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AN EXPERIMENTAL AND FINITE ELEMENT STUDY OF THE LOW-CYCLE FATIGUE FAILURE OF A GALVANIZED STEEL LIGHTING COLUMN

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THEME

Durability

KEYWORDS

Low-cycle fatigue, FEA, test, validation, lighting column, welds, galvanizing.

SUMMARY

This paper presents the results of a low-cycle fatigue test on a lighting column. The wind induced vibration phenomena responsible for low cycle fatigue in such structures is discussed and the failure mechanism is examined. It was initially thought that poor quality weld detail was the major influence on the fatigue life of such columns. However, the significant role of the galvanised coating in the failure process is also highlighted.

The experimental results are compared with those from a detailed 3D finite element model. Various methods of calculating hot-spot stresses at welded joints are examined and use of a simple *peak stress removal* approach is shown to produce significantly different values compared with the other methods examined.

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1: Introduction

Fatigue failure of lighting columns, as a result of wind induced vibration, has received particular attention in recent years with the increased number of premature lighting and traffic column failures on exposed sites. This increase is generally a consequence of resonance, at wind speeds well within the column wind-gust design limits [1, 2, 3]. This type of failure generally occurs in a matter of hours, due to the rapid accumulation of load cycles and the high amplitudes of vibration facilitated by low damping. Figure 1 shows the aftermath of such an event during the installation of new 18m lighting columns next to Glasgow Airport in 60mph winds in early January 1993[4].

Figure 1: **Low-cycle fatigue failure of 18m lighting column.**

It may be observed that the columns failed at the fillet weld adjacent to the swage section above the door. The swage joint is the focus for the present investigation. The period of vibration, amplitude of vibration and time to failure for these newly installed columns, observable from the video footage associated with these failures, would indicate low-cycle fatigue as the failure mechanism. This type of failure proved to be a significant problem with particular designs of column throughout the UK.

Commercial pressure and changes of design have resulted in lighting columns more closely matched to their design loadings without the additional strength that may have been present in earlier designs [4, 5]. In common with many industrial sectors, design envelopes are being pushed in terms of operating conditions and manufacturing costs. It is not unusual when this happens that previously unforeseen failure mechanisms manifest themselves. This has been the case with lighting columns as designs push the maximum 18/20m height limit permitted in Codes of Practice. Modern designs often have fewer swage joints with fewer changes in section. This results in more mass further from the base and greater wind sensitivity. In some cases, the swage joint above the door is more abrupt than previous designs, resulting in poorer fatigue detail as a consequence. These changes are no doubt driven by a requirement to minimise manufacturing costs.

Such incidents highlight the need for a deeper understanding of the fatigue performance of columns arising from wind induced vibration phenomena and this has in turn attracted the attention of researchers. Peil and Behrens carried out theoretical and experimental studies into the influential topics of wind action, dynamic system response and structural behaviour and fatigue strength in the region of the door opening [6], proposing realistic alterations to design to increase column life expectancy. Work carried out by Pagnini and Solari demonstrated the basic hypothesis and examination procedure for mono-tubular columns and steel poles [7, 8]. A. P. Robertson investigated the influence of the base detail on the columns' wind induced response and consequent fatigue loading [9]. Further, in conjunction with Holmes and Smith [10, 11], Robertson verified a proposed simple, closed-form solution for fatigue damage and fatigue life in structures subjected to along-wind loading.

Relevant British Standards and European Design Codes for lighting columns, along with available literature, currently provide very little guidance and often inadequate detail on design requirements with regard to wind induced resonance and resulting fatigue. This is probably due to the fact that large amplitude wind-induced vibration of lighting columns, resulting in fatigue failure, is assumed not to happen for columns designed to relevant Codes of Practice [12, 13]. Further, fatigue of columns is generally not of major concern to most Operating Authorities [5], until unexpected and widespread failures arise, as was the cases mentioned previously at Glasgow Airport and throughout the UK. Although the cost of a single column is low, the cost of replacing such columns can be significant. Apart from the significant disruption to authorities and the general public, the reputations of manufacturers can also be damaged.

While guidance on the fatigue of welded steel structures in general is widely available, very few papers have been published on the fatigue failure of lighting columns. In addition, there is little guidance on the prevention of the wind phenomena, which give rise to resonant vibration in lighting columns.

1.1 Relevant Standards for Lighting Columns

BS EN 40: Lighting Columns [12] does not stipulate any detailed fatigue requirements and its effect only needs to be considered when specified by customers. While there is a requirement that columns must be designed to safely withstand wind loading, the stated loading is solely due to large gusts, with no consideration being given to wind induced large amplitude resonant vibration. Calculation of wind pressure caused by gusts is required however, taking into consideration the basic period of vibration and the damping of the column/lantern system.

The method of calculating the effects of fatigue provided, offers only simplified design guidance for different classes of weld detail and does not extend into the low-cycle domain. Requirements specifically for steel lighting column detail are discussed and reference is given to other standards regarding welding procedures. The importance of key structural joints, including the swage joint, is highlighted. In addition, there is a requirement for all welds to be free from cracks and lack of fusion. No reference is made to the performance of any galvanized coating in relation to fatigue.

Similarly, BD 94/07 [13] provides specific details on fatigue but no guidance with regard to wind induced resonant vibration, with only gust loading and buffeting from passing vehicles being considered. Columns located at very exposed sites are discussed, requiring compliance with BD 2/05: Technical Approval of Highway Structures [14]. For exposed sites, BD 2/05 requires independent checks on columns by an engineer. Procedures used for fatigue assessment have also to be agreed between designer, client and the overseeing organisation. Locations especially susceptible to fatigue damage are highlighted for the shoulder joint, flange plate and door opening. As fatigue is inherently dependent upon geometrical arrangement and fabrication, several constraints are discussed for cross sections of metal columns and the need for a fatigue check to be carried out at and adjacent to each weld, is highlighted.

NHSS 5020: Manufacture and Verification of Lighting Columns [15] states that a minimum of one column be taken from production to be sectioned and visually inspected per annum. In addition, surface protection must be in accordance with BS EN 40 Part 5, Annex A [12] for steel columns, which in turn indicates that hot dipped galvanising should comply with the requirements of EN ISO 1461 [16]. TR 22 [5] provides calculations for the time to failure due to fatigue cracking, indicating that most damage is induced in high winds. The design life is stated as 25 years.

None of the above guidelines provide specific guidance on wind-induced resonant vibration and the resulting low-cycle fatigue which can occur.

1.2 Wind Induced Vibration

Lighting columns are susceptible to two forms of wind loading, variable speed gusts and low speed excitation. Low speed excitation, caused by any of the three main wind phenomena, leads to the majority of column failures due to fatigue. The three phenomena are vortex shedding, galloping and buffeting.

Mass column failures, such as those at Glasgow Airport [4], are probably due to along wind galloping, although this has not been conclusively demonstrated. The critical wind speed is a factor of the column’s diameter [17]. This, along with the shape of the cross section, is one of the main factors in determining the critical speed. Designing a column with a lower surface roughness is another prevention technique against this phenomenon, as the critical Reynolds number region will occur at higher wind speeds. A column design with a higher value and consequential higher frequency is less susceptible to buffeting, again increasing the critical wind speed at which this phenomenon occurs. Additionally, high damping makes the structure less susceptible to any response. Wall thickness will also affect the response to wind loading, due to its effect on the stiffness, mass and damping of the column. It should be noted that “old fashioned” lamp-posts with several changes of sections achieved higher natural frequencies due to the reduction in mass with height. This was achieved through use of additional swaged joints and welding and no doubt manufacturing costs were higher as a result.

Pagnini and Solari, 1999 [8], considered damping in lighting columns to arise from three sources; aerodynamic damping, mechanical damping (structure/foundation/soil interaction) and structural damping (energy dissipated in the material and connections). Instabilities arise when the overall system damping changes. Critical wind speeds arise owing to the fact that aerodynamic loads are dependent on wind speed, and hence, so is the aerodynamic damping. Structural damping is unaffected by wind speed, but the critical point is where the aerodynamic forces cancel out the structural forces. This is where the overall system damping is zero and at this point large amplitudes of vibration are possible.

1.3 Some Comments on Fatigue Standards

BS 7608: Code of Practice for Fatigue Design and Assessment of Steel Structures [18] provides general recommendations for structural steel members exposed to repetitive stress fluctuations. This standard is due to be partially replaced in 2010, in the UK, by the current Eurocode 3: Design of Steel Structures [19]. With fabricated steel structures, welds are often the source of fatigue failure. The fatigue assessment procedures embodied in these standards involve the use of various weld categories and cracking scenarios. Each scenario has its own S-N curve. The probability of failure associated with this data often varies across codes.

ECCS N0105: Good Design Practice: A Guideline for Fatigue Design [20] offers direction for structures susceptible to fatigue loading, including wind induced oscillations. The guideline reviews current knowledge of fatigue design and the production of fatigue resilient structures. It is supplementary to Eurocode 3.

However, the main problem with these standards as far as the present work is concerned, is that the fatigue data do not extend down into the low-cycle regime – stopping at 10^4 cycles. In addition the codes do not provide plasticity correction procedures.

The pressure vessel codes of practice PD5500 [21] and EN13445 [22] use the same general approach as the structural codes, but also provide fatigue data for low-cycle fatigue and plasticity correction, where necessary. Although clearly not directly applicable, the low-cycle fatigue data in PD5500 have been used in this study to examine assessment procedures and design possibilities.

2: Experiment

2.1 Test Lighting Column Specification

The 12 metre steel lighting column selected for test, was designed in accordance with BS 5649 [23], now superseded by BS EN 40 [12] and manufactured to BS EN 10210 [24]. Sectional construction was completed by arc welding to BS 5135 [25]. Hot dip galvanising of the column was carried out according to EN ISO 1461 [16].

Tests were carried out to confirm that the column satisfied tolerance and manufacturing requirements specified in the above standards. In addition to excessive variation in the thickness of the galvanized coating, a further non-compliance, as will be seen, related to the quality of the welds. It was also noted that the shoulder angle of the swage joint was on the specified limit at 35 degrees. Chemical analysis demonstrated compliance with the standards.

2.2 Test Details

A shortened but otherwise full-scale lighting column was fatigue tested in cantilever bending. The test fixture used for the low-cycle fatigue test is shown in Figures 2 and 3.

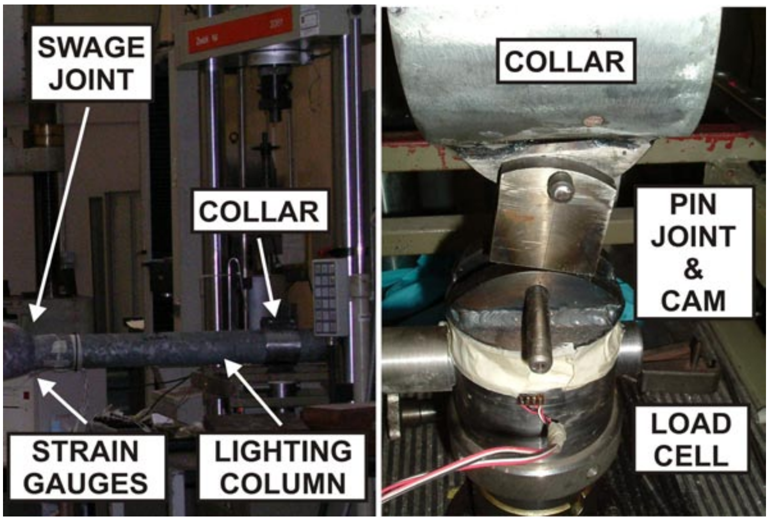


Figure 2: **Test set-up showing servo-hydraulic test machine and loading arrangement.**

The shortened lighting column was instrumented with strain gauges (on both the tension and compression sides of the column) as shown in Figure 4.

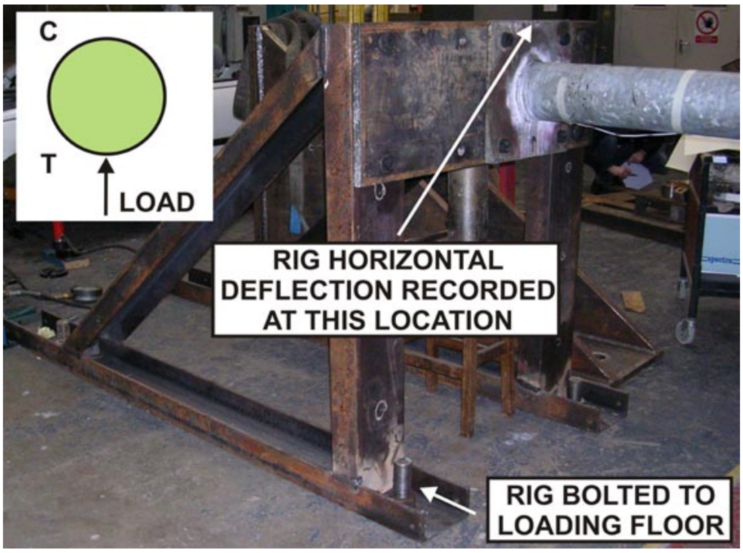
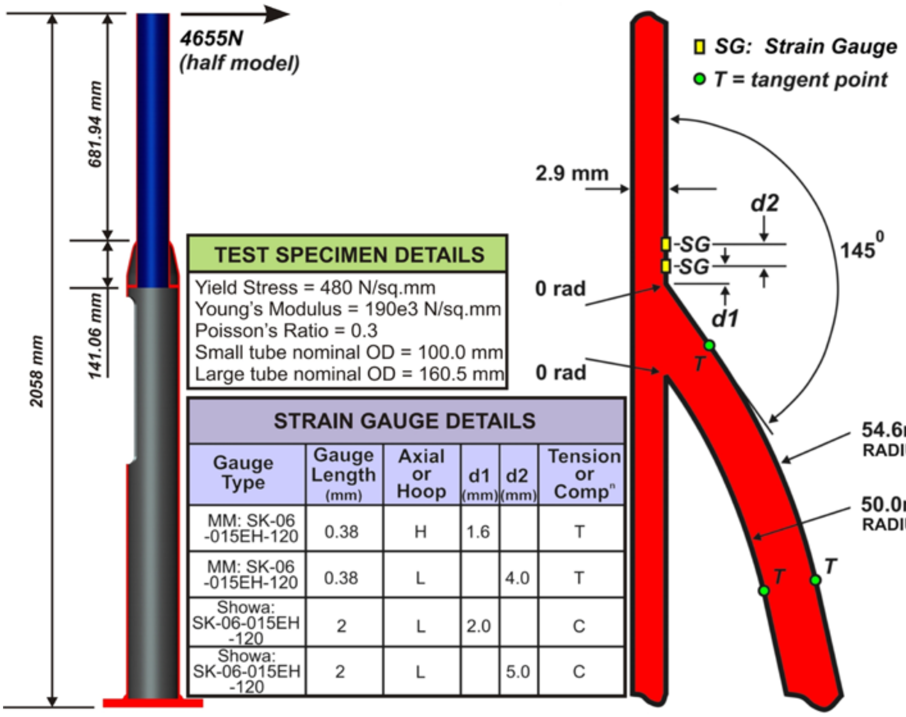


Figure 3: **Test set-up showing test frame and lighting column.**

The column was mounted horizontally in a test frame and the load was applied via a servo-hydraulic test machine. The shortening of the column was necessary due to the particular capacity of the test machine, in terms of load, cross head travel and cyclic rate.



TEST SPECIMEN DETAILS

Yield Stress = 480 N/sq.mm
Young's Modulus = 190e3 N/sq.mm
Poisson's Ratio = 0.3
Small tube nominal OD = 100.0 mm
Large tube nominal OD = 160.5 mm

STRAIN GAUGE DETAILS

Gauge Type	Gauge Length (mm)	Axial or Hoop	d1 (mm)	d2 (mm)	Tension or Comp^n
MM: SK-06 -015EH-120	0.38	H	1.6		T
MM: SK-06 -015EH-120	0.38	L		4.0	T
Showa: SK-06-015EH -120	2	L	2.0		C
Showa: SK-06-015EH -120	2	L		5.0	C

Figure 4: **Test specimen and instrumentation details.**

During testing, load, number of cycles, column end deflection, rig deflection and strain gauge readings were logged. The cyclic test was carried out under displacement control and the displacement amplitude of 58mm was selected to ensure that stresses in the vicinity of the weld were above initial yield. Due to the constraints inherent in the servo-hydraulic machine, a saw-tooth cycle was applied varying from zero to 58mm. This meant that only one side of the column was being subjected to maximum tensile stress and this was the door side of the column.

The resonant vibration of a lighting column is inherently a load controlled process, with the degree of damping and natural frequency of vibration changing as the fatigue crack grows and as the column approaches failure. The test loading represents a fixed amplitude resonant vibration and is a reasonable approach to determining the number of cycles required to grow a visible fatigue crack.

Column material properties were obtained from tests on 6 column specimens taken from the removed length of column. Prior to fatigue testing, dye penetrant testing showed no surface cracks in the welds of the column.

2.3 Test Results

Testing was carried on until 16500 cycles. At this point a crack had developed at the swage joint weld toe, on the tension side of the column. The test lasted approximately 10 hours. The crack was found to be approximately 102mm long in the hoop direction along the weld toe, as illustrated in Figure 5.

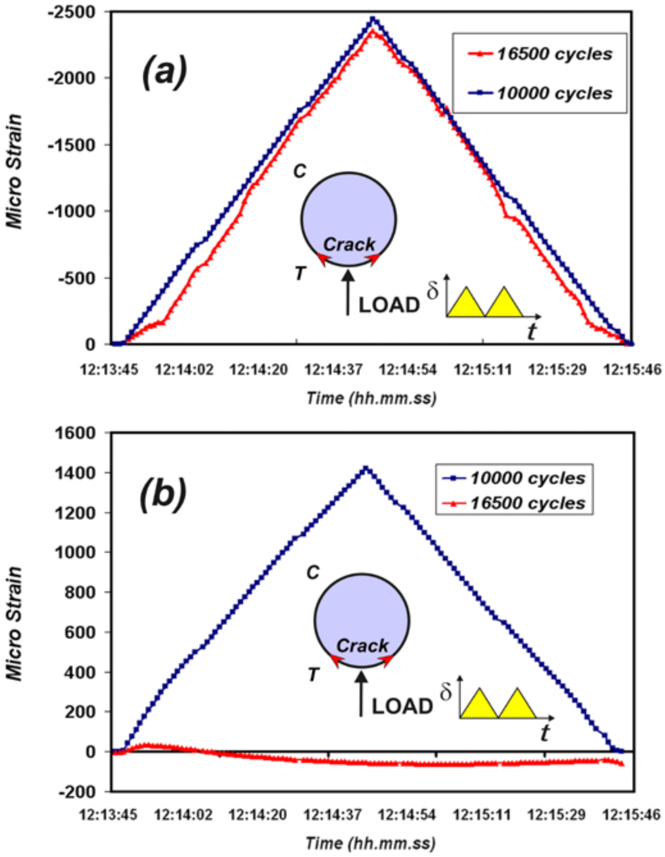


Figure 5: Strain-time variation for 10000 and 16500 cycles.

It should also be noted that a similar crack had developed at the base weld of the column. This area was not the subject of the present investigation however and is clearly not relevant to the buried installations in [4].

Figure 5 shows the strain-time variation for the 10000 and 16500 cycles, for the longitudinal gauges closest to the swage joint on both the compression (a) and tensile (b) surfaces. Figure 5(b) shows the strain drop-off on the gauge on the tensile side as the crack develops around the circumference of the tube. There is clearly no such drop-off on the compression side.

As mentioned previously, this was a displacement controlled test and Figure 6 shows that the load required to displace the column by 58mm had fallen slightly between the 10000 and 16500 cycles. There is also an observable increase in the size of the hysteresis loop, which is to be expected.

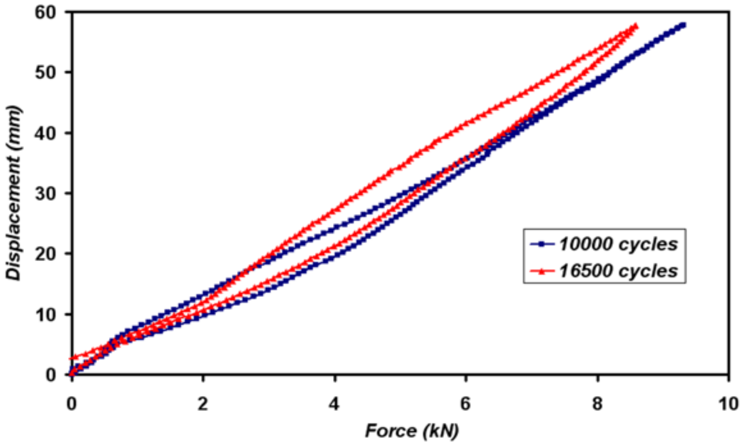


Figure 6: **Load versus displacement for 10000 and 16500 cycles.**

2.4 Micrographs

To allow analysis of the fatigue failure, various micrographs were produced as shown in Figures 7and 8.

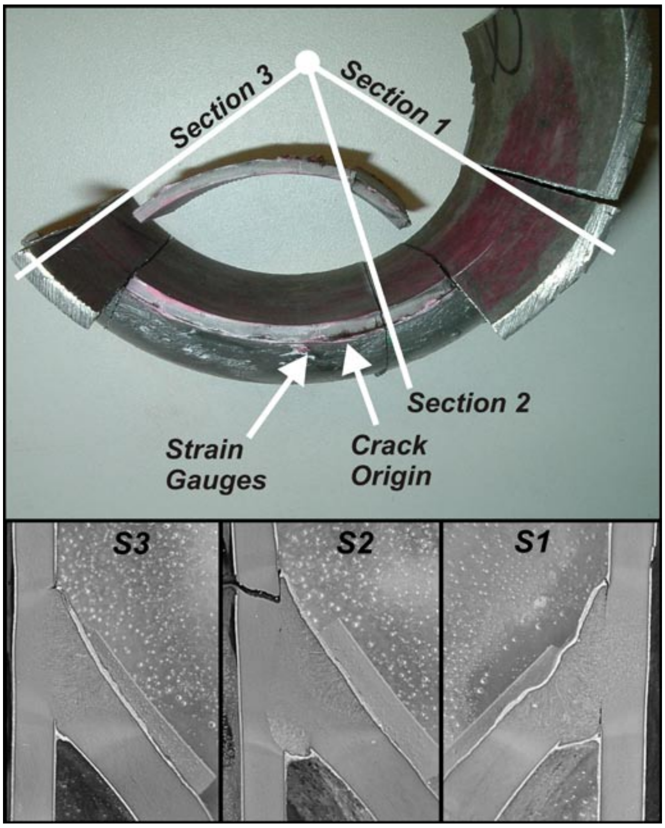


Figure 7: **Micrograph specimen location.**

First of all it may be observed in Figure 7, that the quality of the weld is very poor. At section S1 there is lack of penetration, poor root formation, wall thinning and irregular weld profile. This un-fused weld metal is referred to as a “cold lap” and can be a problem with high deposition rate welding. They can inhibit the successful application of weld toe improvement techniques such as toe grinding and also have the additional problem that the growing fatigue crack is not visible and hence is less easy to detect. Figure 7 also shows that the root formation is variable across all three sections, as is the thickness of galvanised coating. Cracks are evident at the weld toe in all three sections.

The micrographs for section S1 in Figure 8 show that the toe crack, instead of propagating through the thickness, as would be expected, has joined with the “crack” formed by the “cold lap”. The fact that these cracks are due to lack of fusion is apparent in Figure 7, as the root is still sealed with galvanizing.

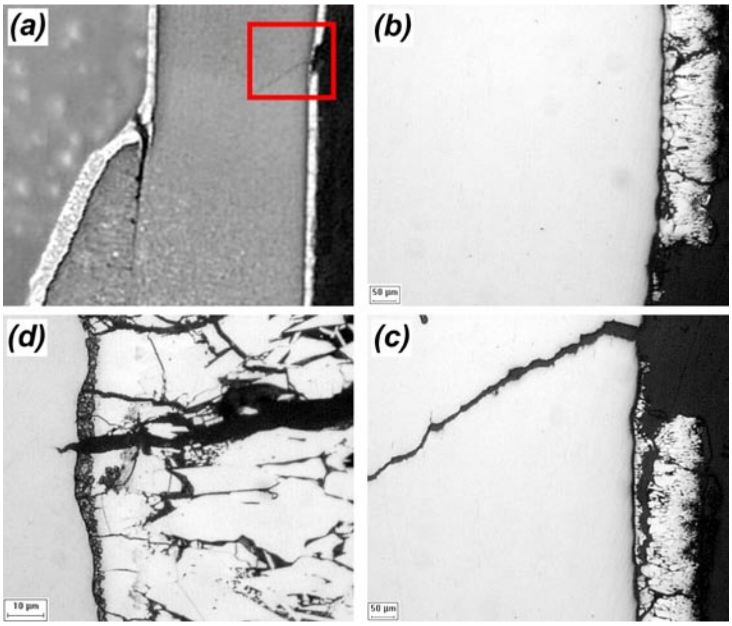


Figure 8: **Micrographs for specimen 1.**

Interestingly however, Figures 8(a) and (c) show a fatigue crack developing from the inside surface of the tube. This is somewhat unexpected, in that the inner surface is not welded and lacks the stress concentrating toe. In addition, the nominal bending stress is not as high on the inner surface as it is on the outer. However, it may be observed that there is a section of the galvanised coating missing. It is postulated that the coating has de-bonded from the steel substrate due the axial compressive stresses in this vicinity. This in turn has acted as a severe stress concentration, leading to the development and propagation of the crack shown. The galvanised coating shows evidence of significant damage and cracking. Figure 8(d) shows a crack just starting to run from the galvanizing into the steel. It would therefore appear that the debonding scenario is not necessary for crack initiation and propagation. These are significant observations and indicate that weld improvement may not in fact increase the fatigue life of galvanized steel lighting columns.

The detrimental effect of galvanizing on the fatigue performance of steel components is not new, although other studies have related to high cycle fatigue. Vogt, Boussac and Foct [26] showed that, under cyclic loading, cracks in the galvanised coating rapidly develop, propagating into the ferritic steel substrate, accelerating the fatigue damage of the material. They noted that the resultant thermal pre-existing crack networks of the galvanising process were not related to the fatigue process, clearly demonstrating that the damaging of the steel substrates arises as a consequence of cracking in the coating. This is not thought to be the case in the present study. Additionally, they observed that galvanising influences crack distribution resulting from fatigue, together with reducing the fatigue strength, particularly when the coating is thick. Under cyclic stresses, Bergengren and Melander [27] showed that the high cycle fatigue strength of high strength steel was reduced when the alloy was galvanised.

Further details of the metallurgical investigation of the fatigue cracks and the galvanizing are presented in a related journal paper, recently accepted for publication [28].

3: Finite Element Analysis

The linear elastic, small displacement, finite element model of the lighting column examined in the present investigation is shown in Figure 9. A half model was used as a result of symmetry. The finite element model consisted of 16011 tetrahedral solids, as implemented in the Pro-Mechanica Applied Structure system [29] from Parametric Technology Inc. The Applied Structure system uses adaptive-p element technology. In the analysis reported, the maximum polynomial edge order was 5 with an RMS stress error estimate of 1.3%. Given the high order nature of these elements, the mesh in the vicinity of the swage joint, as shown in Figure 9, must be regarded as extremely fine – 5 solids through the thickness with 5th order polynomial shape functions.

The base of the column was assumed fully fixed. This is in contrast to the experimental set-up, where slight flexibility in the rig leads to much larger tip deflections due to the rigid body rotation. To ensure a valid comparison, the finite element model was not subjected to an end deflection, but instead was subjected to an end load. The load used (4655N), was the experimental load required to cause the initial 58mm experimental deflection.

Figure 9 shows 3 areas of high stress – the base weld, the lower corner of the door opening and the swage joint. As has been previously indicated, it is the swage joint which is the focus of attention in this study.

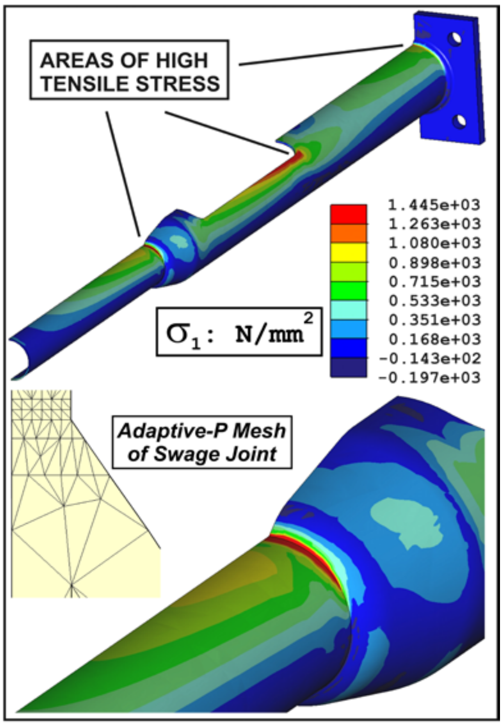
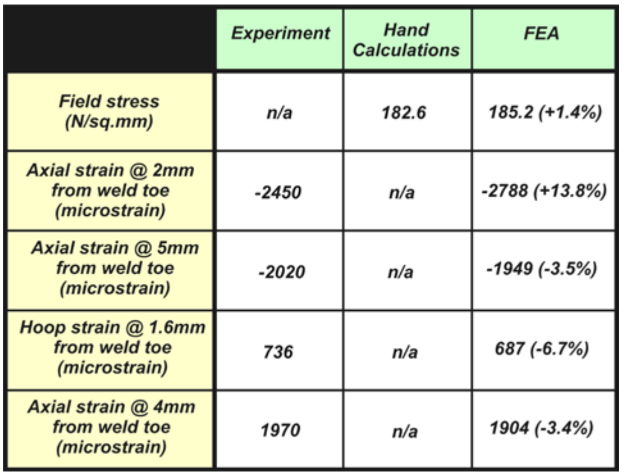


Figure 9: **Adaptive P-method Finite Element Model.**

Selected finite element results are compared with strain gauge results and hand calculations in Table 1, for the applied load shown in Figure 4. The comparisons are generally favourable. The highest errors occur in the comparisons with strain gauges closest to the weld. Given the steep strain gradient in this area and the very small length of these gauges, this is perhaps a reflection of the difficulties in precise location and alignment. The irregular nature of the weld will also influence the experimental results.



	Experiment	*Hand Calculations*	*FEA*
Field stress (N/sq.mm)	*n/a*	*182.6*	*185.2 (+1.4%)*
Axial strain @ 2mm from weld toe (microstrain)	*-2450*	*n/a*	*-2788 (+13.8%)*
Axial strain @ 5mm from weld toe (microstrain)	*-2020*	*n/a*	*-1949 (-3.5%)*
Hoop strain @ 1.6mm from weld toe (microstrain)	*736*	*n/a*	*687 (-6.7%)*
Axial strain @ 4mm from weld toe (microstrain)	*1970*	*n/a*	*1904 (-3.4%)*

Table 1: **Finite Element Comparison with Strain Gauges and Hand Calculation.**

The localized nature of the stress concentration at the swage joint is illustrated in Figure 10. The stress distribution on the toe side tends to infinity with mesh refinement due to the singularity at the weld toe. The extremely localised effect due to the weld (within a quarter of the tube thickness of the weld toe) is superimposed on the gross geometric stress concentration due to the swage. It is the localized component due to the weld that has to be removed for fatigue assessment. This is necessary because the fatigue allowables in the Codes of Practice already include the effect of the weld.

The stress distribution on the inner surface is continuous however. This distribution is terminated opposite the toe for comparative purposes. The very steep gradient on the inner surface is also apparent and the axial bending stress becomes compressive within a distance of one wall thickness of the weld toe position. The compressive strains associated with this stress field could be the source of possible damage of the relatively low-ductility galvanised coating in this region. This in turn will result in fatigue cracks developing from the inner surface as observed in the experiment.

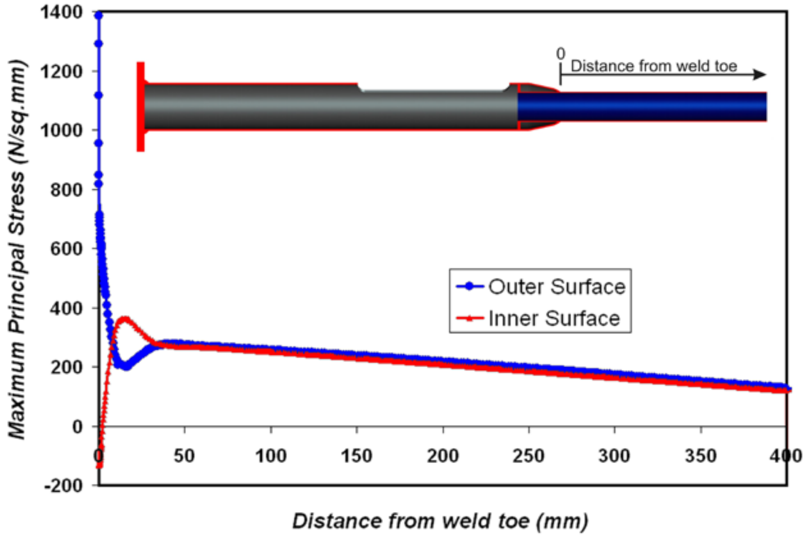
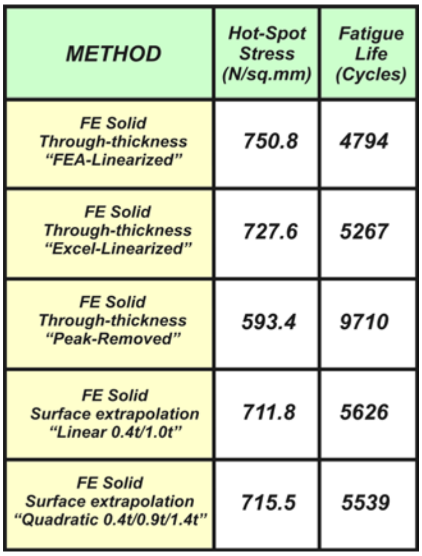


Figure 10: Maximum Principal Stress Decay at Swage Joint.

3.1: Hot-Spot Stresses and Fatigue Assessment

The fatigue assessment procedures in codes of practice require use of a structural stress or so-called hot-spot stress. There are various methods of obtaining these, with the European codes generally favouring surface extrapolation to remove the peak stress component due to the weld and the American codes advocating the use of through-thickness linearization methods. The hot-spot stresses obtained using the various approaches are shown in Table 2. The spread of results is relatively small, apart from the through-thickness “peak removal” technique. Rather than use linearization techniques, which provide bending stress components that are influenced by such peaks, the "peak removal" approach involves simply fitting a straight line to the linear portion of the through-thickness stress distribution and extrapolating this to the toe. This procedure effectively removes the peak component rather than allow it to affect the linearization. It is also argued that this procedure is more in tune with the experimental data, as the tabulated allowable stresses include gross geometrical stress effects but not peak. This relatively small spread in results will be due to some extent, to the fact that highly refined and converged results have been used. The highly refined mesh was used in an attempt to remove "non-convergence" as a variable from the process. It is also argued that ideally, techniques for obtaining hot-spot stresses should be based on the use of converged results and not on notions of coarse meshes, fine meshes and recommended element sizes.



METHOD	Hot-Spot Stress (N/sq.mm)	Fatigue Life (Cycles)
FE Solid Through-thickness "FEA-Linearized"	750.8	4794
FE Solid Through-thickness "Excel-Linearized"	727.6	5267
FE Solid Through-thickness "Peak-Removed"	593.4	9710
FE Solid Surface extrapolation "Linear 0.4t/1.0t"	711.8	5626
FE Solid Surface extrapolation "Quadratic 0.4t/0.9t/1.4t"	715.5	5539

Table 2: Comparison of Hot-Spot Stress Methods.

The differences in through-thickness approach are illustrated in Figure 11.

The fatigue data contained in codes of practice for welds already includes the effect of the weld. Assessment therefore requires removal of this component from analysis results, before comparison is made with the S-N data. Through thickness methods which use a linearization process will be influenced by the curved portion of the "Raw FEA Data" line in Figure 11. That is, the slope of the membrane plus bending line obtained will be affected by the linearization process itself. It is in fact this curved portion of the line which must be removed. The logical approach therefore would simply be to fit a linear trend line to the straight portion of this curve and to extrapolate it to the toe position.

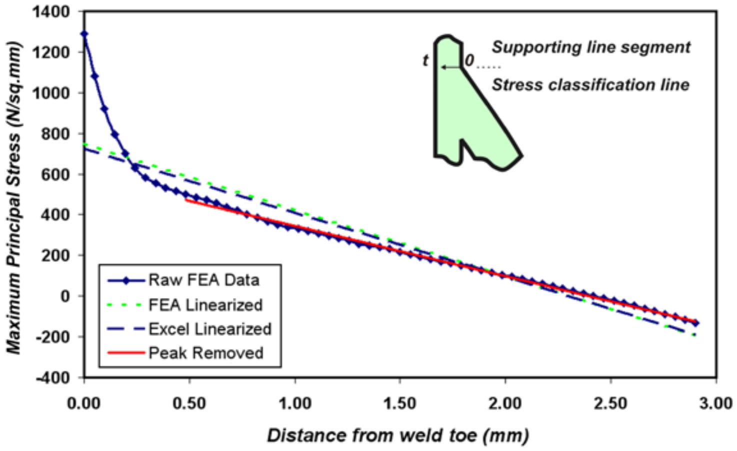


Figure 11: Through-thickness Peak Stress Removal Methods.

The Excel Linearized line is simply a best-fit straight line fitted to the entire data. The FEA Linearized line on the other hand, used the linearization algorithm inherent in most FEA systems. In this case the linearization is carried out on a stress component basis, before recombining to obtain the membrane plus bending components of the principal stresses.

Given the logarithmic nature of S-N data, small differences in stress range can result in large variations in fatigue life. To illustrate this point, the various low-cycle fatigue lives, assuming that the swage joint corresponds to a category F2 weld in PD5500, are shown in Table 2.

If a high quality weld corresponding to a full penetration butt weld was possible and practical, the fatigue life using the peak removal method would increase from 5267 cycles to 30,916. At 1.5Hz (which is not untypical of the fundamental natural frequency for this type of column), this corresponds to a resonant duration of 5hrs 44mins. In reality, use of a full penetration butt weld, located as far away from the swage as possible, would also result in a reduction of stress range, which would enhance these figures further. The use of a galvanized coating with higher ductility may also be possible.

Before concluding this paper, it may be informative to mention the unusual way that these estimates of fatigue life were obtained for the lighting column. First of all, S-N data from pressure vessel codes of practice were used due to the availability of low-cycle fatigue data. The swage joint was assumed similar to a class F2 joint in PD5500. The pressure vessel codes also include procedures for correcting elastic stress ranges due to plasticity effects. These procedures proved unnecessary in this case, due to the fact that the stress range was less than twice yield. Given the nature of design codes, S-N data is presented for low probabilities of failure. In this case, the S-N data were adjusted to mean plus two standard deviations i.e. ensuring a 97.6% probability of failure. It should also be noted that the S-N tests carried out to generate these S-N data would have defined failure as a crack causing leakage of a pressure component. In spite of this less than ideal approach to fatigue assessment, it is argued that the analysis supports the substantive conclusions reached.

4: Conclusions

Attention is drawn to the need for a better understanding of the fatigue performance of columns resulting from large amplitude wind induced resonant vibration. For future cost effective design and safety, further work in this area is necessary. It is recommended that lighting column design codes incorporate guidelines for designing against such wind induced vibration phenomena. It would obviously be preferable if this could be done without unduly increasing the costs of lighting columns.

The swage joint was confirmed as a failure location by both experiment and finite element analysis. This in itself is not surprising and the position of the fatigue failure at the swage joint is consistent with failures observed elsewhere [4]. Of more importance is the fact that the experiment showed that galvanizing can lead to premature failure of such columns. This is a highly significant conclusion in that even improving the weld detail may not in fact increase the fatigue life of lighting columns coated in this manner.

It should be noted that the randomly selected specimen column did not in fact meet the requirements of the codes of practice to which it was designed, as the weld at the swage joint was of sub-standard quality and the galvanising was found to be too thick in some areas.

A low cycle fatigue assessment was carried out using some of the hot-spot stress techniques recommended in codes of practice. An alternative approach, involving removal of the peak stress component, provides a procedure, which is arguably simpler and more in keeping with the fatigue allowable data. It is further argued that finite element techniques for the determination of hot-spot or structural stresses, should be developed on the basis of an accurate understanding of local stress distributions in the vicinity of the welds. This would require the use of converged results i.e. results that are insensitive to further mesh refinement. It is also likely that trends indicating the increasing use of solid elements, finer meshes, bigger models, increasing automation and more powerful computers will continue. It would seem prudent therefore to develop new techniques with this in mind. Clearly current constraints must also dictate practical solutions for the time being however.

The approach adopted so far for lighting column resonant vibration, has been to try and avoid it. Clearly this has not always been possible as designs push the limits permitted by Codes of Practice. This study has identified the variables governing the low-cycle fatigue performance of lighting columns and has shown that it should be possible to improve the fatigue life of columns subjected to wind induced large amplitude resonant vibration. This phenomenon is of a relatively short duration and the goal would be to ensure that the columns survived the period of vibration.

Given that the most severe corrosion will be on the outside of columns and the detrimental effect of galvanizing on fatigue performance, then columns may benefit if the inner surface was not galvanized in regions of stress / strain concentration.

While one test does not provide a very satisfactory basis for drawing conclusions, it is argued that the general thrust of the conclusions presented in this paper are worthy of further investigation and consideration.

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