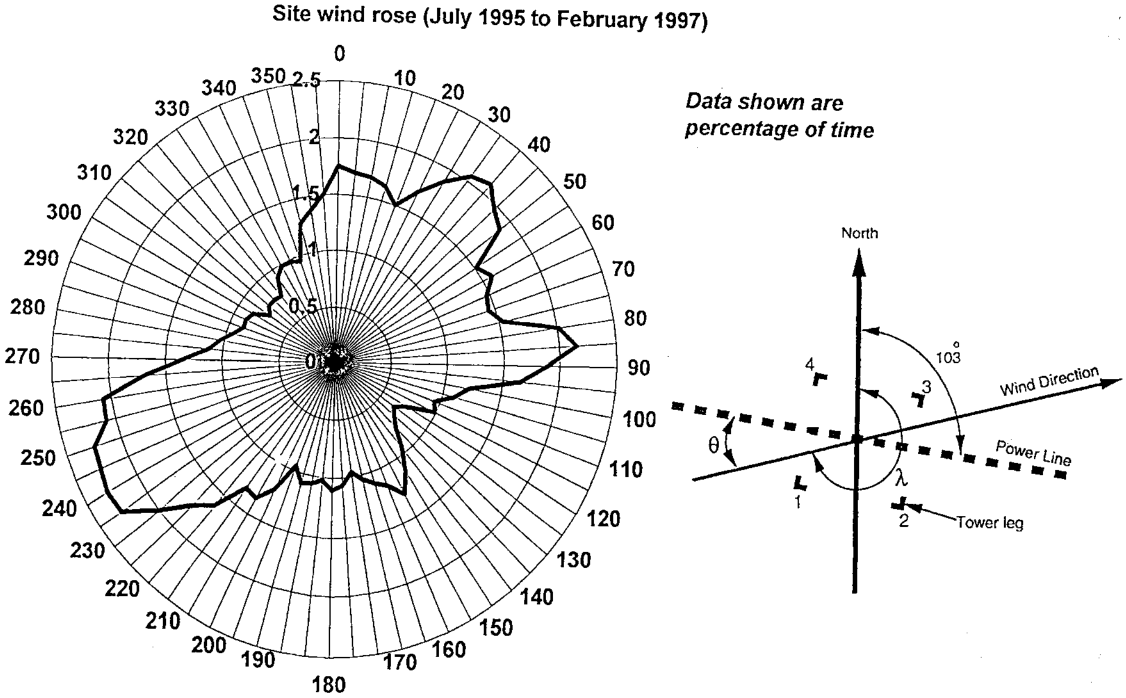

Fig. 3 Site wind rose (direction divided into 5 degree sectors) and tower orientation

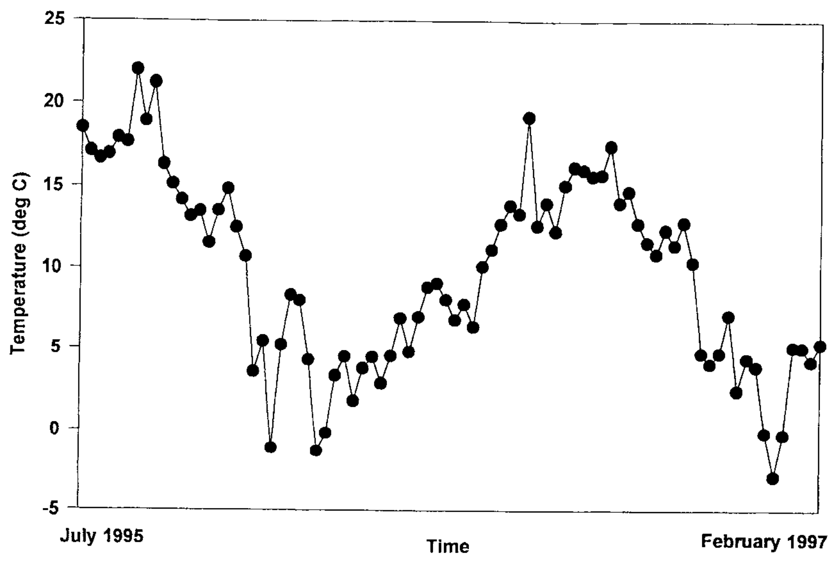

Fig. 4 Variation of weekly average site temperature with time

expected. Some instrumentation problems, particularly in the early stages of monitoring, have meant that the data record is not entirely continuous over the period to June 1998. Nevertheless, the yearly wind rose indicates that the data set obtained is very representative. The accompanying local air temperature plot is given in Fig. 4 as an approximate weekly average, whilst the annual variations of average and peak recorded wind speeds, computed on the same basis are shown in Fig. 5. It may be seen that there is generally a small increase in the mean and peak wind speeds in the winter months, although this effect is not pronounced due

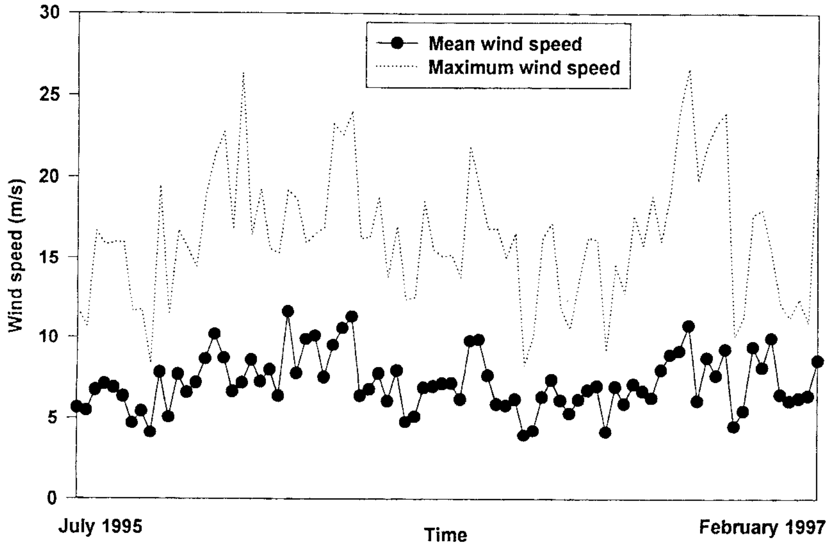

Fig. 5 Variation of weekly mean and maximum wind speed

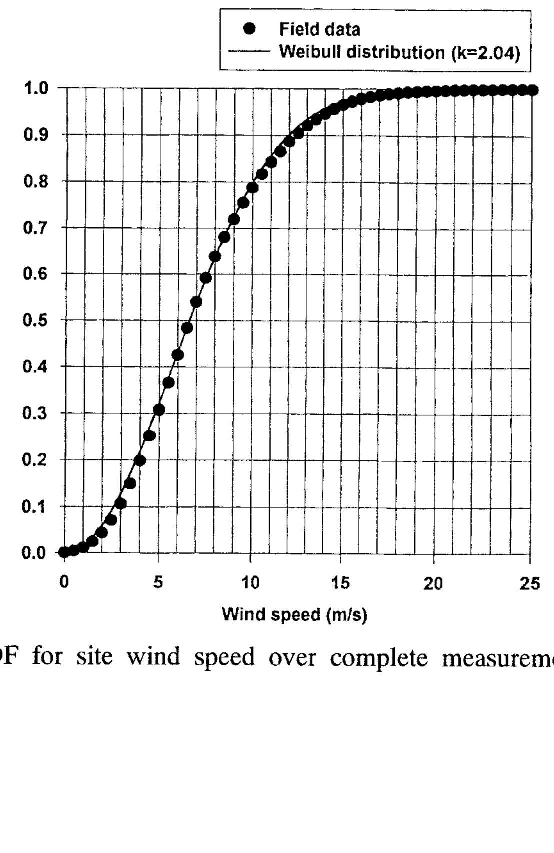

Fig. 6 CDF for site wind speed over complete measurement period

to the exposed nature of the site and its proximity to the coast. The cumulative distribution function (CDF) for the wind speed over the measurement period is given in Fig. 6 and this shows good agreement with the Weibull distribution with a "*k*" factor of 2.04. This is a reasonable value since, as noted by Cook (1985), UK parent wind speed distributions, irrespective of direction, are

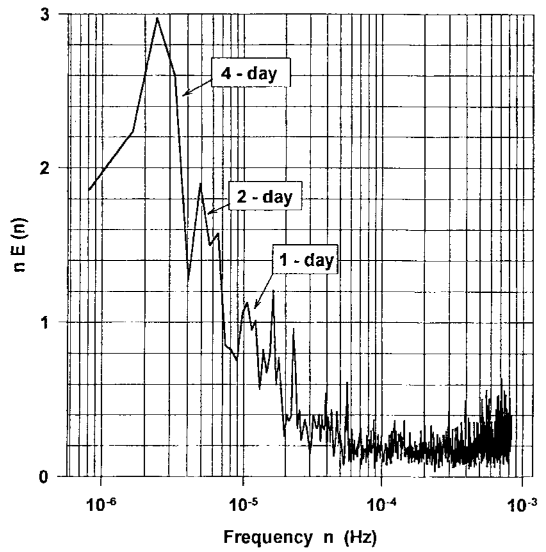

Fig. 7 Site wind speed spectrum over a 3 month period

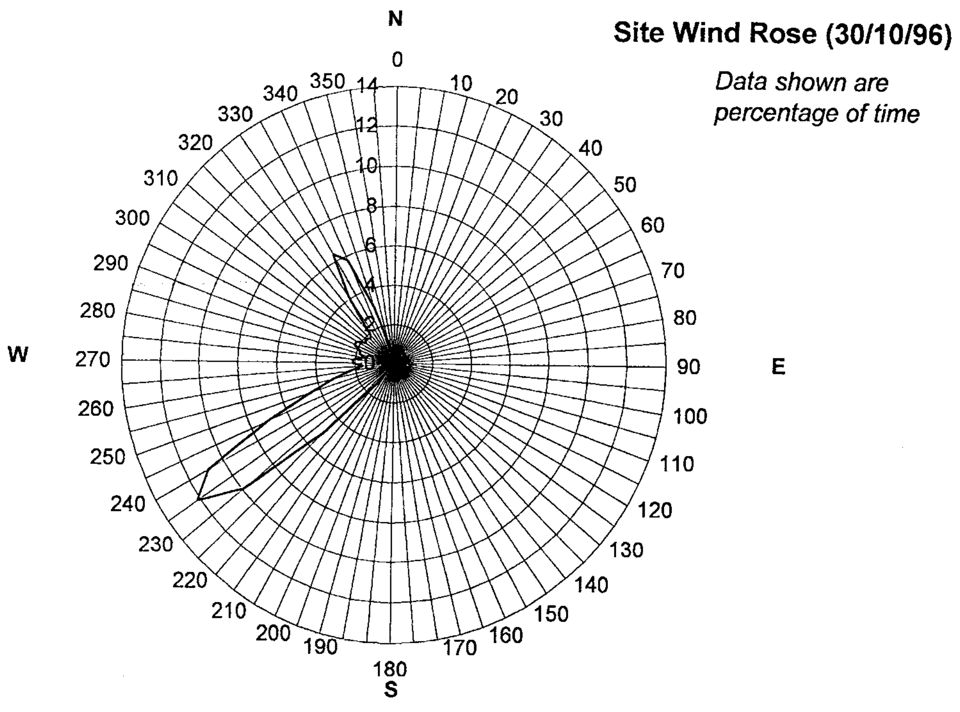

Fig. 8 Wind rose for measurements taken on 30th October 1996

well-represented by Weibull distributions with k in the range from 1.7 to 2.5.

Although the breaks in the data record have prevented a complete spectral analysis of the low-frequency aspects of the site wind characteristics, processing of the results over shorter time periods has produced some very useful information. For example, the spectrum shown in Fig. 7 for the three months from December 1995 to March 1996, shows broad agreement with the shapes of the standard Van Der Hoven (1957) and Gomes and Vickery (1977) gust

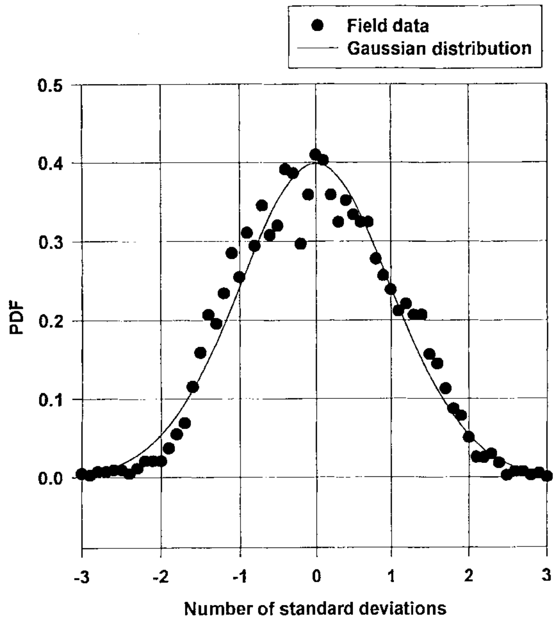

Fig. 9 PDF for wind speed data taken on 30th October 1996 (South-Westerly wind)

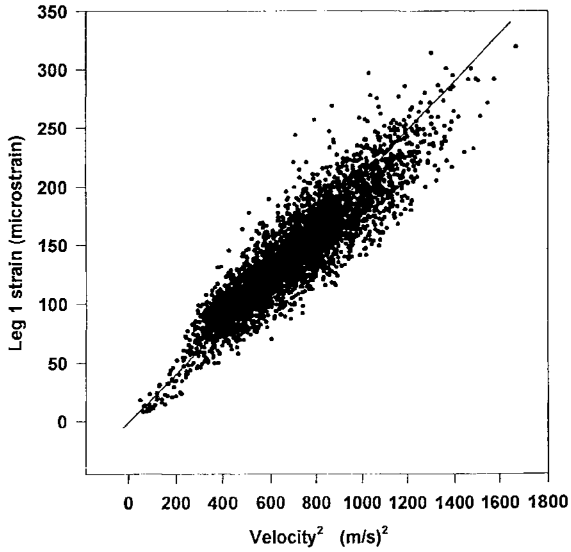

Fig. 10 Variation of Leg 1 strain with square of wind speed (30th October 1996)

spectra for that return period. The 1-day and 2-day responses are evident, with the 4-day peak also being very clearly defined.

Measurements taken during a storm on 30th October 1996, in which the wind speeds and leg strains were sampled at 10 second intervals, have also provided some useful data. During the first 12 hours the wind direction was South-Westerly with an average speed of 26 m/s, whilst for the next 6.5 hours the wind abruptly swung around to North-Westerly direction and the average speed dropped to 14 m/s. The wind rose for this storm is given in Fig. 8. The probability density function(PDF) for the first part of the storm is given in Fig. 9 and this shows a reasonably good Gaussian profile. The variation of strain in Leg 1 (being the most

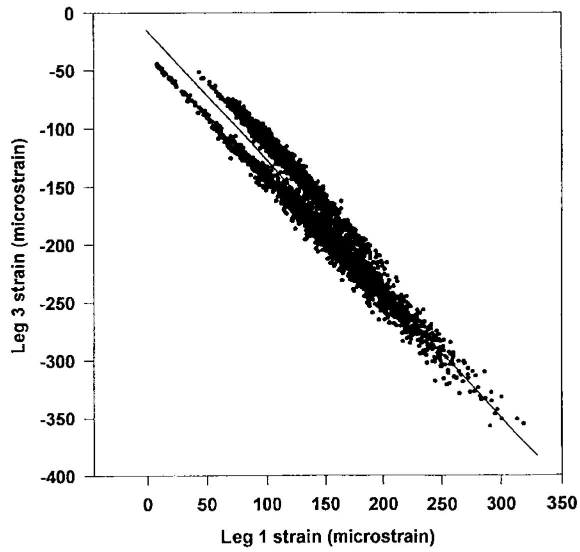

Fig. 11 Correlation between Leg 1 and Leg 3 strain (30th October 1996 South-Westerly wind)

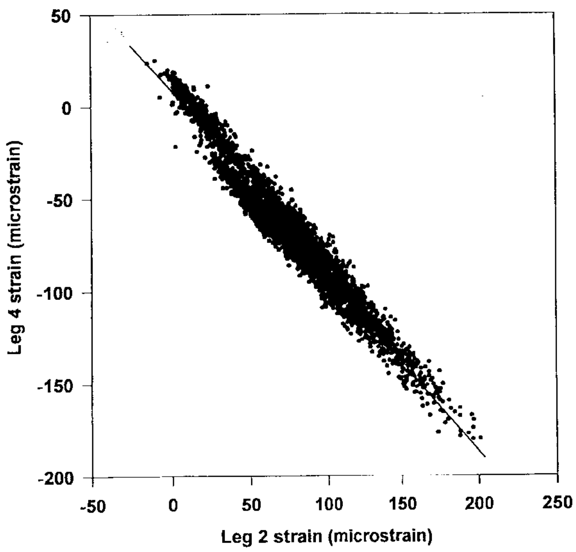

Fig. 12 Correlation between Leg 2 and Leg 4 strain (30th October 1996 South-Westerly wind)

windward leg of the tower) with (*reference wind speed*)^2, given in Fig. 10 shows a good correlation, particularly when it is considered that these are instantaneously sampled data points rather than averaged mean or r.m.s. measurements. It should be noted that, from some recent field measurements on a tall transmission tower in mountainous terrain, a good correlation was achieved between member r.m.s. strains and (*reference wind speed*)$^{1.6}$, Momomura *et al.* (1997). From the present results, there is also quite a good correlation between the loads in Legs 1 and 3 (Fig. 11)

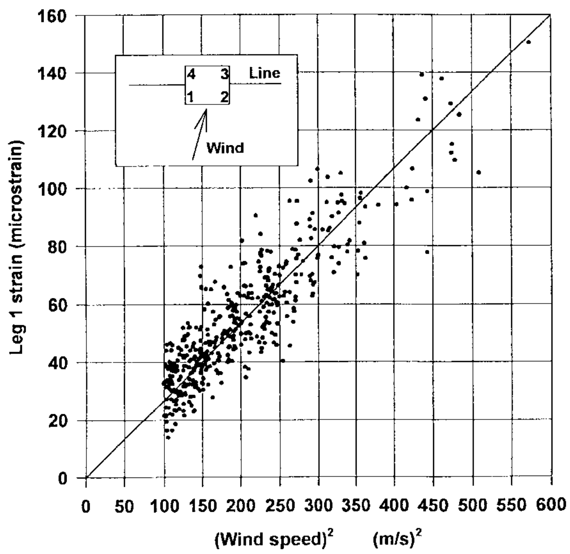

Fig. 13 Variation of Leg 1 strain with (wind speed)^2 for Southerly winds relative to the transmission line (incidence angle of 200°) from July 1995 to February 1997

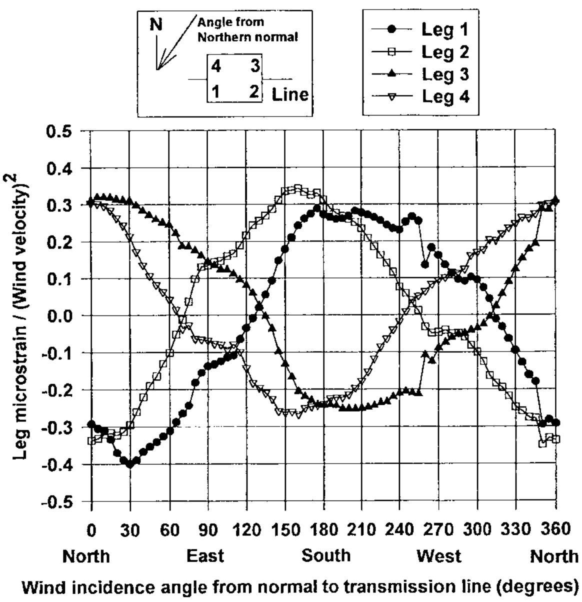

Fig. 14 Variation of leg strain/(wind speed)^2 with wind incidence angle measured from the normal to the transmission line

and Legs 2 and 4 (Fig. 12). As the wind changes to North-Westerly the PDF remains essentially Gaussian but the reduction in wind speed means that the correlation between applied wind load (square of the wind speed) and the structural response is rather poor. Indeed, it has been determined that wind speeds of at least 10 m/s are required in order to produce reliable data.

Hence, from consideration of the variation of the loads in each of the four legs with the square of the wind speed (only for wind speeds above 10 m/s), a series of charts for each wind direction was obtained, with a typical case for Leg 1 illustrated in Fig. 13. These charts include all the data measured over the complete monitoring period and indicate that, for a given wind speed and direction (within a 5 degree band), the leg loads can be determined to within ± 30 microstrain. Bringing all the results together, Fig. 14 shows the variation in *strain*/(*wind speed*)^2 for the four legs with variation of wind direction from the "Northern" normal to the transmission line. The plots show a good degree of consistency and may be used to provide a sensible indication of the wind-induced load at the base of any given leg for a given wind speed and direction.

3.2. Comparison between measured foundation loads and Code of Practice calculations

A comparison has been carried out between the measured foundation loads and those determined from calculations using the appropriate UK Code of Practice, British Standards (1986a,b). In the Code calculations the Terrain Category was taken as type II, with the terrain roughness factor (K_R) and the power law exponent (α) being 1.10 and 0.14, respectively. The conductor and insulator drag coefficients (C_N) were taken as 1.0 and 1.2, respectively, whilst the tower load was derived using a drag coefficient of 3.07 and an average area solidity ratio (ϕ) of 0.168.

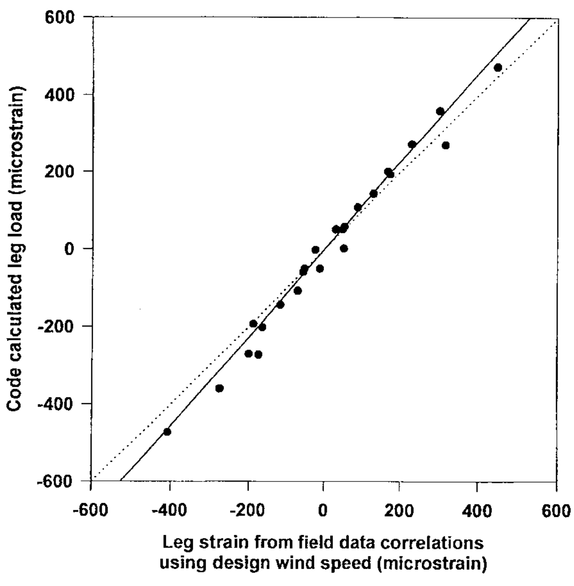

Fig. 15 Comparison between leg strain data from Code calculations using BS8100 and those computed from field data using the same reference wind speed

The comparisons undertaken so far indicate that, in most cases, the measured leg strains are lower in magnitude than the Code-predicted values. However, the results also confirm that for winds aligned with the conductors the wind load is greatly reduced, since there is little contribution from wind acting on the cables and, in a limited number of cases, the measured loads are greater than those predicted from the Code. This discrepancy generally arises under conditions of low wind speed or for cases where the wind is aligned with the conductors, so that the magnitude of the measured loads is small enough to be close to the resolution of the measurement instrumentation. Fig. 15 shows a comparison between the leg strain data computed from the Code, for a range of wind speeds and directions, and the corresponding data measured in the field. The Code calculations are not based upon the "design" wind speed derived from the Code but, rather, the actual wind speed measured in the field. Hence, each data point in the graph represents the same wind condition but with one axis giving the measured value and the other the Code-calculated value. The solid line is the best-fit to the data, whilst the dashed line is the 45 degrees line of equivalence. On this basis, the data show a good correlation, with the Code tending to overestimate the field loads by about 14%, especially at the higher wind speeds.

3.3. Numerical analysis of dynamic behaviour of transmission system

The experimental work has been complemented by a numerical structural analysis of the dynamic behaviour of the transmission system. As noted by Smith (1993), the loads on the conductors are associated with very long correlation lengths and so the dynamic magnification of the loads on the tower is generally small. Hence, the dynamic problems on such a transmission

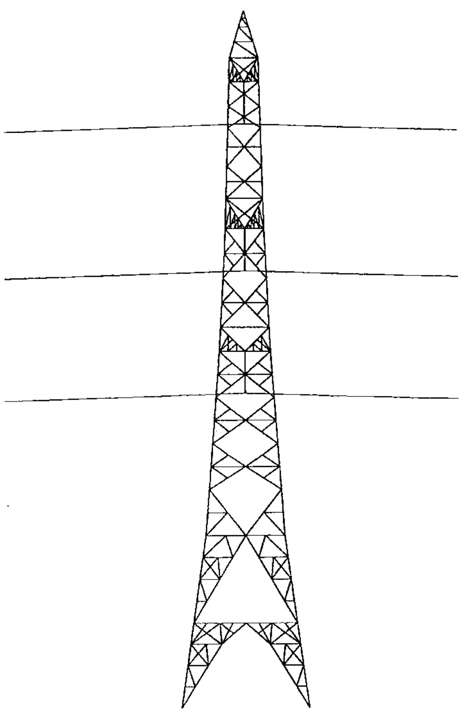

Fig. 16 Longitudinal elevation of tower structure used in analysis model

system under the action of normal "straight" (non-tornadic) winds is effectively due to the conductors. There are several approaches to the numerical structural analysis of such systems, with the simpler approaches adopting linear elastic analysis in which the tower lattice connections are pinned and only axial loading is considered. However, a comparison between the loading distributions measured in a typical bolted lattice truss test structure and those calculated assuming pin connections has shown that the effects of moment-induced stresses, arising from the bolting, also need to be considered, Knight and Santhakumar (1993). Without such a consideration the failure loads are normally overestimated. In the test they described the failure occurred at 90% of the calculated critical load. Detailed analysis of the behaviour of the lattice structure requires consideration of the non-linearities of the system, principally, geometric non-linearity, material non-linearity and joint flexibility/slippage, Al-Bermani and Kitipornchai (1993). It was also noted that typically, the tower members are asymmetric, thin-walled angle sections and eccentrically connected so that the detailed behaviour is difficult to analyse. The finite-element model presented by Al-Bermani and Kitipornchai (1993), called AK TOWER, incorporates such non-linearities and geometric details. Their predictions for a range of tower structures showed generally good agreement with measured data for ultimate loads, failure modes and deflected shapes. However, they also noted the fundamental problem that it was not possible to directly compare calculated member forces and actual member forces since such detailed full-scale measurements did not exist. In the present work the structural analysis has been carried out using the finite element code ABAQUS (1996), where the tower is treated as a space frame with six degrees of freedom (three translations and three rotations) per node, giving a total of 2898 degrees of freedom. Since the aim of the computations has been to assess the fundamental modes of the dynamic response, rather

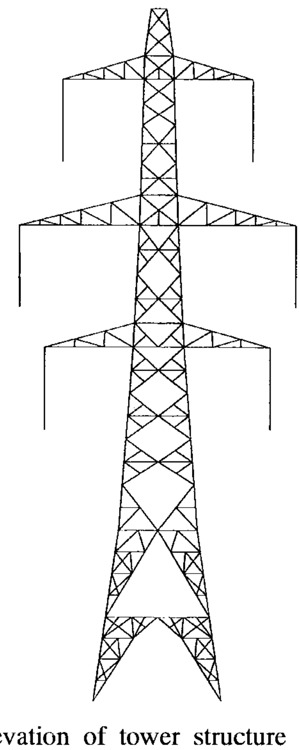

Fig. 17 Transverse elevation of tower structure used in analysis model

than detailed member loads, the eccentricities associated with the bolted connections have not been modelled. The non-linear analysis includes an extrapolation which gives the frequency and mode shapes associated with the dynamic behaviour of the tower. The input loading for the model is derived from Code of Practice calculations, British Standards (1986a,b). The arrangement of the structure used in the numerical model is illustrated by the longitudinal and transverse elevations in Figs. 16 and 17, respectively. The connections between the vertical insulators and the conductors and the tower cross-arms are modelled with pin-joints. Two sets of computations were carried out, the first concerning the tower alone, without any conductors present, and the second with the complete tower and conductor system. Although each transmission line between the insulators was comprised of four conductor cables they have been represented in the model by a single conductor of equivalent elastic properties. It is considered that the presence of the cable spacers ensures that, for the principal modes of oscillation computed here, the four cables will behave in a similar manner to a single conductor. Clearly, this will not be the case under extreme combined wind and icing conditions where flutter of the conductor bundles may occur.

Considering first the behaviour of the tower alone, the ABAQUS predictions show that the first mode is a transversal deflection at a frequency of 2.22 Hz, with the second mode being longitudinal and at a slightly higher frequency of 2.24 Hz. A hand calculation of the fundamental frequency, using the method described by Bolton (1983) and assuming a uniformly distributed mass of approximately 20 tonnes and a stiffness of 8.6×10^6 N/m, yields a value of 3.4 Hz, which is in reasonable agreement with the numerical model prediction. There are also a number of empirical expressions for estimating the natural frequency available in the literature, derived from full-scale test measurements. For example, Glanville and Kwok (1997) equate the frequency to $75/h$, where h is the tower height, thereby giving a value of 1.7 Hz in the present case. The

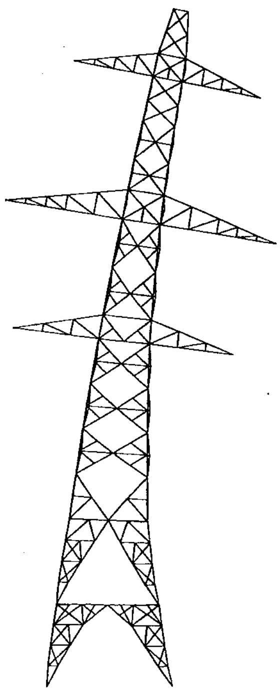

Fig. 18 Mode shape for dynamic behaviour of tower structure alone (Mode 1)

shape of the tower is taken into consideration by Holmes (1994), such that the natural frequency is given by 750 $(w_b+w_t)/h^2$, where w_b and w_t are the width of the tower lattice at the bottom and top, respectively. For the present case this yields a value of 3.5 Hz. Hence, it may be seen that the natural frequency predicted by ABAQUS lies within the range of the empirically computed values.

The corresponding mode shapes, resulting from the ABAQUS analysis are illustrated in Figs. 18 and 19, respectively. The third mode, not illustrated here, is in the vertical direction, at a frequency of 3.9 Hz, with the greatest displacements predicted for members within the third section of the lattice above the foundations. The fourth mode is torsional and also at a frequency of 3.9 Hz.

With the conductors included in the model, the behaviour of the system is fundamentally different with the primary modes of oscillation being determined by the cables, such that the dominant frequencies are much lower and of the order of 0.1 Hz. This effect was also noted in the field tests on a transmission tower by Momomura *et al.* (1997) in which the natural frequency of the 96 m tall tower alone was 1 Hz, whilst the first six modes of the combined conductor/tower system were all less than this value. However, in the present work, after iterating to give the first 400 modes it was found that the conductor behaviour still dominated the system, with the 400th mode occurring at a frequency of only 1.3 Hz. The first eight modes are illustrated in Fig. 20, all at a frequency of 0.1Hz. From the figure, it may be seen that the following mode shapes occur :

Mode 1 : Governed by lower conductors oscillating in phase on either side of cross-arm with nodes at the towers. Either side of the tower the cables are also in phase. The upper conductors on either side of the cross-arm are also in phase but with much smaller am-

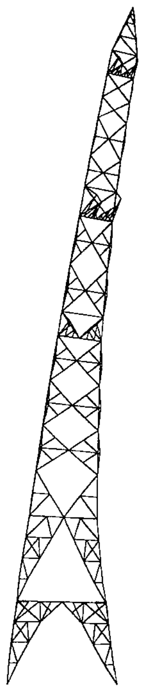

Fig. 19 Mode shape for dynamic behaviour of tower structure alone (Mode 2)

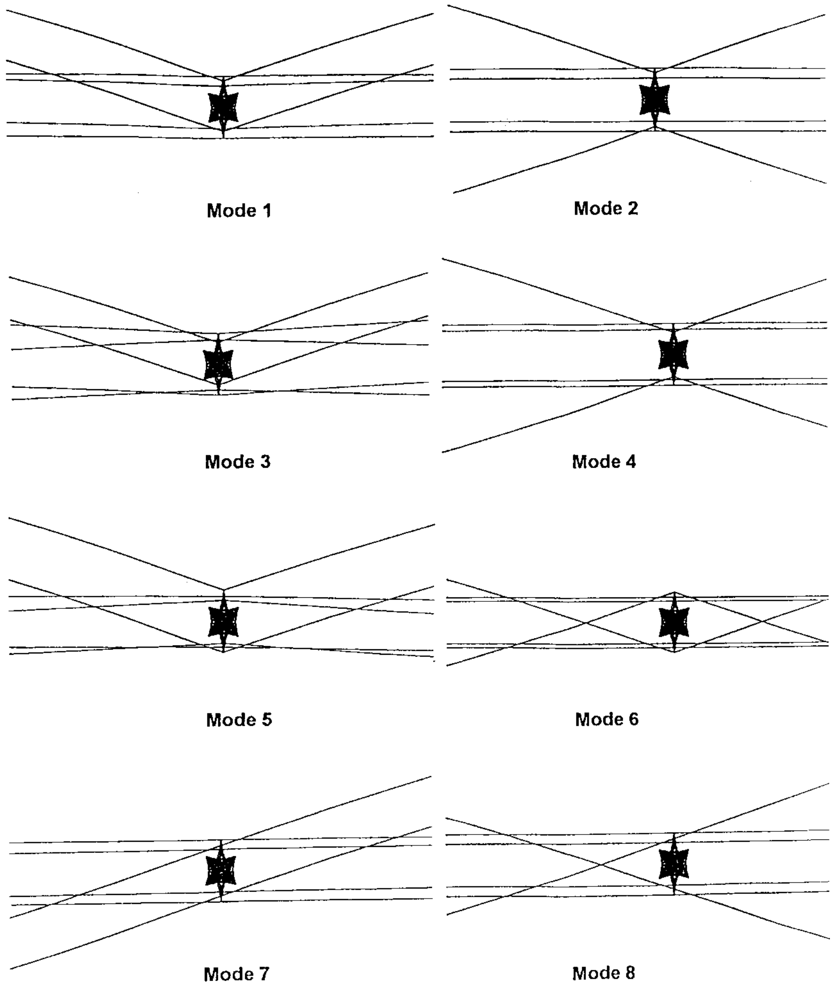

Fig. 20 First eight mode shapes for complete tower and conductor system

plitudes.
Mode 2 : Lower conductors on either side of cross-arm out of phase by 180 degrees, but conductors either side of the tower oscillating in phase. Conductors at other levels relatively stationary.
Mode 3 : Upper conductors on either side of cross-arm in phase and also in phase either side of the tower. Other cable pairs in phase but with much smaller amplitudes.
Mode 4 : Upper conductors on either side of cross-arm 180 degrees out of phase but cable system in phase either side of tower. Other cable pairs relatively stationary.
Mode 5 : Middle conductors on either side of cross-arm in phase and also in phase either side of the tower. Other cable pairs relatively stationary.
Mode 6 : Middle conductors on either side of cross-arm 180 degrees out of phase but cable system in phase either side of tower. Other cable pairs relatively stationary.
Mode 7 : Lower conductors on either side of cross-arm in phase but 180 degrees out of phase on either side of the tower. Other cable pairs relatively stationary.

Mode 8 : Lower conductors on either side of cross-arm 180 degrees out of phase and cable system similarly out of phase either side of tower. Other cable pairs relatively stationary.

Certainly, the first mode has been clearly observed by the authors at the monitored tower under conditions of only moderate wind speeds. However, due to the relatively low sampling rate of the tower leg strain gauges, it has not been possible so far to detect this dynamic behaviour in the foundation load spectra.

4. Conclusions

Within the limitations of the recorded time-histories for the winds at the tower site, the statistical data show no unexpected trends, with generally Gaussian PDF distributions and a prevailing South-Westerly wind direction. Analysis of all the data over the measurement period shows that the Weibull distribution gives a good fit to the wind speed values, with a shape factor (k) of 2.04 that is typical of UK parent winds. The structural analysis has shown that the tower structure, in the absence of the conductors, has a natural frequency of approximately 2.2 Hz and so significantly higher sampling rates will be required for an analysis of the dynamic behaviour of this tower. The tower instrumentation is continuing to provide data such that more detailed analyses will be carried out and published in the future. Nevertheless, the results obtained so far are presented in a graphical form which permits a reasonable prediction of the leg strains to be made for a given wind speed and direction. In addition, it has been found that, for a given wind speed and direction, the UK Code of Practice tends to over-predict the leg, or foundation, loads by approximately 14%, suggesting that the Code is not particularly conservative in this respect. Hence, in the absence of further data concerning the dynamic wind effects on the design foundation loads, the present experimental field data support the validity of the UK Code of Practice for assessing the wind-induced foundation loads for design purposes.

Acknowledgements

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