Guidance Note 22/19

Asset-Management Toolkit: Minor Structures

Risk management guidance applicable to supports for luminaires, signals, signage, CCTV, electronic equipment and the like used in highways, transportation, rail, water, docks and harbours, retail and similar

Supersedes Institution of Lighting Engineers Technical Report TR22 *Managing a Vital Asset: Lighting Supports*, which is now withdrawn

Copyright © 2019 ILP

All rights reserved. No part of this publication may be reproduced, stored in a retrieval system or transmitted in any form or by any means: electronic, electrostatic, magnetic tape, mechanical, photocopying, recording or otherwise, without permission in writing from the Institution of Lighting Professionals.

Institution of Lighting Professionals
Regent House
Regent Place
Rugby
Warwickshire
CV21 2PN

Tel: (01788) 576492
Email: info@theilp.org.uk

Website: www.theilp.org.uk

Registered Charity Number 268547

Preface

It is intended that this document will be applicable to those responsible for the management of assets such as poles used as supports for lighting, CCTV, signs, signals, electronic equipment and the like. Many of these supports will be present on the highway but, equally, they are extensively used within the rail network, docks and harbours, retail establishments such as car parks, industrial plants, and utility compounds. These supports or structures can be overlooked or treated as 'fit-and-forget' but it is essential to ensure there is a regular inspection regime in place if catastrophic failure of the asset is to be avoided.

GN22: *Asset-Management Toolkit; Minor Structures* (AToMS), replaces the Institution of Lighting Engineers' (now Institution of Lighting Professionals, ILP) Technical Report 22 *Managing a Vital Asset: Lighting Supports* (TR22). Though TR22 could have been applied to more than just highway lighting and traffic sign poles, GN22 has been written to apply also to asset managers responsible for supports in the other sectors.

This document promotes an asset management risk-based approach, demonstrating how supports can be assessed and their structural integrity understood and managed drawing on the highways sector code of practice Well-managed highway infrastructure, published in 2016, as well as the Highways Maintenance Efficiency Programme (HMEP) guidance. These documents are equally applicable to any owner of non-highway assets with responsibility for similar supports. GN22 incorporates the relevant aspects of ILE TR22, which is superseded by this document.

The step-by-step risk management guidance contained in this document will allow the owner of the supports to develop a management strategy for their long-term operation, and develop a deterioration model which can be applied to other assets. This will allow the asset manager to optimise the available funds and support the decision-making process regarding priorities and programmes.

Institution of Lighting Professionals
Regent House
Regent Place
Rugby
Warwickshire
CV21 2PN

HEA Highway Electrical Association

LIGHTING UK LIGHTING BOARD

TRANSPORT FOR LONDON

ILP INSTITUTION OF LIGHTING PROFESSIONALS

Free4m Consulting

Department for Transport

UK ROADS LIAISON GROUP

wsp

Acknowledgements:

Project sponsors

This document was sponsored, funded, and has the full endorsement of:

- Institution of Lighting Professionals
- UK Lighting Board of the UK Roads Liaison Group
- Highway Electrical Association
- Transport for London.

Organisations

Special thanks to the following organisations for their assistance in the research and development of this document.

- Westminster City Council
- The British Institute of Non-Destructive Testing (BINDT)

The Institution of Lighting Professionals wishes to express its appreciation in the development of this guide for the assistance and cooperation extended by:

Authors

Allan Howard	Technical Director WSP
Tony Parasram	Free4m Consulting Ltd

Project steering group

Peter Burbidge	Association of Directors of Environment, Economy, Planning and Transport (ADEPT)
Dave Franks	UK Lighting Board (UKLB)
Peter Harrison	Institution of Lighting Professionals (ILP)
David Lodge	Lighting Column Technical Forum (LCTF)
Gareth Pritchard	Highway Electrical Association (HEA)
Graham Smith	Highway Electrical Association (HEA)
Matthew Wass	Lighting Column Technical Forum (LCTF)
Haydn Yeo	Institution of Lighting Professionals (ILP)
David Yeoell	Independent advisor and Past President of the Local Government Technical Advisers Group (TAG)

Non-destructive test houses

The following organisations supported the production of this document and supplied information obtained through their activities:

- Electrical Testing
- Kiwa
- Roch Services
- Reilux

Foreword

If you are responsible for poles supporting luminaires, signs, mobile phone apparatus, smart city equipment, signals, hanging baskets, cables, or similar then this document will be of use to you.

There are more than 7.5 million lighting columns and sign posts, generally known as lighting supports, on the UK roads' infrastructure. In addition, there are many more thousands of supports in publicly accessible areas such as railways, docks and harbours, parks, supermarkets, industrial premises, and the like. These supports are not 'fit-and-forget'; they need to be maintained just as much as the equipment they support.

Since 1997 the Institution of Lighting Engineers' (now Institution of Lighting Professionals) Technical Report 22 (TR22) *Managing a Vital Asset: Lighting Supports*, has been the 'go to' document containing the underpinning methodology upon which most, if not all, of the UK lighting supports have been managed. Unfortunately, there has been a perception that TR22 is applicable just to highway assets; this is not the case.

TR22 refers to lighting supports, but within the Design Manual for Roads and Bridges (DMRB) reference is made to minor highway structures, including:

- lighting columns;
- cantilever masts for traffic signals and/or speed cameras;
- CCTV masts;
- fixed vertical road traffic signs.

For consistency, lighting supports will be referred to in this document as minor structures.

Until 2016 the management of highway assets has, generally, been in line with the following codes of practice produced by the UK Roads Liaison Group:

- Well-maintained Highways
- Management of Highway Structures
- Well-lit Highways

In October 2016 the UK Roads Liaison Group published '*Well-managed Highway Infrastructure: A Code of Practice*' (WMHI). This code of practice supersedes the previous codes and was introduced as the basis for the risk-based management of all highways in the UK rather than the descriptive approach of the previous codes, and is published as a single document in four parts:

Part A: Overarching Principals

Part B: Highways

Part C: Structures

Part D: Lighting

Changing from reliance on specific guidance and recommendations in the previous codes to a risk-based approach determined by each asset owner will involve appropriate analysis, development, and gaining of approval through owners' executive processes. There is a requirement for all highway authorities to have implement a full risk-based approach contained in WMHI by October 2018.

The delivery of safe and well-maintained assets through a risk-based approach relies on good evidence and sound engineering judgement. The intention of this document is to assist asset owners to develop their own levels of service.

This tool kit has been developed to provide guidance for those responsible for minor structures – whether on the highway or not – to consider when developing their risk-based approach in accordance with local needs, priorities, and affordability.

Haydn Yeo
Vice President Technical
Institution of Lighting Professionals

Contents

Introduction

There are more than 7.5 million lighting columns plus many other forms of luminaire supports such as brackets in public ownership in the UK as well as many other luminaire supports and minor structures under private ownership within retail developments, private estates, rail network, airports, car parking facilities, utility companies, commercial premises etc, all of which require maintenance and upkeep. This document refers to minor structures that support luminaires, signals, CCTV, communications and surveillance equipment, signage etc.

The majority of these minor structures are steel with a painted, metallic sprayed, or galvanised protective coating, and of various construction configurations and material wall thicknesses. These can be subject to corrosion leading to catastrophic failure, which poses a significant health and safety risk to the public and infrastructure.

Lighting columns make up the majority of minor structures and are also subject to attachments which can, if outside the supporting design parameters, lead to the minor structure being overloaded, and hence result in premature and catastrophic failure.

A great variety of minor structures exists in the UK, from 'functional' types to more elaborate heritage or ornate types. However, for conventional lighting the vast number of minor structures falls into one of the following main types:

- Lighting column, with or without brackets, of varying profiles and designs;
- Cast iron columns/pedestals;
- Columns with the addition of embellishment kits;
- Wall brackets;
- Specialist supports such as catenary systems;
- Other columns/supports such as those for:
    - Signals in the highway and rail sectors;
    - Signage.

Most exterior lighting is supported on a form of column or pole; these are the main area of industry concern and hence the focus of this guidance. Other forms of minor structures such as wall brackets and catenary systems are covered in Appendices 17 and 18.

The appendices contain further details regarding specifying minor structures, lighting columns, column history, and specific types of columns, together with comments on specific details where problems, including corrosion, can occur.

In addition, there is a range of other materials used such as stainless steel, aluminium, cast iron, bronze, wood or fibre-reinforced polymer composites, and it should be noted that some columns may by design or materials used also be passively safe in nature.

All luminaire supports deteriorate and will require inspection, testing, maintenance and replacement. The life of the support, the frequency of maintenance and the type of inspection will vary according to the location, use, design and materials of the support in question.

All these factors require consideration and can involve various components such as luminaires, brackets, attachments and embellishment kits. It is essential therefore that planned regular structural inspection and testing is carried out as part of the maintenance regime. This then feeds into a risk-based asset-management system that can be used to monitor and understand the supports' condition, advising on future inspection/testing frequencies/regimes as well as advising on the degradation of the column with an indication of remaining operational life expectancy.

To ensure that such assets are efficiently maintained, asset-management principles and techniques should be applied. Asset management is about 'doing things well and delivering the agreed service (vision, goals and priorities) in the most cost-effective and sustainable manner'.

In England, the Highways Maintenance Efficiency Programme (HMEP), which is endorsed by the UK Roads Liaison Group (UKRLG), recognised that better advice and information was required if local authorities and other asset owners were to benefit consistently from the potential that asset management offers. HMEP developed a number of guidance documents for local highway authorities to support the adoption of asset management principles and enable implementation of the benefits of long-term planning. These are:

- *Highways Infrastructure Asset Management Guidance*, HMEP May 2013 http://www.highwaysefficiency.org.uk/efficiency-resources/asset-management/highway-infrastructure-asset-management-guidance.html;
- *Lifecycle Planning Toolkit*, HMEP November 2012 http://www.highwaysefficiency.org.uk/efficiency-resources/asset-management/life-cycling-planning-toolkit.html.

These guidance documents support a more effective and efficient approach to the management of highway infrastructure including public lighting.

In addition, the national code of practice *Well-managed Highways Infrastructure* http://www.ukroadsliaisongroup.org/en/codes/ should be adopted as it provides guidance on the efficient, effective and economic delivery of maintenance services, and includes advice on all new and emerging issues and technical developments.

Both the HMEP asset-management guidance and the code of practice for highways infrastructure recommend that:

> *'A risk-based approach should be adopted for all aspects of highway infrastructure maintenance, including setting levels of service, inspections, responses, resilience, priorities and programmes'.*

This guidance has been produced to provide:

> *step-by-step guidance to local authorities and other asset owners/managers with regards the full-life management of their lighting asset with regard to maintenance management of luminaire supports, columns, structures and associated support systems.*

It considers the lighting support as a structural element supporting a lighting asset, and accordingly assesses the impact of deterioration using structural engineering principles. It provides the asset manager with guidance regarding the full-life management of lighting assets through the adoption of sound inspection, assessment and testing practices, enabling the development of a full-life asset-management strategy, condition-assessment strategy and risk-assessment strategy.

This guidance promotes the application of good asset management using a risk-based approach. In respect of inspection and testing, it provides clear guidance to asset owners/managers as to how to develop strategies that best suit the asset base, the service requirements for inspection, and how to select the most appropriate test regime considering the purpose of the test and what results it will give the asset manager.

ISO 55000 defines asset management as:

'The systematic and coordinated activities and practices through which an organisation optimally and sustainably manages its assets and asset systems, their associated performance, risks and expenditures over their life cycles for the purpose...'

This guidance introduces the application of the 'As Low As Reasonably Practicable' (ALARP) principle, published by the Health and Safety Executive, to manage risks arising from deteriorating stock. As the appetite for risk varies from asset owner to asset owner, life-cycle plans will inevitably vary to deliver the asset owner's defined outcomes. Life-cycle plans are formulated to deliver the asset owner's defined service levels, which are non-technical statements of service objectives or outcomes easily understood by the public reflecting the asset owner's priorities and interests.

Strategies to deliver service levels are discussed with examples of potential strategies provided in Appendix 8. Maintenance strategies may be developed to deliver a condition-based maintenance regime, ie to maintain the stock at a defined condition.

This guidance looks to deliver a clear understanding of all the relevant considerations applicable to managing the full life-cycle assessment of a lighting asset, including:

- Inspection, assessment and testing techniques;
- Interventions consideration and timescales;
- Recording and reporting requirements;
- Analysis and potential modelling;
- Inventory requirements (minimum required fields);
- Risk-management strategy that accounts for the potential additional loadings that are or may be attached;
- Common measurement matrix that all assessment techniques and practices are measured against (in a simple linear scale – Excellent, Good, Fair, Poor);
- Prioritisation of work options and investment planning.

This guidance assists the reader by leading them through the process of the need for an inspection and maintenance programme, how to develop strategies for such, what is a life-cycle plan and how it should be used. It advises as to why test, the purpose of the testing and what the results would provide and how these can then be considered and analysed. (See Figure 1.1.)

Testing generally arises from the need to investigate:
- The severity and extent of a defect or damage;
- The root cause of deterioration;
- The rate of deterioration;
- The type of maintenance required and the extent of any repairs;

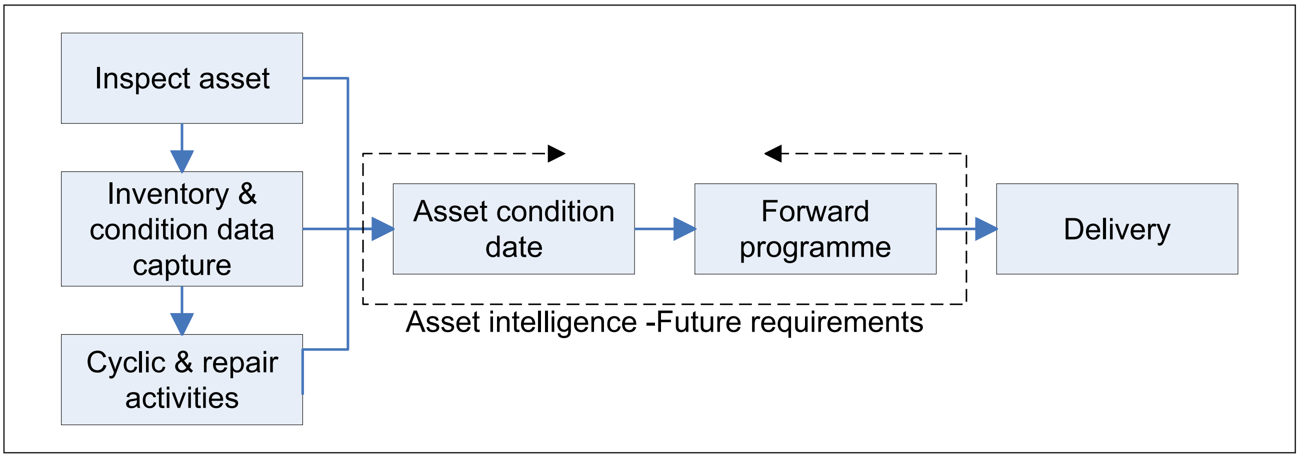


Figure 1.1: Asset condition assessment

- Material characteristics;
- Data for a structural assessment;
- A condition index.

A regular inspection and testing programme will gather information about defects and can record deterioration over time. The overall purpose of an inspection, testing and monitoring regime is to check that the physical assets are safe for use and fit for purpose, and to provide the data required to support the good management practices identified in the codes of practice and the organisation's policies and maintenance strategies.

An organisation's inspection, testing and monitoring strategies should be designed to:

- Provide data on the current condition and performance;
- Inform analyses, assessments and processes, for example change in condition, cause of deterioration, rate of deterioration, maintenance requirements, effectiveness of maintenance and structural capacity;
- Compile, verify and maintain inventory data;
- Meet the other requirements of the guidance in the code of practice.

The guidance introduces the application of deterioration modelling and provides an indication that specific service requirements for inspection and maintenance should be based on life-cycle plans that:

- Provide the long-term strategy for maintaining an asset, with the aim of providing the required level of service while minimising whole-life costs;
- Justify expenditure based on need, for both revenue and capital (zero-based budgeting).

This guidance also introduces a 'condition index' that can be used to determine whether the overall condition of the lighting stock is deteriorating, and use this as a means for monitoring whether adequate funding is being provided for maintenance work.

2. Scope

This guidance considers all luminaire supports including columns that would be covered by BS EN 40 *Lighting columns*, namely columns with a nominal height (including any bracket) not exceeding 20m in concrete, steel, aluminium and fibre-reinforced polymer composite materials. This guidance also considers surface mountings, wall brackets and catenary systems, and many aspects of the document also apply to other supports such as traffic signposts, CCTV masts and traffic signals. In this document, these are referred to generically as minor structures.

This guidance looks at common structural problems associated with different support types and materials, and provides details of inspection and testing regimes to help maintain the structural integrity of the minor structures stock. Through the application of this document, the asset owner can develop a risk-based asset-management approach to their minor structure assets based upon condition modelling.

The different non-destructive tests available at the time of publication are described, setting out their advantages and limitations and possible use. A risk-management strategy is detailed to assist owners and operators to assess when a minor structure needs to be inspected, tested or replaced.

The document does not address the subject of inspection and maintenance of high masts as covered by the ILP's Professional Lighting Guide 07 *High Masts for Lighting and CCTV* although many of the principles will still apply; nor does it address in any depth materials which are not in frequent use, such as timber/engineered wood, fibre-reinforced polymer composite or stainless steel, all of which will have specialised considerations. It is therefore mainly intended to give guidance on minor structures constructed from concrete, steel, cast iron and aluminium.

Electrical aspects, including site safety, are not included, but further information is available from the documents listed in the References and Bibliography.

3. Legal responsibilities

All asset owners have a responsibility to ensure that their assets are maintained in a safe and reliable condition and that everyone involved is competent in respect of the duties they must perform. Specific legal duties may relate to some lighting applications such as highways, and a number of these, together with the core principles, are discussed in this section and expanded on in Appendix 10.

Local highway authorities have a statutory duty to ensure that their highway networks are in a safe and reliable condition. The *Highways Act 1980* sets out the main duties of highway authorities in England and Wales. In particular, Section 41 imposes a duty to maintain highways that are maintained at public expense.

Useful guidance and interpretation of highway law can be found in either *An Introduction to Highway Law* or in *The Highways Act 1980 with annotations by Charles Cross and Stephen Sauvain*. Both publications include a number of examples of case law to demonstrate the duties and powers of a highway authority. Another useful reference document is *The Encyclopedia of Highway Law*.

Within the rail sector the Office of Rail and Road (ORR) operates a Risk Management Maturity Model (RM^3) which defines what excellence in risk management looks like, and allows organisations to assure themselves that their risk management is operating to an adequate level.

Private asset owners have a duty of care to those using the space and should also note that the outside environment surrounding their facility can also be considered as a work area, for example retail car parks where staff collect trolleys, offer car washing services etc.

The obligation embraces the two essential functions of 'Safe for Use' and 'Fit for Purpose'. The two functions are not the same:

- *Safe for Use* requires an asset to be managed in such a way that it does not pose an unacceptable risk to public safety;
- *Fit for Purpose* requires an asset to be managed in such a way that it remains available for use by those permitted for the route.

The recommendations made in the national codes of practice/standards are based on accepted good practice but are not explicitly mandatory on owners of highways (including public lighting). However, in cases of claims or legal action, the codes may be treated as a relevant consideration. In view of this, those who elect, in the light of local circumstances, to adopt policies, procedures or standards differing from those suggested by the codes, should identify these departures on a risk basis together with the reasoning for such.

Competency

All persons undertaking any assessment, design or appraisal must be competent regarding their roles. They must hold and be able to demonstrate the necessary behaviours gained through training, experience and application for the duties they are undertaking. This may relate to the undertaking of visual assessment, testing, specification, design and so forth. Such competencies must be demonstrable, and may include but not be limited to the appropriate grade of membership with an appropriate professional organisation, qualifications, and evidence of where such work has been undertaken within the past three years. The right people/resources must be employed for the relevant tasks.

Construction (Design and Management) Regulations

Any asset manager, asset owner, or asset supervisor commissioning any works must comply with the requirements of the *Construction (Design and Management) Regulations 2015* (CDM). If you issue instructions or orders for works to be carried out then you will probably be defined as the client under CDM. In that, no duty-holder must appoint a principal designer, designer, principal contractor or contractor unless they have taken reasonable steps to ensure that the organisation or individual they propose to appoint has the skills, knowledge, capacity and experience and, if they are an organisation, the organisational capability, necessary to fulfil the role in a manner that secures the health and safety of any person affected by the project.

An overview of health and safety, competency and CDM is provided in Appendix 9.

4. Asset management

To ensure that infrastructure is efficiently maintained, the principles and techniques of good asset management contained in *Highways Infrastructure Asset Management Guidance* issued by HMEP in May 2013 should be applied together with the adoption of the standards set out in the national code of practice *Well-managed Highways Infrastructure*. These provide guidance on the efficient, effective and economic delivery of highways infrastructure maintenance services and include advice on all new and emerging issues and technical developments.

The work needed to ensure effective maintenance of lighting assets can be split into three general headings:


Inspection and testing


Routine and reactive (ad hoc) maintenance


Planned maintenance

What is asset management?

This is not an easy question to answer because asset management means different things to different people and organisations. Part of the problem is that asset management covers such a wide range of activities (people, processes, data and systems at strategic, tactical and operational levels) that it is difficult to fully appreciate all the issues that asset management covers; as a result many people interpret asset management as the specific area in which they are directly involved.

The ISO 55000 series of asset management standards define asset management as

> *'The systematic and coordinated activities and practices through which an organisation optimally and sustainably manages its assets and asset systems, their associated performance, risks and expenditures over their life cycles for the purpose'.*

The *Highways Infrastructure Asset Management Guidance* (HMEP, May 2013) defines asset management as

> *'A systematic approach to meeting the strategic need for the management and maintenance of highway infrastructure assets through long-term planning and optimal allocation of resources in order to manage risk and meet the performance requirements of the authority in the most efficient and sustainable manner'.*

In simple terms, asset management means

> *'doing things well and delivering the agreed service (vision, goals and priorities) in the most cost effective and sustainable manner'.*

The introduction of asset-management practices does not replace existing maintenance good practice. Instead it provides the overall framework within which existing good practice may be more effectively applied and complemented by other practices.

Why adopt asset management

The main benefit of asset management is the ability to make better use of resources. This is normally demonstrated by:

- The same or better level of service at a reduced cost; or
- A better level of service for the same or marginally increased cost.

Specific benefits provided by asset management include:

- Improved decision-making through better information and resource allocation.
- Improved short-, medium- and long-term planning.
- Improved risk management through explicit identification and analysis of risks.
- Improved financial control through better understanding of costs and expenditure.

The majority of local authorities in the UK have adopted asset management and developed their own highways/transport asset-management plans (HAMP/TAMP). The extent to which each authority and/or asset owner has embedded asset-management principles is a measure of their asset-management maturity.

All asset owners start somewhere, and as their level of knowledge and use of asset-management techniques grow their asset-management maturity will develop through basic asset management to full asset management whereby significant benefits will be achieved. This can be illustrated by reviewing three typical management scenarios:

- short-term reactive management
- basic asset management
- asset management

Section 6 discusses the need to have an accurate inventory and how this information can aid the development of condition data and service levels. Adopting asset-management principles shows the outcomes migrate from a reactive approach, with poor value for money as there is little understanding as to asset condition, to a targeted-maintenance approach with planned interventions achieving a defined service level.

As the quality of the asset stock data improves, notably inventory and condition, life-cycle modelling becomes a useful tool to produce various strategies to fit either a fixed budget or a fixed stock condition.

Inventory

The starting point to any asset-management approach is asset inventory. Compared to knowing the condition of assets, an inventory is a relatively inexpensive dataset to collect/confirm. Section 6 discusses inventory in more detail, with a table of parameters used for life-cycle modelling. The data fields in the inventory table are all relatively easily completed using a single site visit with visual inspection. Data fields not measurable at a site visit such as traffic count, existence of a flange plate, or pedestrian density may be conservatively estimated in order to provide a worst-case scenario until more detailed knowledge is obtained.

Inspection and testing

Following inventory, inspection and testing data is the next critical dataset required to produce meaningful financial models. An understanding of existing asset condition and vulnerabilities is key to planning maintenance intervention. Modelling can either track the condition of assets for a fixed financial budget or can determine the budget required to achieve a defined condition for the stock of assets.

The overall purpose of an inspection and testing regime is to:

- check that the physical highway assets are safe for use and fit for purpose;
- provide the data required to support the good management practices identified in the codes of practice;
- meet the requirements of the asset owner’s maintenance strategy.

Inspection and testing are discussed in more detail in Sections 7 and 8, with an example given of a regime which may be employed.

Using the equipment manufacturer's data as a baseline, periodic testing provides an understanding of the actual deterioration rate of an asset, with trends emerging for individual assets or groups of assets that are subject to localised conditions.

Modelling to understand overall asset stock deterioration leading to financial planning is possible without test data by using industry-accepted deterioration rates – if basic data exists such as asset installation date, material type, geometry, and corrosion protection system(s) applied – coupled with the inventory data fields.

Following the identification of a defect by inspection, testing generally arises from the need to investigate:
- The severity and extent of a defect or damage;
- The cause of deterioration;
- The rate of deterioration;
- The type of maintenance required and the extent of any repairs;
- Material characteristics;
- Data for a structural assessment.

Types of inspection (risk-based strategy)

Inspection types vary from general visual inspection to specialist structural inspection. The application of such inspection is discussed in Sections 7 and 8.

Safety inspections predominantly focus on ensuring the asset is safe and serviceable, whereas condition inspections will look at the long-term condition of the asset. The condition inspection will identify defects that require monitoring, rescheduling of inspection frequencies as necessary and inspection gathering of data and evidence.

It is recommended that visual inspection be carried out on a regular basis throughout the life of an asset. The visual inspection is carried out by a competent person assessing the severity and extent of defects against a fixed list of data fields. The general visual inspection is an opportunity to positively confirm whether a more specialist inspection is required using a risk-based approach, ie specialist inspection is used on a need-only basis.

Requesting a specialist inspection, based on the recommendation of a visual inspection, is considered additional to the planned testing regime an asset owner may adopt. They are ad hoc and localised to a particular asset or assets.

What test to use?

Inspection and testing is not a strategy, given that there exists a wide range of tests available all of which give a particular set of results. There is no single test that provides *all* the data required for development of a robust asset-management strategy. Accordingly, consideration is required in using the most appropriate test or tests at the appropriate column age. For assets, testing may be categorised as:
- Indicative tests carried out by competent asset inspectors;
- Strength/non-destructive tests (NDT);
- Destructive testing by dissection of material to understand strength and loss of section through corrosion following asset removal after, for example, accident damage or replacement as part of a renewal scheme.

Indicative tests are simple rules of thumb that can be used to check the condition of an asset and confirm or dismiss a concern identified during a visual inspection. For example, where corrosion loss is suspected at ground level of a lighting column, this may be qualitatively tested by tapping with a hammer to look for sections that easily dent or puncture due to corrosion loss. An experienced tapper may listen to the sound produced to deduce the relative condition. The use, by a competent person, of ultrasonic thickness meters on areas of the asset above ground level will provide a quantitative test of the same inspection observation.

Strength/non-destructive tests (tests that do not have the potential to damage the asset during the test) are more specialised and may include static load testing and/or eddy current testing, all of which are generally provided by test houses. All tests should be carried out by competent persons otherwise they may be of no value due to the failure to understand and control uncertainties such as the resulting repeatability or ability to reproduce results.

Any form of inspection and testing assists in managing risk, but adding quantitative testing to qualitative inspections can better assess the relative condition of assets.

This established approach allows the assessment costs and consequences to escalate in a measured way depending on the findings. A low-cost inspection may flag a defect that a medium-cost investigation can quantify. Then, if required, the column could be removed and a destructive test of the column carried out. Overall, this may have the same test costs as the NDT but the risk of the column failing can be better assessed to understand the extent of the problem, and therefore the need for wholesale replacement can be assessed.

Testing is discussed in detail in Section 7, and Section 8 and Appendix 1 discuss test methodologies currently available and the types of deterioration that can be detected.

How to use the data

Good asset-management practice relies on having and analysing data on the condition and performance of assets. A basic level of data obtained through visual inspection is a starting point to:

- Develop inspection and testing regimes;
- Determine levels of intervention;
- Predict investment programmes.

This will rely on using classic industry-accepted deterioration curves as shown in Appendix 6, to predict future performance, whereas more accurate data on the asset group, obtained through testing, can be used to migrate from basic reactive maintenance strategies, delivering poor value for money, to an enhanced risk-based approach using asset-management principles and tools.

Lighting asset owners have historically carried out inspections as guided by the Institution of Lighting Professionals' Technical Report 22 *Managing a Vital Asset: Lighting Supports* using the inspection pro forma SIF001. The data collected using SIF001 may be categorised as:

- Structural, for example shaft, bracket, base plate;
- Non-structural, for example luminaire, bowl/lens;
- Electrical, for example wiring, backboard, earthing.

The SIF001 form used a numbering system 1–4 to describe the severity of defects noted. The public lighting inspection pro forma develops recording of defects to capture both severity and extent (element condition) which are then used to better understand the condition and performance of the asset. As the dataset builds over successive inspections for a given asset, vulnerabilities become visible. This guidance deals with the use of the structural inspection data to develop asset-management tools such as Column Condition Index. Asset owners may develop wider condition-index tools such a Lighting Condition Index (LCI) using the non-structural inspection data gathered as discussed in Section 14.

Visual inspection data may be used as raw data within the following high-level strategic tools useful in developing, at a basic level, inspection and testing regimes, and maintenance and investment programmes:

- Column Condition Index (CCI): used to understand asset/stock condition.
- Deterioration Model (DM); used to determine the assumed residual life of a lighting asset to develop maintenance regimes and life-cycle plans.
- As previously stated the CCI and DM tools developed using visual inspection data rely on industry-accepted deterioration curves to determine asset performance. This may be considered a conservative approach.
- Data collected and held on SIF001 pro formas (as indicated in Appendix 3) is considered of value and can be converted to determine element condition using the conversions in Table 10.4 in Section 10. The converted data can then be used to develop an asset-level CCI or stock-level CCI.
- Test data provides additional information allowing refinement of the inspection, testing, maintenance and investment planning. Whereas visual inspection data relies on industry-accepted deterioration rates to determine asset performance test data, testing carried out at regular intervals identifies actual rates of deterioration on which life-cycle planning may be carried out.

Condition-index scores are simply values and need to be interpreted to describe the general condition of the lighting asset or stock. Recognisable terms to describe condition used in this guide are:

- Excellent (E)
- Good (G)
- Fair (F)
- Poor (P)

The asset stock level CCI may be used to inform testing on those assets in the Poor category for immediate attention in addition to those assets in the Fair category which will migrate into Poor through deterioration. The CCI is only an indicator of asset stock performance and is used to identify maintenance works.

Assumed residual life as determined through use of the deterioration model with test data is for financial planning purposes only and is not intended to be a replacement for structural testing

Need for common results

A column condition index and deterioration models are useful tools in developing targeted inspection, testing and maintenance regimes as previously discussed. To ensure comparability from asset owner to asset owner – irrespective of location, funding source, local environment and suchlike – a common approach is required.

Whilst visual inspection has always been harmonised across the lighting industry via use of common inspection pro formas, the results from test houses have not traditionally been harmonised. To enable asset owners to use the deterioration model and obtain comparable results across the industry and varying test providers, common data fields and results are necessary.

Guidance is given in Section 10 listing the data fields required and the dimensions (measurements) for each field. This will produce common assumed residual life data for assets across the industry.

Life-cycle planning

A life-cycle plan is a long-term strategy for managing an asset, or group of similar assets, with the aim of providing the required level of performance while minimising whole-life costs.

A life-cycle plan should give the optimal mix of inspection, testing, monitoring, maintenance, renewal and enhancement works that minimises the whole-life costs and meets the service requirements.

Good asset management uses life-cycle planning which calculates the total funding necessary to sustain the network over the complete life of the asset.
Life-cycle plans assist in:
- Identifying the required asset performance (ie condition, capacity, availability, standard);
- Identifying the deterioration mechanisms and establishing deterioration rates and residual lives based on engineering experience and judgement;
- Identifying work options and the associated costs (revenue and capital expenditure);
- Undertaking option appraisal to identify optimal solutions.

Guidance in respect of life-cycle planning is contained in the *Lifecycle Planning Toolkit* issued by HMEP in November 2012, which includes three toolkits based on the same principles that operate in the same manner. The *Ancillary Assets Toolkit* is aimed at providing users with planning-level decision support in the maintenance management of ancillary highway assets and is applicable to minor structures.

Risk assessment

Risk assessment and prioritisation is at the centre of the risk-based approach (RBA) using asset management principles. ISO 31000:2018 *Risk management* sets out three stages of risk assessment – identification, analysis and evaluation – which can be applied to the management of all assets. The approach, summarised in Section 12 of this document, is listed as:
- Identify the risk
- Analyse the risk
- Evaluate the risk

Ongoing inspection, testing and maintenance strategies stemming from initial inspection and testing is an asset owner's response to risks associated with the deterioration of assets. Current and predicted future risks can be assessed using predictive models discussed above. The *Well-managed Highway Infrastructure* code of practice emphasises the need for regular evidence-based reviews to form part of the asset owner's RBA. The application of safety inspections, testing and defect repair as well as recording and monitoring of information, are the most critical with respect to risk managing liability.

Whilst the principles of a risk-based approach may be common across all asset owners, the selected intervention, maintenance regimes and financial budget will vary depending on their appetite for risk. The level of risk that is acceptable may vary from industry to industry based on the consequences of failure. Within a single asset owner's stock, the acceptability of risk may vary within each asset group, based on location, traffic/pedestrian density, network hierarchy and suchlike. Whilst there are obvious boundaries to what is acceptable and not acceptable, there exists some latitude within which different asset owners will take differing approaches on when to intervene on a deteriorating asset. This is known as the principle of 'As Low As Reasonably Practicable' or ALARP.

Principles of risk assessment (ALARP)

The Health and Safety Executive has published guidance on ALARP, the origins of which are described in Section 15.

The ALARP or 'tolerable region' can be stratified based on an asset owner's assessment of the acceptability of the consequences of failure. See Figure 4.1.

This appetite for risk informs the inspection, testing and maintenance programmes.

Intervention and timescales

The timing for which intervention is deemed appropriate is informed by levels of service. Asset owners set their own levels of service, which are non-technical statements of service objectives or outcomes, easily understood by the public, reflecting the asset owner's priorities and interests. They are concerned with how the asset performs in supporting delivery of those outcomes, rather than in performance in a technical sense.

There are two types of level of service that are applied to achieve service objectives:

- *Short-term service levels* relating to the response times in respect of reactive maintenance (urgent and non-urgent defects), timescales for carrying out routine inspection and maintenance activities;
- *Long-term service levels* which relate to the overall condition of assets, and overall performance in delivering the objectives

The priorities of an asset owner may be performance driven for safety critical assets where the consequences of failure are considered high.

Lifecycle plans are formulated based on intervention (maintenance strategies) designed to deliver the accepted Levels of Service. Examples of maintenance strategies are:

- do minimum and service restrictions

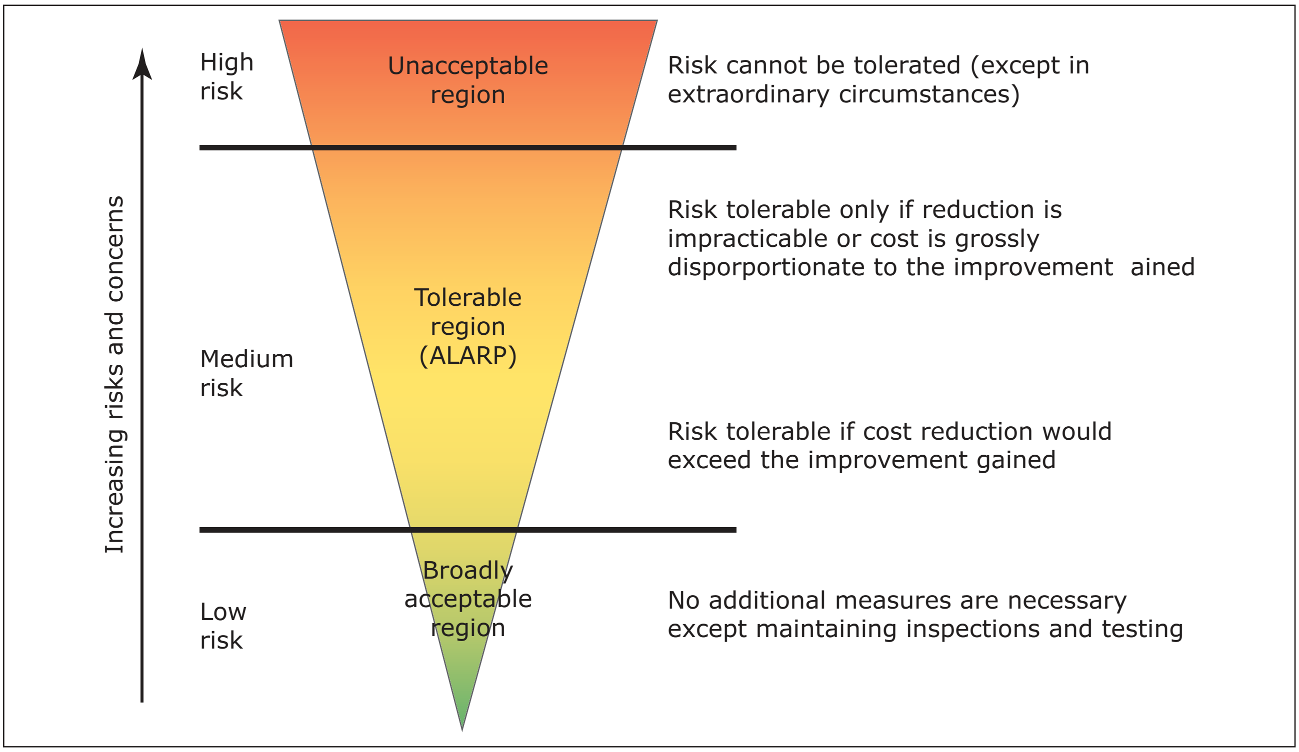


Figure 4.1: ALARP chart

    - for example undertake reactive maintenance only to keep assets at the operational threshold;
- corrective maintenance
    - for example undertake reactive maintenance reinstating deteriorated elements;
- time-based preventative maintenance
    - for example column replacement every say 30 years based upon manufacturers data;
- condition-based preventative maintenance
    - for example undertake maintenance when the condition falls to a defined threshold.

These are further discussed in Section 13 but an example of varying intervention strategies is shown in Figure 4.2.

The examples of varying intervention strategies in Figure 4.2 will produce different lifecycle plans, each with their own investment level and timeframe. This form of modelling is used to satisfy a range of constraints such as budgetary, political, social, and legal compliance.

Use of transition matrix E/G/F/P

The focus of asset management tools is to better understand the point at which lighting stock transitions from one condition category to the next, for example from excellent to good, good to fair and so on. The CCI for a lighting asset or stock is a number that itself means very little until translated into recognisable terms expressed as:

- Excellent (E)
- Good (G)
- Fair (F)
- Poor (P)

Given that a lighting asset comprises structural, non-structural and electrical components, weighting of the elements is applied, taking account of the importance of an element. An example of structural elements of a typical lighting column is shown in Figure 4.3. In simple terms the elements directly affecting public safety carry the highest weighting. Figure 4.4 and Table 4.1 describe the approach to weighting:

The structural elements described above are then given an element condition score (ECS) during visual inspections to describe the severity and extent of defects. The

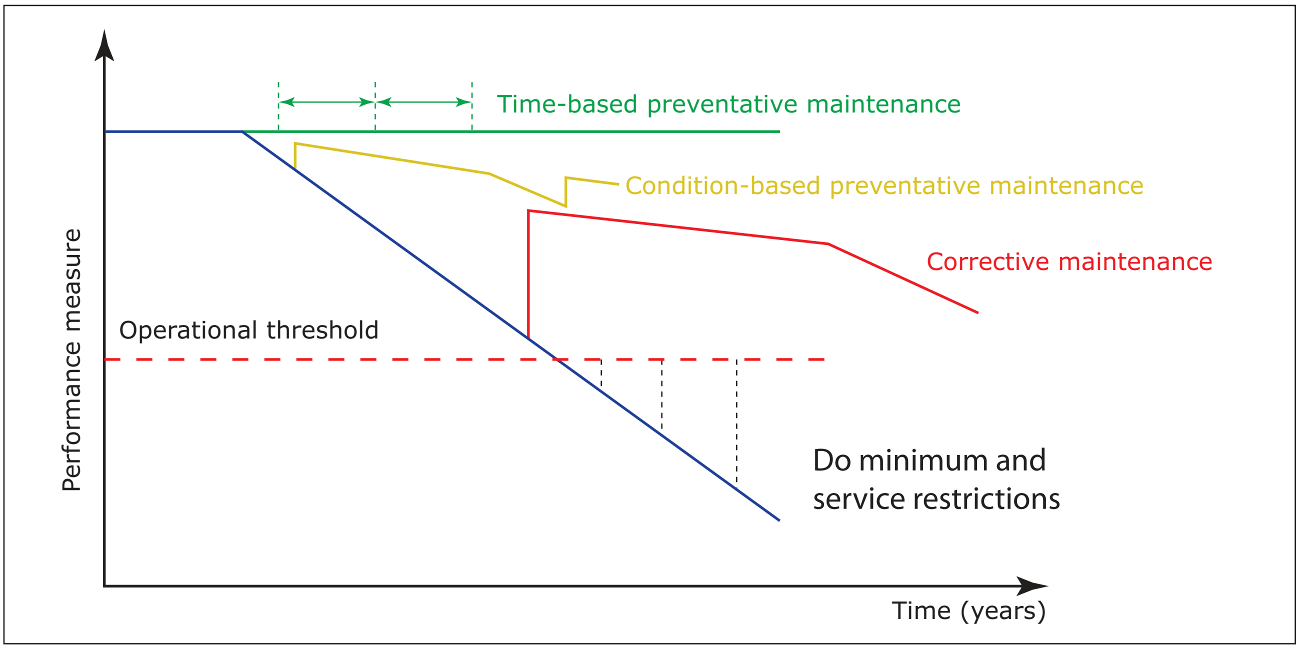


Figure 4.2: Intervention strategy effects

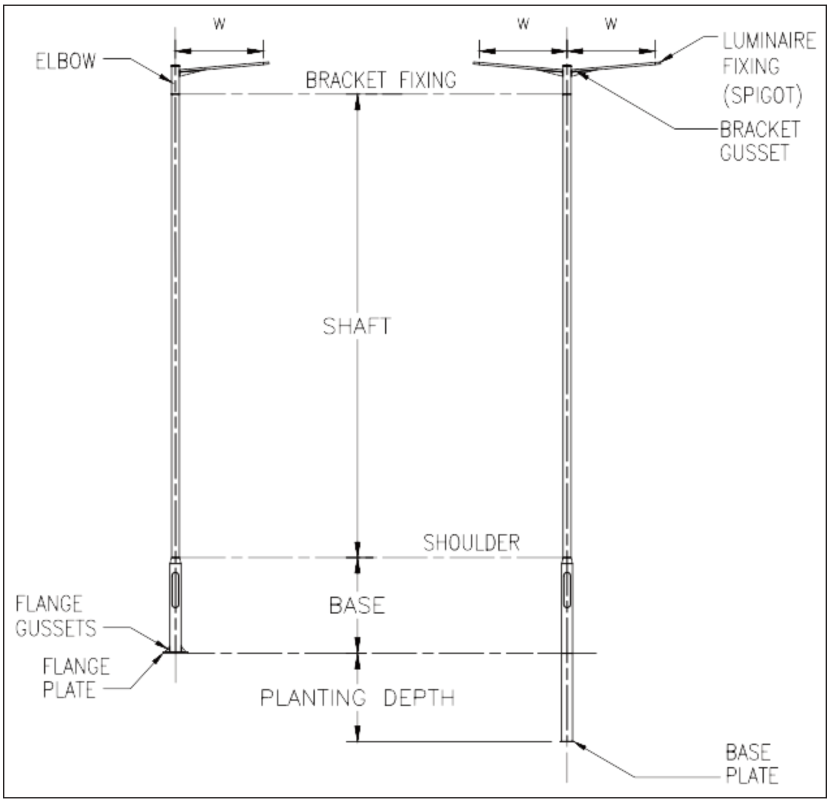


Figure 4.3: Column conditions locations

ECS is expressed as an alpha-numeric with extent ranging from A–E and severity ranging from 1–5 on the scale of 1 (best) and 5 (worst).

Consideration is then given to the element condition factor (ECF) taking account of the influence of the condition of an element on the overall lighting asset. The influence of a particular part of the lighting asset may be better understood as very high, high, medium or low.

The condition of an element (ECS) and its overall influence on the lighting asset (ECF) is mathematically developed to an element condition index (ECI) indicating the

For n elements:

$$CCI = \frac{100}{n} \times \Sigma_{i=1 \text{ to } n}[1-(\text{Defect Extent} \times \text{Defect Severity} \times \text{Element Importance})]$$

where

Defect extent is the proportion of the element affected from 0.00 (none) to 1.00 (all).

Defect severity is rated from 0.00 (not severe) to 1.00 (most severe).

Element importance is the probability the element failure would result in the asset failure from 0.00 (never) to 1.00 (always).

n is the number of elements making up the total asset

Equation 4.1

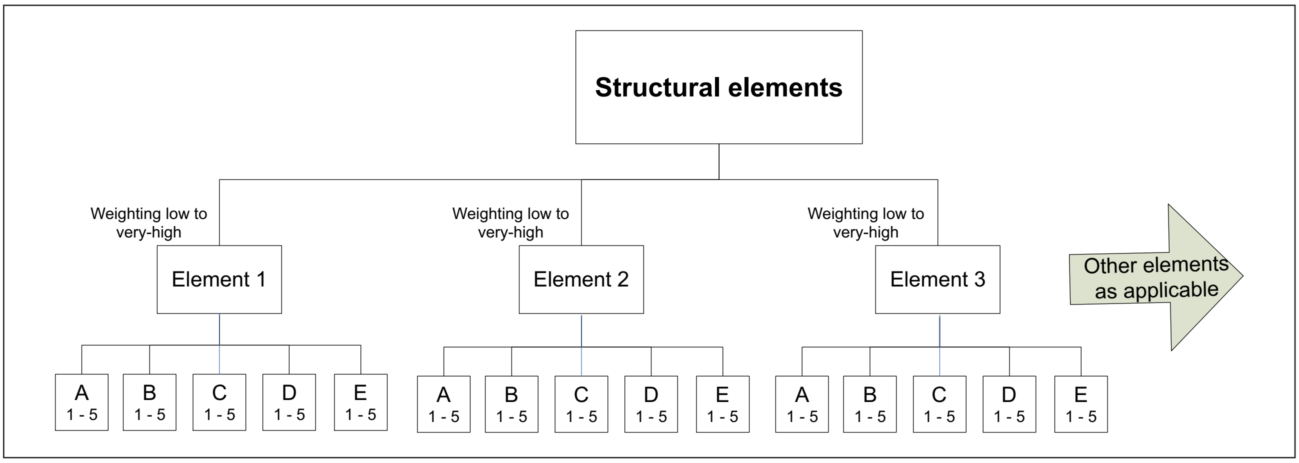


Figure 4.4: Structural element weighting chart

contribution of the condition of an element to the lighting asset as a whole.

The final stage of the process is understanding the element importance factor (EIF) used to weight the ECI against the functionality of the element, for example load carrying or durability.

All of the above processes are used mathematically to derive the CCI; the calculation is given in Equation 4.1 and a worked example is provided in Appendix 7.

Table 4.2 is then used to translate the CCI value.

The thresholds (CCI) given in table 4.2 may be used to develop life-cycle plans showing the distribution of lighting stock in each condition category (assuming no intervention).

Intervention strategies can then be imposed on the life-cycle plans to model the effects of investment. Changing the input data to the base level ie 'no intervention' life-cycle plan will produce strategies to suit either a budget-conscious or condition-sensitive life-cycle plan and consequent investment model. Input data to achieve this includes:

Table 4.1: Lighting support element importance

Type	No	Element	Importance
Column	1	Foundation	Very high
	2	Flange plate	High
	3	Flange gussets	Medium
	4	Holding-down bolts	Very high
	5	Column base	Very high
	6	Column shaft	Very high
	7	Door	Low
	8	Door surround	Medium
	9	Shoulder	High
	10	Bracing (including fixings)	High
	11	Elbow	High
Wall Mounted	12	Substrate	Very high
	13	Anchorage bolts	Very high
	14	Wall plate	High
	15	Gusset plates	Very high
	16	Bracketry	Very high
Catenary	17	Substrate	Very high
	18	Fixing bolts/eyes	Very high
	19	Catenary	Very high
	20	Wall fixing connecting luminaire to catenary	Very high

Table 4.2: CCI assessment table

CCI	Condition	Description
100–95	Excellent	The stock is in excellent condition. Very few assets may be in a fair to poor condition. The assets represent a very low risk to public safety.
94–85	Good	The stock is in a good condition. Some may be in a fair to poor condition. The assets represent a low risk to public safety.
84–65	Fair	The stock is in a fair condition. A significant proportion of the stock may be in poor condition. The assets represent a moderate risk to public safety.
64–0	Poor	The stock is in a poor condition. A substantial proportion of the stock is in poor condition. The assets represent a high risk to public safety.

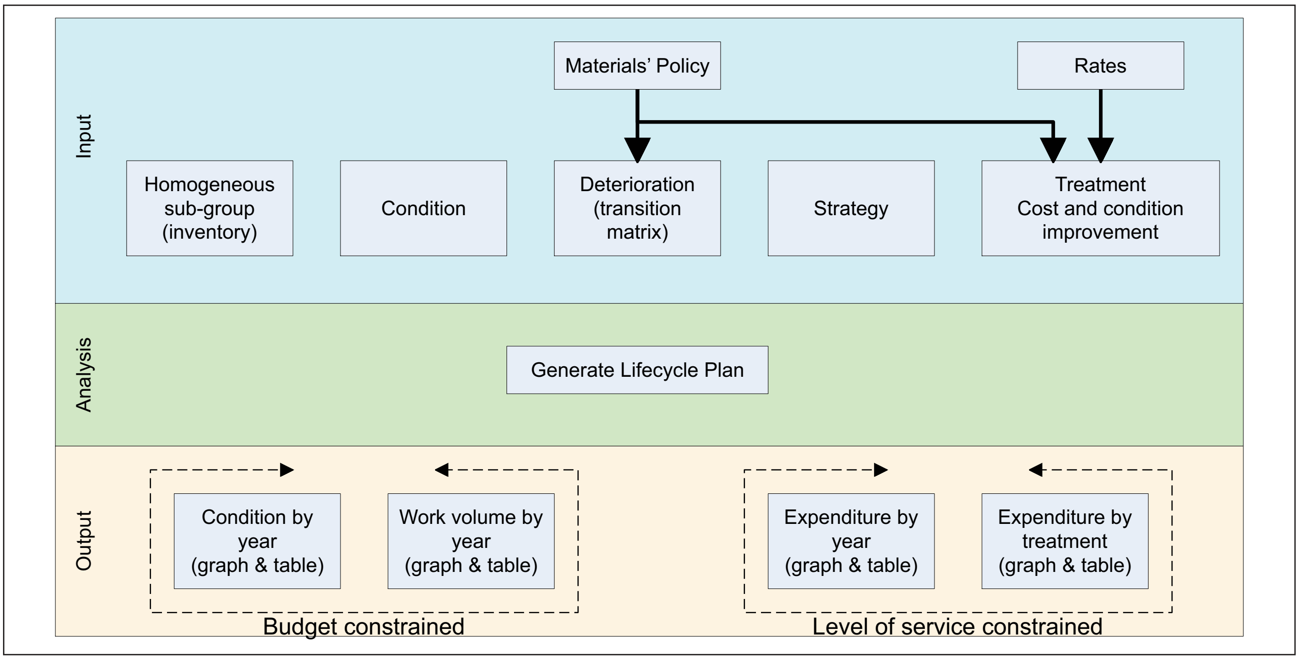


Figure 4.5: Lifecycle planning considerations

- Homogeneous asset groups – inventory data may be grouped into homogeneous asset groups and is loaded into the investment model.
- Condition – CCI condition data from visual inspections.
- Deterioration (transition matrix) – selection of the appropriate deterioration curve determines how much of the stock transitions from 'excellent' to 'good' and so on over time.
- Strategy – User-defined proportion of the assets to be treated each year. This may be driven by the user pre-determining the minimum condition of the stock or by expenditure of an annual budget.
- Treatment cost and condition improvement – inputting cost of treatment and the improvement the treatment provides determine the step up in condition achieved by carrying out treatments.

From this, the desired life-cycle plan is produced with the key outputs being either 'budget constrained' (and the improvement on asset stock condition that achieves) or 'level of service constrained' identifying the budget required to achieve that.

Figure 4.5 describes the process.

5. Luminaire supports as structures

Luminaire supports (columns, wall brackets and catenary systems) are *structural elements* that support a luminaire as well as other attachments, such that it can perform its task of lighting a space or area as required. This is an important change from the view previously taken by most lighting asset managers and perhaps a move away from how they have previously been considered and managed. This is not to say that they should not be managed by, for example an asset owner's lighting department but there may be a change in how their design is specified (see Appendix 2) and how they are managed throughout their operational life. It is important that this distinction is recognised and that the right competent professionals, as discussed in Section 4, are responsible for the management and assessment of these assets.

It is therefore may be appropriate for those responsible for lighting installations to arrange for competent civil/structural engineers to assist in the management of their lighting columns and other support structures.

This is a step change from how such structures have previously been considered and managed through, for example, an asset owner's lighting department.

Whilst this document provides guidance with regards to the structural integrity and assessment of minor structures, it does not convey structural/civil engineering competency to the reader. Instead, it sets out the issues, concerns and requirements to enable an understanding of the changes in structural condition throughout an asset's operational life.

As discussed in Section 3 we need to be mindful that in these days of litigation the competency of those assessing and taking responsibility for an asset may be called into question; the best defence is to ensure that the most appropriate competent professional has responsibility. It is important that this distinction is recognised and that appropriate competent professionals are responsible for the management, inspection and assessment of these assets.

6. Inventory

Understanding the current position

> *"If you can't measure it, you can't manage it"* – Mayor Bloomberg

In developing an efficient minor-structure maintenance strategy, it is important to establish and understand the baseline asset data. With this understanding, an asset owner will be able to carry out a systematic assessment of the current asset condition and will be able to develop, with more confidence, strategies to better understand the condition of luminaire supports over their operational life. In addition, based on this information one may apply suitable interventions to maximise the support's operational life, as well as plan future inspection and testing regimes and finally a replacement programme and associated budget.

The bedrock of this should be an asset database/inventory system. It is important that a register of lighting assets is maintained and the essential data is recorded, stored and kept up to date, as this drives the whole asset-management process.

An asset owner will need to ensure that that the data contained within the register is valid, up to date and maintained in such a way that it provides the information required to effectively manage the lighting service. This will provide the core data for the development and future performance measurement[1]. The information should not only relate to the lighting asset but also any maintenance operations that may form part of the asset decision or treatment process.

1. UK Roads Liaison Group; *Well-managed Highway Infrastructure, A Code of Practice*

For highway authorities, asset-management data and systems are dealt with in the UKRLG *Highway Infrastructure Asset Management Guidance* (HIAMG), Part B. The following is an synopsis from that guidance:

- *The information managed in the database or management system must be:*
  - *Relevant to support the required maintenance management decisions;*
  - *Readily accessible and in a format suitable for those who need to manage and evaluate maintenance strategies and practices;*
  - *Affordable and cost-effective so that regular collection and updating can be sustained; and*
  - *Reliable and adequately accurate for the intended purposes;*
- *Lighting asset management systems should provide and support the following list of functions:*
  - *Collection, storage and retrieval of inventory data and condition data;*
  - *Works management and prioritisation;*
  - *Production and reporting of national and local performance data;*
  - *Deterioration modelling and life cycle planning;*
  - *Management and storage, in electronic format, of drawings, photographs and reports;*
  - *Identify different cleaning intervals for installations with different conditions, luminaire types, environmental zones and luminaire IP ratings; and*
  - *Identify different routine maintenance intervals for installations with different conditions, including for example: luminaire types, light source type, optic configuration, LED driver or control gear type, environmental zones and luminaire and gear compartment*

ingress protection (IP) ratingsIt is vital that when a minor support is replaced that the asset is archived and a new asset generated with the year of installation appended.
Accurate recording of asset data, inspection records and maintenance activities is essential. A suitable monitoring regime should be in place to ensure good quality information is in use.
Asset data will also support the calculation of Gross Replacement Cost and Depreciated Replacement Cost for lighting associated with highway infrastructure, as required for Whole of Government Accounts.

The asset-management system should be kept up to date to ensure the currency of the data held, and responsibility for updates should be confirmed.

Appropriate data sets

Over the years, inventory structures have been published giving recommendations as to the numbers and types of data (fields) that should be held in an asset register. This guidance gives suggestions as to the minimum data fields that should be collected, including those needed to determine the LCI presented in this guidance. However, these could be added to or amended by individual asset owners to suit their specific service requirements.

To use the risk model described in Section 12, information on the data fields listed in Table 6.1 is required. The definitions for these factors will either be advised by a competent designer, identified locally, or obtained through the design and specification process such as PD 6547, which is advised in Appendix 2. The data should be extracted from an inventory database. Three types of field are required; numeric, text and flag fields. When data is entered in to the database, it is essential to limit the number of possible descriptions to those specified in the model; a series or drop-down pick lists would be appropriate for some fields. ‘Not known’ may be specified for certain fields. In this case, users of the risk model should be prompted for further information to ensure the risk model performs satisfactorily. If users do not know the year installed, they should be prompted to enter the earliest possible date otherwise the columns concerned must be assigned a priority for action.

If more than three data fields are ‘Not known’ the assumed residual life risk model becomes too crude to be of any use. The user should be prompted to reconsider the relevant fields as the risk model must assume the worst case. Any new data should be entered into the inventory database before priorities for testing are assigned by the risk model.

Table 6.1: Data fields analysis required for risk

Factor	Type of field	Data examples
Year installed	Numeric	Exact or approximate installation year [say +/- 5 years]; 0 for not known,
Material	Text	Concrete; steel; aluminium; cast iron; timber/engineered timber; stainless steel; other.
Material grade	Numeric	

Continued…

Table 6.1: Data fields analysis required for risk (continued)		
Factor	**Type of field**	**Data examples**
Column type	Text	Cohen (steel); Stewarts and Lloyds or older British Steel with leaking brackets (steel); Stanton and Staveley Types 1805 and 2005 (concrete); other steel; aluminium; concrete
Column height	Numeric	Metres; not known
Ground conditions	Text	Poorly drained, clay; moderately drained, loam; well drained, sand; not known
Salting of road or footway	Text	Precautionary; occasional; not salted; not known
External influences	Text	Terrain category (residential; other urban; rural; coastal; not known)
Environmental conditions	Text	As defined in PD 6547 table 2.6.3.4 (coastal; industrial pollution; inland, rural; not known)
Protective treatment	Text	External protective coating only; hot-dip galvanised; hot-dip galvanised and external protective coating; other; not known
Root protection	Text	No additional protection; bituminous coating; full depth concrete foundations; paint, thermoplastic, HDPE; none; not known
Wind exposure	Text	Exposed site; normal site; sheltered site; not known
Designed for fatigue	Flag	No; yes; not known
Designed for attachments or height	Flag	No; yes; not known
Approved attachment dimensions	Text/ numeric	Area, mounting height and off-set details
Existence of attachments or height extension	Flag	No; yes; not known
Flange plate	Text	Buried flange plate; plate exposed but no additional protection; plate exposed but with additional protection; none
Effect of location on probability of failure causing accident or injury	Text	Sited on central reserve; sited to side of road in pedestrian area; sited away from road but in pedestrian area; sited away from traffic and pedestrians; not known
Traffic flow	Text	High; moderate; low; not on a road; not known
Traffic speed	Numeric	Average speed in mph rounded to nearest mph; not known

Continued...

Table 6.1: Data fields analysis required for risk (continued)		
Factor	**Type of field**	**Data examples**
Location	Text	On a bridge over a trunk road or main line railway; on a bridge over a principal road or passenger railway; on a bridge over a minor road or minor railway; not on a bridge or on a bridge over open land; not known
Pedestrian density[2]	Text	Prestige walking zones; primary walking routes; secondary walking routes; link footways; local access footways; minor footways
Effect of failure: traffic disruption	Text	Major disruption; moderate disruption; minor disruption; not on a road or no disruption; not known
Inspection data	Text	
Test data	Text	

2. See *Well-managed Highway Infrastructure* Table 2.

7. Inspection

Developing a strategy for inspection and testing

To develop an inspection and testing strategy it is essential to identify the purpose of the inspection and what data is to be collected. Then there is the decision as to the frequency that such inspection and testing should be carried out and who should undertake this work.

Historically, local authorities in particular have carried out a routine visual inspection of a column either yearly or two yearly. This has been irrespective of the age of the asset and/or the condition of the asset. In addition, condition assessment may not have been considered as part of a routine, planned or reactive attendance at a lighting column. All persons undertaking any work to the minor structure or just inspecting the highway should have a level of relevant competence to undertake a visual inspection in accordance with their duties. For inspectors

- Is there any visible rust, weld cracks, impact dents or vandalism?
- Is the door cover off?
- Is the column leaning?
- Is the luminaire bowl hanging?

Those with more specific roles associated with the lighting asset should have a level of training such that they are able to assess the risk to the asset, and where concerns exist report them to the most appropriate professional person and request instructions before proceeding.

Within the motor vehicle industry, new vehicles are not required to have a MOT test until after three years. A similar approach can be applied to minor structures with older assets inspected and tested with a greater frequency than newer assets. However, particular frequencies should be determined on a risk basis taking into consideration the type of asset, location, age and the results from previous inspection and testing.

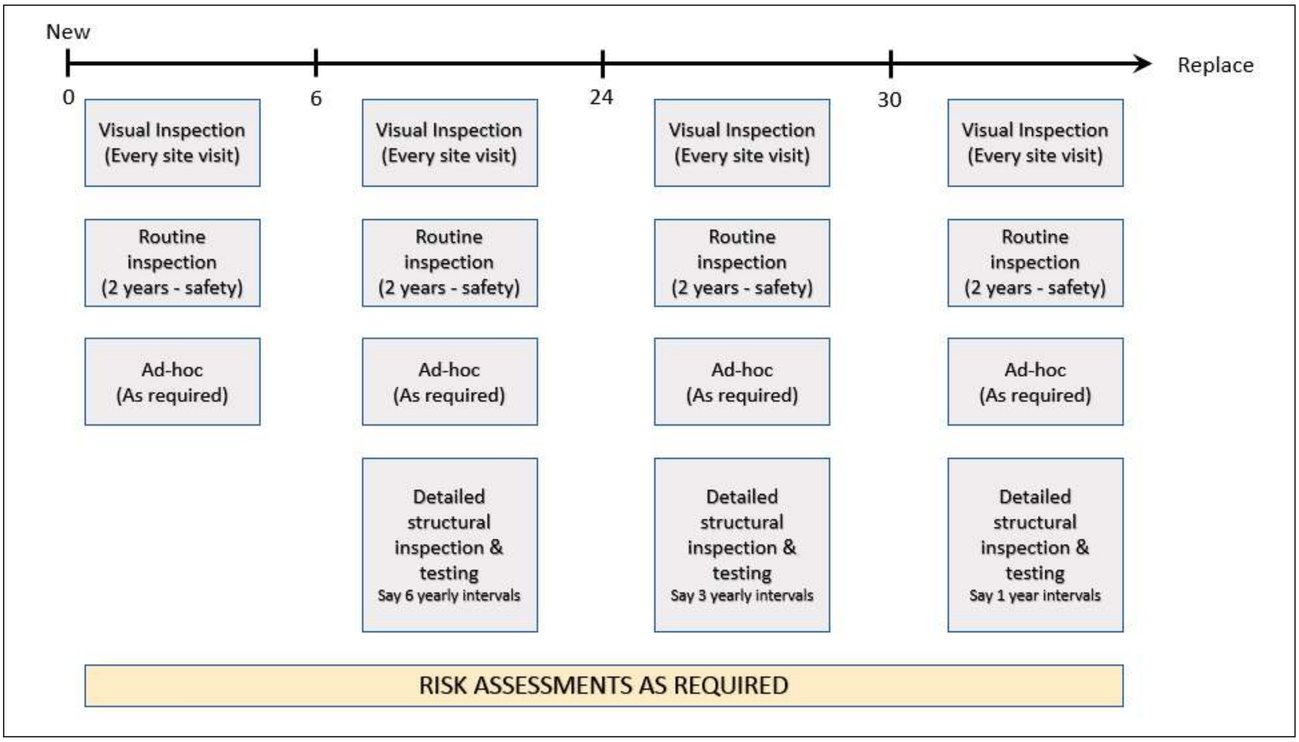


Figure 7.1: Example lighting column inspection regime

An example inspection and testing regime for lighting columns, on the assumption that columns have an assumed life of 40 years[3] to 'life-replacement', is given in Figure 7.1. This shows a visual inspection for every site visit. This is to identify any potential risks eg door missing, bracket arm displaced etc, which should be referred to the most appropriate qualified and/or experienced person to determine what action is required; for example, a structural problem must be referred to a structural engineer/technician.

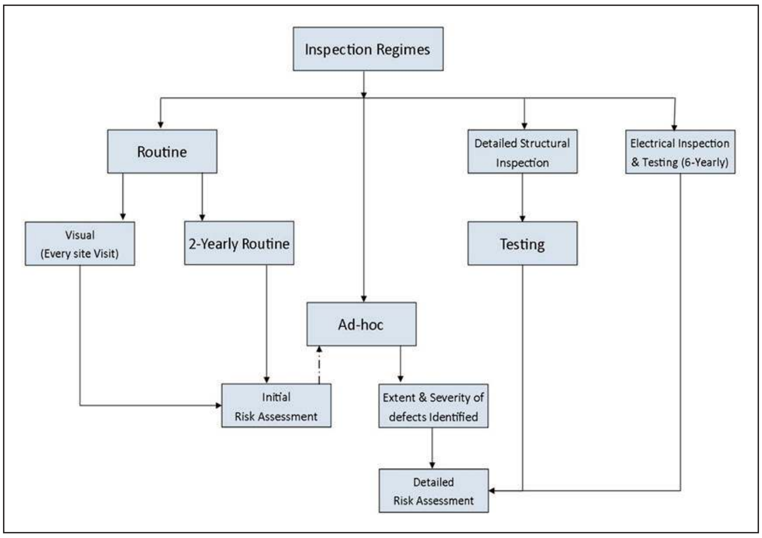


Figure 7.2: Risk assessment embedded within an inspection regime

In Figure 7.1 the routine inspections are safety related, and include the collection of data for the CLI. The detailed structural inspections would be undertaken at the same time as a routine inspection (to avoid duplicate site visits) and collect data including those from testing to enable a full structural assessment to be carried out. This would identify the assumed residual life and any data that can calibrate deterioration models.

For flange-base columns it is recommended by minor structure technical forums that after nine years the frequency of structural inspection is undertaken on a three-yearly cycle.

Figure 7.2 shows an example of how risk assessment is embedded within an inspection regime:

This example shows electrical inspection and testing at six yearly intervals. This should also be considered on a risk basis, whereby new installations could be inspected and tested at longer intervals and older installations inspected and tested at shorter intervals, taking into consideration type of equipment and the results of previous inspections and tests.

Appendix 3 to this guidance provides a standard list of items against which condition should be recorded to enable determination of CLIs and a determination of condition in terms of an easily understood scale of excellent/good/fair/poor. It also identifies those items that are structural, non-structural and electrical.

There are non-destructive tests available to assist in determining structural condition and assumed residual life. Details of these tests are given within Appendix 1.

Careful consideration should be given to determine the most appropriate test based on asset age and element condition, and from this the frequency of testing should be reviewed. This would be a factor in deciding what form of non-destructive testing would be appropriate.

3. BS EN 40 gives the *design working life* of lighting columns as 25 years. With good inspection and maintenance processes, lighting columns in less severe environmental locations, may have an *actual working life* of perhaps 40 years although in more severe locations it may be less than 25 years. For financial modelling of replacement the *estimated actual working life* should be used.

8. Testing methods

General

Testing minor structures can be an expensive and time-consuming process, particularly given the number of minor structures potentially involved. It is important that any testing is carried out as economically as possible. It is therefore essential that minor structures suspected of needing attention are targeted for inspection on a priority basis. This is the only way to ensure that limited budgets can be used to provide additional lighting or maintain existing lighting, rather than spending it on inspecting otherwise sound columns.

Every opportunity should be taken to record economically all available information on the existing minor structure stock in an area, including the original installation date and report dates (to assist in establishing deterioration rates). There are several opportunities for doing this provided there is a system to record received data on a regular basis. Minor structures are visited for routine scouting and/or maintenance requirements, to clean luminaires and replace lamps on a group change or on an 'as needed' basis. The opportunity to inspect and record the condition of a minor structure when visited for other purposes should not be overlooked as this can produce invaluable ongoing reference data.

With very little additional training of operatives carrying out scouting and maintenance operations, a basic inspection can be carried out. It should not be expected that electricians or maintenance personnel will have the expertise to judge in every case whether a minor structure is sound or not, but with assistance and training they should be able to carry out and complete a simple risk assessment that would highlight those minor structures which should be looked at more closely and those which can be left to a later date.

In addition to the planned and routine inspections, there are other opportunities for obtaining information on minor structure condition. Accidents cannot be eliminated and on occasions minor structures will be knocked down. All too often, these minor structures are simply placed in the skip and ignored, but a simple inspection of these – as well as minor structures removed prior to disposal – would give a lot of useful information, particularly on the condition of minor structure roots. The condition is very likely to be indicative of minor structures of a similar type and age in the immediate vicinity. Further opportunities may present themselves, for instance when minor structures have to be re-sited.

If all the information could be collected from such occurrences, there would be a considerable database of information on minor structure condition with little increase in expenditure. Such information would allow a review and selection of minor structures for detailed inspection and testing.

Visual inspection

- *Stage 1:* The initial visual inspection should be planned to identify areas requiring closer attention. Because these inspections will be frequent and regular, the personnel will often be those carrying out routine maintenance. The purpose of the inspection is to identify minor structures that have been hit, are leaning (possibly due to ground conditions with no physical sign of damage), or have suffered physical mechanical damage and deformation or cracking. Inspection will also identify the onset of areas of corrosion caused either by flaking paint or failed metal coatings, which could result in the onset of serious corrosion. Details of any attachments should also be recorded, checked, and images taken.

- *Stage 2:* Once problems have been identified, it may be necessary for a more detailed inspection of a targeted number of minor structures from the Stage 1 inspection reports. Visual inspection by an experienced inspector or engineer can determine more precisely the extent of corrosion and cracking, and through knowledge, the likely structural problems that should be considered for further investigation. Less accessible areas may be inspected by means of fibre optics using a borescope or video image scopes, some of which allow the recording of photographic images for reference.

Proposed inspection procedures

All maintenance operatives should be given basic training on identifying typical faults and assessing the condition of minor structures. During routine maintenance visits, operatives should be asked to complete an assessment and report on the condition of the minor structure prior to doing any work. In addition to the planning risk assessment and method statements, it is essential that a close visual inspection and dynamic risk assessment of the structural condition of the minor structure be carried out before using any minor structure as a support for a ladder[4]. Appendix 3 contains a typical Inspection Report with guidance notes for its completion; this is supported by Appendix 16 which provides condition examples to achieve a consistent approach on reporting of minor structure condition. Completed reports should be submitted to the organisation responsible for the condition of the minor structures, which should ensure that the data is stored on the asset database and used as part of the overall assessment of the condition of the minor structure stock.

On the Routine Minor Structure Inspection Report, it will be seen that, as well as identifying the type of column concerned, the operative is asked to comment on details using a severity scale of 1 to 5 observed over an extent scale of A to E. A1 implies that the item is visually in excellent condition and no action is necessary, whereas E5 implies that there appears to be a significant problem. It should not be implied that the operator is accepting any responsibility for structural decisions in this process. However, the operative has a responsibility to inspect and report any defects on any particular minor structure that, from his experience, is in a better or worse condition than others or is of concern.

The selection of severities ranging within 3 or 4 should trigger the consideration of a more detailed and informed inspection report. Severity 3 might be viewed in conjunction with the adjacent minor structures. Depending on the importance of the detail from a structural aspect, and if all the remaining minor structures of a similar type in the immediate area appear to be in better condition, it may be decided that no immediate action is required but periodic monitoring of the condition is scheduled. However, severity 4 identifies the need for a further inspection at the earliest opportunity. Special attention should be given to any modifications such as holes for electrical supplies and clamped-on equipment such as signs, baskets or ornamentation that are additional to the basic minor structure.

Severity 5 identifies a condition that the inspector should not walk away from and requires immediate attention/action. It is considered essential that the detailed minor structure inspection and report is

4. Comply with *Work at Height Regulations 2005* if planning work from a ladder.

undertaken and produced by a competent person familiar with problems that occur on lighting minor structures or with the specific type under consideration in the area.

It is not possible to give an assessment of the percentage loss of wall thickness through visual inspection. Non-destructive testing using a thickness meter would provide a section-loss measurement and then a structural calculation would give an accurate guide as to whether the minor structure was overloaded. However, based on current practice at the time of publication, a loss of more than 50% of thickness is likely to indicate that replacement of the minor structure is required in the near future.

A loss of 25% probably indicates that factors of safety on loads and materials have been reduced and the remaining material is only sufficient to carry the factored loads. Minor structures showing a loss of 25% of thickness should be regularly monitored over time and structural calculations completed, the outcome of which will determine whether the minor structure needs replacement or not.

Appendix 3 gives an example of a more detailed report, allowing more freedom for informed description of the item causing concern. Thickness measurements of corrosion protection may well be included in this report and average readings recorded, but there is room for recommendations for further testing if this is considered necessary. The detailed report should be carried out by personnel with appropriate experience and authority to be able to identify and recognise serious problems.

The detailed inspection report should also be completed on any minor structures returned to the depot following accidents, replacements or realignments. It is requested that reports with significant findings be shared with the Lighting Column Technical Forum (LCTF) to allow improvement of future advice[5]. Adequate information should be provided and recorded about the original minor structure location.

Where no information is available on the condition of column roots, it may be necessary to carry out sample excavations to assess the degree of corrosion. A typical minor structure in the area should be selected and an excavation 300 to 600mm deep made, ensuring that the minor structure does not become unstable. After inspection and non-destructive testing of the root, the ground should be reinstated to provide the same level of support and protection as before.

Adoption of systems of inspection together with any additional testing required could be used to provide evidence that a practical and responsible structural inspection programme was in use, giving protection to the maintaining authority in the event of litigation.

Suggested frequency of inspections

In the past, lighting columns have rarely been considered as ‘significant structures’ requiring formalised inspection procedures, as have bridges and more major structures. However, a lighting column can cause serious personal injury and even death, and if the responsible asset owner is found to be negligent in any way in the inspection and maintenance procedures, the potential damages can be extremely onerous. Personal liability also cannot be ruled out if negligence is proved against an organisation or individual.

5. lctf@theilp.org.uk

The following inspection programme is considered reasonable.

Routine column inspection

To be carried out on every occasion that the column is visited, but at least once in every two years.

Detailed inspection

As required following reports from the routine inspection the frequency of which is discussed in Section 7. The inspection/test intervals should be reduced as the columns age and begin to show signs of corrosion or other problems. Sample excavations to allow inspection of the root should be included if similar information is not available by other methods.

9. Testing

Background

The process of inspecting minor structures given in the previous section will inevitably have several phases, from with the routine visual inspection, to a more detailed inspection where problems are identified which may require testing. This section describes the most frequently used methods for inspecting and testing minor structures, but it is not intended to provide a full specification. A detailed discussion regarding a wide range of testing processes is contained within Appendix 1.

The purpose of any non-destructive test (NDT) method should be to determine whether a deteriorated column either satisfies the service criteria or is in danger of collapse. Whereas a visual inspection may sometimes be acceptable, in most cases this requires a structural analysis to compare the strength of the critical sections of the column with the bending and torsion moments induced by wind and dead loads. The characteristic strength of the column material is required to calculate the strength of the critical sections. Most steel columns are manufactured from three grades of steel, S235, S275 and S355. It will not be practical to obtain a physical sample from a column in use to have it tested. It would therefore be prudent to use the lowest-grade material characteristic in any desktop structural check if the material grade is unknown. If information is not available from suppliers, where possible the grade should be determined by analysing material samples, for example from a removed column.

It is important to get this information from the manufacturer in the first place and record it in the asset inventory for each column.

It should be the responsibility of the asset owner to specify the design and service criteria in accordance with PD 6547 and BS EN 40 as discussion in Appendix 2 and ensure that the appropriate fields exist within the inventory. The column design criteria can then be stored within the inventory and be used by the test house when estimating the induced turning moments. The moments are dependent on several factors, including the geometry of the base, the shaft and the bracket(s) of the column, the type of luminaire, and the size and position of any signs or other attachments. A different luminaire or a sign, banner or hanging basket may be included to allow for seasonal variations and/or a margin of safety.

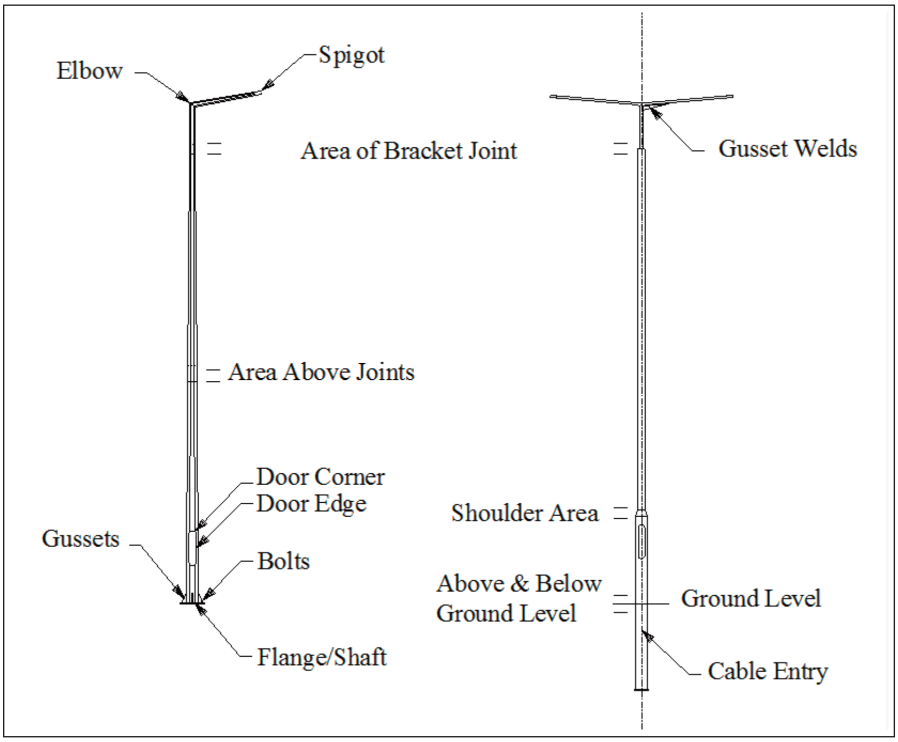


Figure 9.1: Main areas for inspection

The structural analysis must determine the minimum wall thicknesses that satisfy the service criteria. They may be carried out by a test house as part of the service they offer, or by the asset owner/local authority. Some minor structure suppliers offer this information. It is recommended that asset owners should specify this as a requirement on new service contracts.

Where a column is subject to cracking then the cost of monitoring, assessment and any remediation work is likely to exceed the cost of replacing the minor structure.

It is reported that some asset owners/local authorities assess the condition of columns based on percentage loss of section. It is not recommended to assess the structural suitability of columns based purely on the loss of section measured as this makes assumptions about the design loading and service criteria that may be invalid, particularly where a column has additional attachments.

Key requirements of all NDT methods are that they do not damage or reduce the service life of columns, and that false positives and false negatives of column condition are minimal.

Appropriate test methods

Two types of method are proposed for determining the condition of columns (Jordan and Vassie, 1999; 2000). They are:
- Indicative test methods;
- Quantitative/NDT test methods.

Based on current information at the time of publication the indicative and quantitative test methods that appear to be the most suitable are listed in Tables 9.1 and 9.2. However, other test methods are available and are reviewed in Appendix 1.

Indicative test methods give an indication that a column has deteriorated. They must:
- Detect defects quickly and at low cost;
- Be sufficiently accurate to detect defects that may cause a significant loss in strength of one or more critical sections;
- Identify significant defects that require columns to be repaired or replaced.

Indicative test methods are generally unable to determine the strength of the critical sections accurately.

Quantitative test methods can determine the strength of one or more critical sections accurately. One or more strength tests may be required to determine the condition of a column.

Ideally, only structural capacity tests would be employed, but when site access is physically or financially limited (for example on busy roads, rail station platforms or due to high traffic-management costs), asset owners may find it more cost effective to use a combination of indicative and quantitative tests, especially when the condition of the critical sections at risk can be assessed with sufficient accuracy by an indicative test such that quantitative tests are not required. For example, columns that are identified for testing may be visually inspected first. If the visual inspection finds them in poor condition, they may be considered for remediation, where practical, or replaced without further testing. If the visual inspection finds them in good condition, other indicative tests may be carried out. If these indicative tests find the columns in poor condition, again they may be repaired or replaced without further testing. However, all critical sections must be assessed by either indicative or quantative tests to

Table 9.1: Suitable indicative test methods	
Test	**Type of deterioration that can be detected**
Visual inspection	Presence of external corrosion and (significant) cracking at all locations; presence of internal corrosion with access through door opening
Aural test (tap test)	To an experienced operative, the tone/ring of the structure can indicate the presence of defects
Thickness monitoring	Loss of section near ground level and away from changes in section. Loss of section from the base of the door to below ground, and other locations such as the swage joint.

determine whether the columns are safe or unsafe.

For multi-stepped columns where each step is formed by a construction joint (rather than an extrusion) then each step is considered a critical section and will require inspection. This is likely to require suitable access equipment and associated costs. The asset owner would need to be aware of the potential for additional costs.

It may be considered that a critical section be repaired but such considerations should be assessed taking account of the whole column as well as the costs of the repair against the cost of replacement.

Tables 9.1 and 9.2 indicate that no single test method is suitable for all the significant defects listed in Appendices 12, 13, and 14 on all columns. Some test methods can monitor more defects than others but, at present, different test methods must normally be used to cover all the defects that may occur at the critical sections of columns.

However, certain defects are unlikely to occur or are insignificant for some columns, so the number of tests may be reduced if:

- The ground is well-drained and the root does not flood;
- There are no ungalvanised swaged joints, or corrosion at swaged joints is unlikely as the brackets prevent the ingress of water;
- Fatigue at welded joints is unlikely to occur.

Asset owners/local authorities have different views on when columns should be replaced or repaired. Although columns must be replaced when they are below strength, many asset owners replace columns that have serious defects that

Table 9.2: Suitable quantitative/NDT capacity test methods for steel columns		
Parameter measured	**Test method**	**Type of deterioration that can be detected**
Strength	Static loading	Loss of section and cracking from the connection between the base and the shaft to below ground level, although maybe not on all columns. Defective anchorages and holding-down bolts of flange-plated columns.
Section thickness	Ultrasonic testing	Loss of section at all locations; foundation anchorage cracking
	Loss of section monitoring	Loss of section at all locations
Crack length	Magnetic particle inspection	Cracking at door openings and welded joints
	Ultrasonic testing	Cracking at welded joints

may undermine public confidence, even if they have sufficient strength.

Additional test methods

The other test methods that have been considered are listed below and described in Appendix 1:

- Corrosion potential and current monitoring;
- Ultrasonic testing using guided waves;
- Eddy current testing;
- Penetrant dye inspection;
- Static loading with acoustic emission.

10. Common measurement matrix

Common measurement

Asset owners may employ a regime of inspection and testing to achieve a risk-based approach for management of minor structure assets. A difficulty arises due to the different methodologies employed when collaborating asset owners attempt to compare asset condition.

To enable a comparison to be made of asset condition, the appropriateness of strategies, the efficacy of maintenance regimes and suchlike, a common method of measurement, analysis and assessment is required.

The ability to compare asset stock condition across asset owners using a common approach has, in other disciplines, resulted in across-the-board learning, the ability for asset owners to form effective lobbying, and a simple, transparent method of determining investment level required to achieve a desired outcome. The reverse is also true insofar as one can determine the likely stock condition for a fixed investment level.

Table 10.1: Data fields and unit sets

ID	Field	Unit
1	Lighting column height	m
2	Projection length	m
3	Shaft circumference	mm
4	Base circumference (parent material)	mm
5	Base height	mm
6	Door opening width	mm
7	Door opening length/height	mm
8	Door bottom height (Bottom of door above ground level)	mm
9	Base parent material thickness	mm
10	Shaft thickness	mm
11	Door corner radius	mm
12	Luminaire 1 nominal height	m
13	Luminaire 2 nominal height	m

Requirements for deterioration modelling

Deterioration modelling, as discussed in Section 15, is the analysis of asset data to assess the assumed residual life of the structural elements within a given environment without maintenance intervention.

This is based on computational modelling for mild steel columns with a yield stress of S235 N/mm^2 as the majority of the national stock comprises mild steel. To avoid computational errors, fixed fields of data with prescribed units are advised in Table 10.1.

Requirements for column condition index

As previously stated, comparison of stock condition and associated targeted maintenance regimes across asset owners requires a consistent approach in the computation of a condition index.

Using the standard inspection form, condition is recorded in terms of the extent and severity of defects using the scales of:

- Severity: 1 (as new) to 5 (element is non-functional/failed);
- Extent: A (no significant defect) to E (extensive – more than 60% of the area/length)

It is essential that for the elements listed on the inspection form that there is a clear distinction between structural, non-structural and electrical elements.

The 'element score' is computed utilising a set table (see Table 10.2) taking account

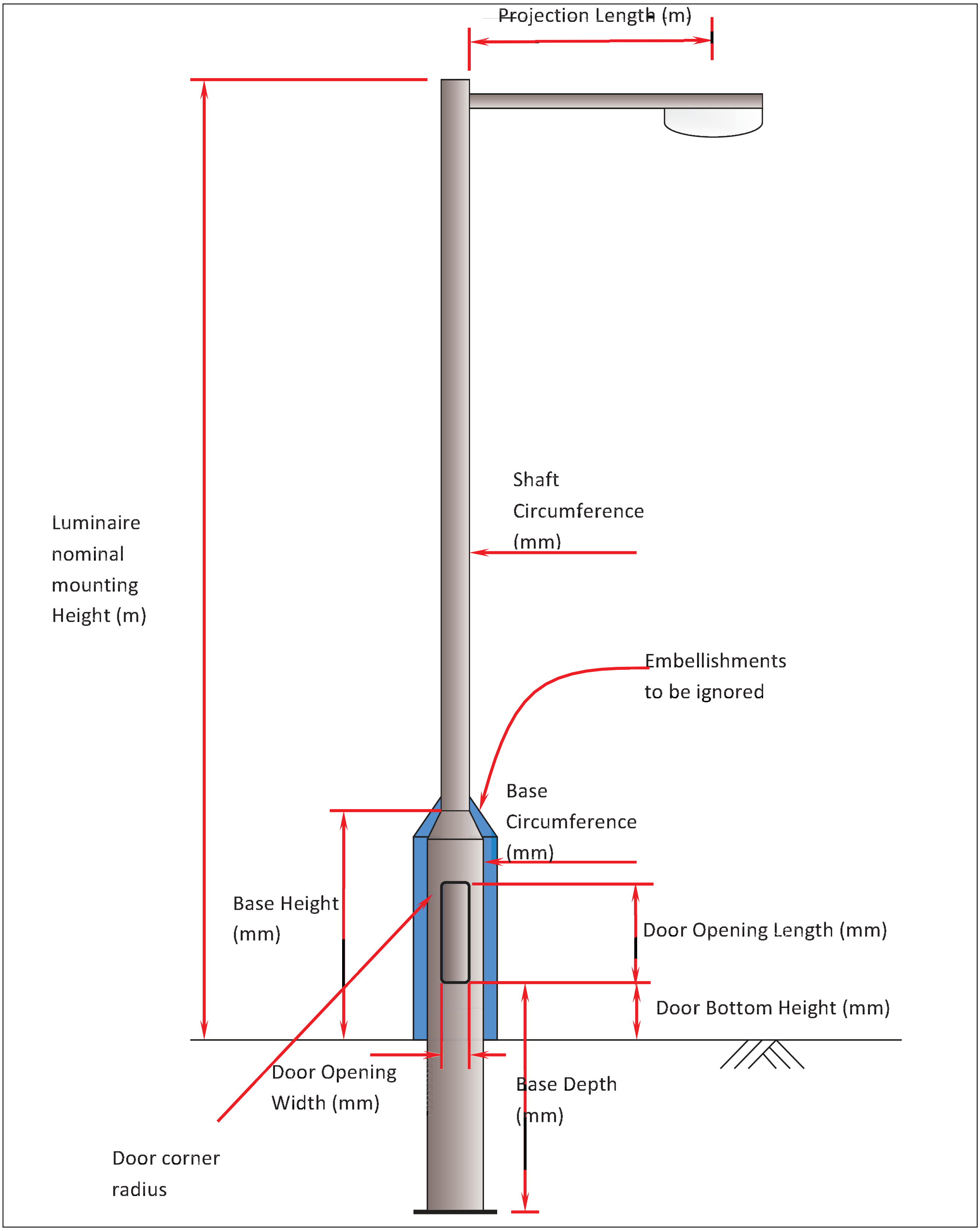


Figure 10.1: The locations for the inspection and testing.

of the non-linear deterioration of steel elements:

This guidance focuses on development of the CCI accounting for the structural elements of lighting assets as described in Section 4.0 of this guide.

Data required for computation of the CCI:

- *Visual inspection data* obtained during planned routine inspections recorded using the public lighting inspection pro-forma (see Appendix 3). Condition is assessed visually recording severity and extent of deterioration to visible elements of the asset.

Development of a more comprehensive light condition index (LCI) is not covered within the scope of this document and may incorporate the following additional inspection data.

Data required for computation of an LCI:
- *Structural:* Visual inspection data obtained during planned routine inspections recorded using the public lighting inspection pro-forma (see Appendix 3). Condition is assessed visually recording severity and extent of deterioration to visible parts of the asset
- *Non-structural:* Visual inspection data recording condition of non-structural elements eg lens/bowl, luminaire. Inspection records whether the elements are securely fixed. This is not intended to assess light output as it is solely condition based.
- *Electrical:* Visual inspection data recording the condition of the electrical elements, noting whether wiring appears satisfactory or if the back-board is not secure.

Computation of indices such as CCI and LCI are concerned with the visual condition of the asset. Other fields of data such as reliability, efficiency, public safety, social factors and suchlike are reserved for value management models as they are largely driven by local factors and needs.

Table 10.2: Element score tables

Extent	Severity				
	1 As new	2	3	4	5 Failed
A None	1.0				
B	1.0	2.0	3.0	4.0	5.0
C	1.1	2.1	3.1	4.1	
D	1.3	2.3	3.3	4.3	
E >60%	1.7	2.7	3.7	4.7	

Conversion of existing asset condition data

This guidance provides for conversion of inspection data recorded using the SIF001 pro formas as provided in the Institution of Lighting Professionals' Technical Report 22 – *Managing a Vital Asset: Lighting Supports*. The data collected using SIF001 may be categorised as:
- Structural, for example shaft, bracket, baseplate;
- Non-structural, for example luminaire, bowl/lens;
- Electrical, for example wiring, backboard, earthing.

The SIF001 form used a numbering system 1–4 to describe the severity of defects noted. The public lighting inspection pro forma, located in Appendix 3, develops recording of defects to capture both severity and extent (element condition) which are then used to better understand the condition and performance of the asset. As the dataset for a given asset builds over successive inspections vulnerabilities become visible. This guidance deals with the use of the structural inspection data to develop asset management tools such as CCI. Asset owners may develop wider condition index tools such as LCI using the non-structural and electrical inspection data gathered.

Visual inspection data may be used to develop the following high-level strategic tools useful in developing, at a basic level, inspection and testing regimes,

Table 10.3: Conversion table for assessments undertaken using TR22 form SIF/001 (See Appendix 3)							
Area	**Code**	**Inspection**	**No**	**Excellent 1**	**Good 2**	**Fair 3**	**Poor 4**
	a	Number of doors					
A	b	Flange plate condition		A1	B2	C3	E4
	c	Base compartment condition		A1	C2	E3	E5
	d	Door opening condition		A1	B2	E3	E5
	e	Base compartment shoulder condition		A1		B2	E5
	f	Internal compartment condition		A1	B2	D2	E4
B	g	Shaft		A1	C2	D2	E4
	h	Illegal attachment (yes/no)					
C	i	Bracket type					
	j	Bracket/shaft interface		A1	C2	D3	E4
	K	Elbow condition		A1	C2	D3	E5
	l	Elbow/web condition		A1	C2	D3	E4
E		Back board condition		A1	C2	E4	E5

maintenance and investment programmes:

- CCI used to understand stock condition.
- Deterioration model (DM) used to determine the assumed residual life of a lighting asset, to develop maintenance regimes and lifecycle plans.7
- As previously stated the CCI and DM tools developed using visual inspection data rely on industry-accepted deterioration curves to determine asset performance. This may be considered a conservative approach.
- Data collected and held on SIF001 pro formas (located in Appendix 3) is considered of value and can be converted to determine element condition using Table 10.3. The resulting converted data can then be used to develop a stock level CCI.

11. Recording and reporting requirements

The value of data recording and reporting

The implementation of asset-management principles for the management of minor structures is fully dependent on the regular and systematic collection of data. The value of reports produced from the data directly correlates with the quality and collection frequency of asset information on elements such as physical aspects of the minor structures, for example parent material thickness, condition of coating systems and suchlike, pass/fail data on electrical systems, column geometry, luminaire types and weights and attachments.

Whilst such systems of inspection and recording may be established, the quality of reports produced through data analysis can be compromised by inconsistent or subjective views from different inspectors. Minimising such differences through training, provision of images to normalise interpretation of deterioration, and other means remains as important an aspect of data recording as the data itself.

Methods of recording and reporting

Data recording and reporting may be split into three areas:

- Recording of visual inspection data;
- Recording of non-destructive testing (NDT) results;
- Third party reports from the general public.

Reliance on any one of the above-mentioned sources in isolation provides a limited view of stock condition which may lead to poor decision making with respect to maintenance regimes, stock replacement strategies and consequent budget allocations.

Historically, the practice for asset owners has been to engage external test houses to carry out NDT on minor structures to assess structural adequacy and certify for a fixed period. The information obtained from such tests may be further used to carry out computational modelling to determine rate of deterioration and, using structural analysis modelling, the asset's ability to accommodate additional loading such as heavier luminaires, banners, decorations, flower baskets, CCTV cameras and communications antennae.

Visual inspection of lighting assets tended to remain as an in-house service carried out by lighting inspectors. The data recorded in written format would then feed into maintenance programmes or ad hoc repair works.

Finally, third party defect reporting, such as calls to asset owner emergency call centres by members of the public, captured data on a reasonably limited number of elements such as day burners and light out. A review of the data from call centres suggests the priority given to the defects by call centres are higher than would possibly be the case if inspected by a local authority lighting inspector. Understandably, calls from members of the public do place a higher urgency on such defects than local authority risk registers might.

Recording of visual inspection data

Visual inspection of minor structures on a periodic basis has always formed part of the data stream used by asset owners to inform maintenance, asset replacement and work programmes.

The data is usually recorded using pro forma report sheets.

Recording of NDT data

Non-destructive testing carried out by test houses is a valuable source of data not only on the structural adequacy of an asset but also for assessing and understanding rates of deterioration. Additionally, when mapped with other data fields – for example the influence of ground substrate types, dog walking routes, de-icing grit-spreading frequency, etc – using geographical information systems (GIS), it can lead to understanding which assets may be subject to environmentally accelerated deterioration.

Data from test houses is usually collected electronically against each lighting asset reference. The data may be exported into other software to understand trends and asset condition.

Methods of NDT are explored in greater detail in Appendix 1 of this document.

Recording of third party data

Most asset owners record third-party data from members of the public using means such as website contact forms and call centres. In these cases, the data is manually entered, either by the public or the operative at the call-centre, into an electronic format which is then uploaded to a local authority management system. The system then generates an alert leading to inspection of the asset defect.

Data analysis and reporting

Data collection together with data analysis is a key part of good asset management.

Information is fundamental to the development of maintenance policies and the ability to communicate effectively with stakeholders.

Effective and sustainable management of that information – which comes from many sources – and the distribution of that information to stakeholders and network users is crucial.

A risk-based approach to lighting maintenance needs to be founded on information that is robust enough to enable decisions (for example on levels of service, inspection and testing timescales, programmes of maintenance and repair) to be taken and reviewed over time.

From time to time, government may require specific information to be reported, either to them or publicly, for example on asset-owners' websites, and the asset-owners' information systems should facilitate this.

When minor structures have reached the end of their service life and are replaced, the asset data should be archived and the installation date reset following installation of the new asset.

12. Risk assessment

The principles of risk assessment

ISO 31000: 2018 Risk Management – Guidelines sets out the principles of risk management and the organisational framework and process required to develop and implement a risk-based assessment (RBA). The risk-management process described within ISO 31000 is illustrated in Figure 12.1.

ISO 13100 sets out three stages of risk assessment:
- Identification;
- Analysis;
- Evaluation

which can be applied to the management of all assets. The approach is summarised in Table 12.1.

For a minor structure to be kept in service the risk associated with it should be within an acceptable range.

The current risk and predicted future risk can be assessed using the developed risk model. This model uses a definition of unacceptable risk where a column is considered at risk of failure and should be removed from service. Assumed residual life (ARL) is taken from the period of assessment to the point at which the risk of structural failure poses an unacceptable risk.

ARL is approached using well-understood structural engineering principles. Determination of a stock's ARL is further discussed in Chapter 15.

This information will assist asset managers to develop budgets, targeting those assets most in need.

The risk associated with a minor structure is calculated based on the probability of failure and the consequences of failure. Lighting minor structures are usually sited to light roads and public areas and so failure could cause a traffic accident, pedestrian injury or disrupt rail travel. Some minor structures are situated away from roads and are used to light industrial and utility areas or support transportation such as signals within the rail sector, where a minor structure failure could injure pedestrians or cause severe damage to a major infrastructure network. The consequences of a failure of minor structures near different infrastructure types means that each group of minor structures should be

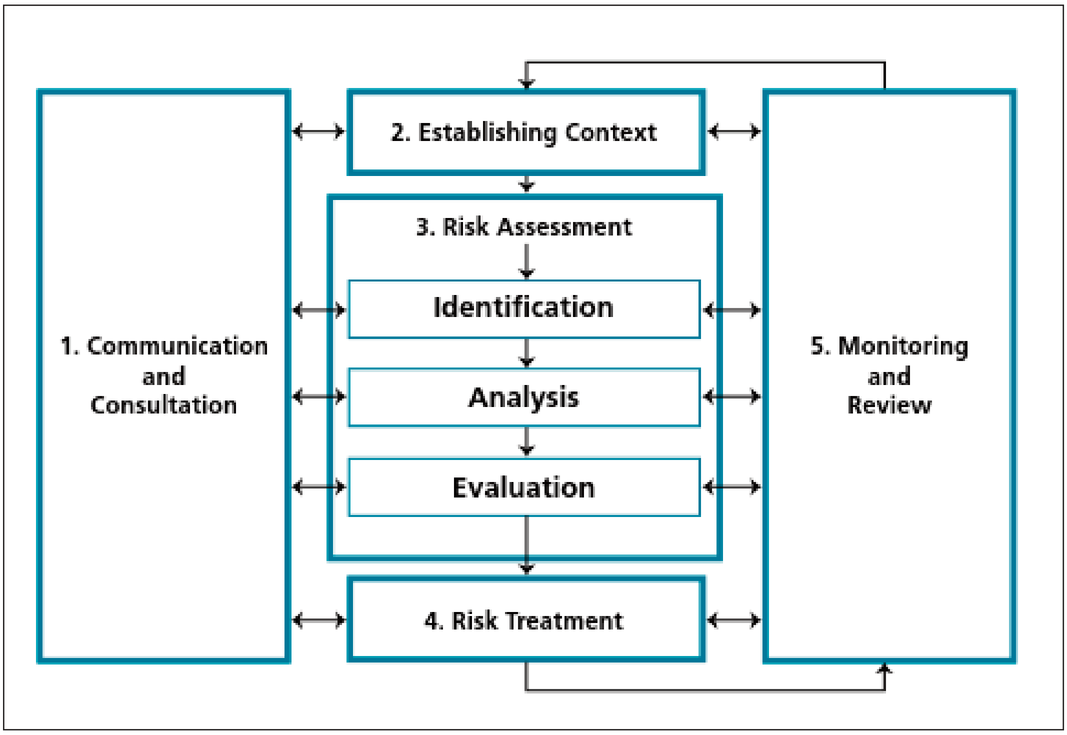


Figure 12.1: Risk management process

Table 12.1: Risk assessment approach

Heading	Heading
Identify risk	This stage involves identifying the circumstances that might result in liability claims. Examples include liability for exposure for: • Personal injury following defects • Property damage caused by defects
Analyse the risk	At this stage the risk is analysed to determine the potential consequences should it occur (that is, the reasonably foreseeable extent of injury or damage that might result) and the likelihood of it occurring. There are many sources of information that will assist asset owners to analyse consequence and likelihood of the various risks identified. These include but are not limited to: • Underlying condition surveys and information • Network hierarchy/type of road/area and usage • Number and scale of defects identified during inspection/testing • On-site assessments • History of complaints/claims • Criticality of area or network section
Evaluate the risk	The evaluation stage of risk assessment determines the asset-owner's response to risk. This will be based on the analysis undertaken and the asset-owner's appetite or tolerance for the scale of risk this reveals. This is the stage at which a decision is made on the course of action to be taken. It is frequently the case that risk registers and/or matrices using a RAG (Red/Amber/Green) assessment are used to plot and record the analysis of the risk and help risk evaluation and determination of actions to be taken.

considered separately and may not be comparable.

The probability of failure is calculated using the safety margin concept. As mentioned in Appendix 5 the applied loads and the load capacity of the minor structure are used to calculate its margin of safety.

The risk can be further managed by the application of suitable actions and activities that look to manage and mitigate the likelihood of the risk occurring such as:

- Routine inspections by competent operatives;
- Maintenance and remediation programmes that respond to the defects identified in a timely manner taking account of on-site judgement and specific local circumstances;
- Condition surveys and a programme of planned preventative maintenance.

Managing liability risk

The *Well-managed Highway Infrastructure* code of practice emphasises the need for regular evidence-based reviews to form part of the asset owner's risk-based approach. The application of safety inspections, testing, and defect repair, as well as recording and monitoring of information, are the most critical with respect to liability risk management. Key aspects of this include:

- Understanding the legal basis of liability, is discussed in Section 3 and in detail within the Chartered Institution of Highways Transportation *Well Managed Highway Liability Risk*. If it comes to legal action, a minor structure asset

owner will need to demonstrate that they have taken reasonable care to identify and respond appropriately to a defect that presents a danger;
- Establish the frequency of safety inspections and testing. This guidance advises on how such an approach can be developed by the asset owner to best suit their own circumstances and environments;
- The method of inspection and testing, including what to inspect, ensuring that there is a consistent level of quality and that those undertaking such assessments are trained, qualified and competent as discussed in Section 7.

Risk analysis

Columns can fail when the loading effects due to wind and dead weight exceed their strength. The strength is reduced by material degradation caused by either corrosion or fatigue cracking. The age at which a minor structure fails is dependent on a range of factors relating to the loads and its strength.

The time for corrosion to start is dependent on the protective treatment and factors that affect the condition of the treatment. The time for corrosion to weaken the minor structure is dependent on the rate of corrosion and any excess wall thickness at the corrosion site. For example, for a tubular column with an unreinforced door opening, unless the column has been over-designed, relatively little corrosion need occur at the base of the door opening or at the base of the shaft before the column is below strength.

The rate of minor structure parent-material corrosion is dependent on similar factors to those that affect the rate of deterioration of the protective treatment, although some factors have relatively more effect on the rate of corrosion than the rate of deterioration and vice versa. Acidic soils may cause increased corrosion rates once corrosion protection systems have failed. High structural stresses at the corrosion location can accelerate corrosion rates relative to less stressed positions.

There are a range of other corrosion processes that are not frequently observed but may be a root cause of corrosion in some very isolated cases. These include:
- Induced currents from power cables or impressed currents from earth faults in the minor structure cabling;
- Galvanic or sacrificial corrosion, particularly of aluminium minor structures close to buried cast iron pipes in salt laden soils;
- Microbial induced corrosion; soils containing organic pollutants, sulphate-reducing bacteria and sulphate-oxidising bacteria can create a cathodic corrosion cell causing localised deterioration of galvanised steel or concrete.

Failure due to fatigue

The age at which minor structures fail due to fatigue cracking is dependent on the number and amplitude of the stresses induced by wind loading. Fatigue damage can be induced immediately after installation, and the amount of damage induced by each stress cycle is dependent on the amplitude of the stress range raised to a specific power; the power is a factor of three for welded joints and four for base metal with stress concentrations. Hence most damage is induced in high winds.

A fatigue crack may not be initiated before 80% or more of the fatigue life and, clearly, the crack must be a reasonable length, ie 1–2mm before it can be detected by ultrasonic testing, or longer before it can be detected by static loading. Therefore, if a column is susceptible to infrequent high winds, it may be difficult to detect fatigue cracking before the column fails, and the minor structure may fail in a short time in very high winds, a far shorter time than the time between inspection periods. For this reason, there is less benefit in testing to detect cracking than testing to detect corrosion, and minor structures with fatigue cracks should be assessed.

Age when there is a likelihood of corrosion or fatigue

Unfortunately, there is a lack of detailed knowledge regarding how the protective treatment and material of a minor structure within current asset stock deteriorate and affect strength. This is a problem experienced generally in structural engineering, and results primarily from the non-uniform and localised nature of deterioration. As a result of these difficulties, an approach was adopted based on estimating the ages when there is a likelihood of corrosion or fatigue under the most and the least favourable conditions for specific column types. Within this age range, formulae were developed to take account of other relevant factors.

The basic information used in the model regarding expected deterioration and the relative effects of various factors on the deterioration is based on information gathered from other experts in the field, and data received from a local authority assisting trialling this approach.

Known issues

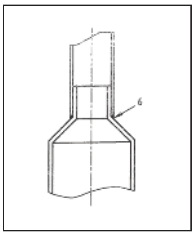


Figure 12.2: Type 6 swage joint

Certain features such as right-angled door openings and a type 6 swage joint (Figure 12.2), as per EN30-3-3 joints found on old un-galvanised British Steel/Corus columns, have resulted in rapid deterioration and occasionally structural failure. It is recommended that these columns are given a 'Highest' priority for action.

All columns types with known issues should be given a high priority.

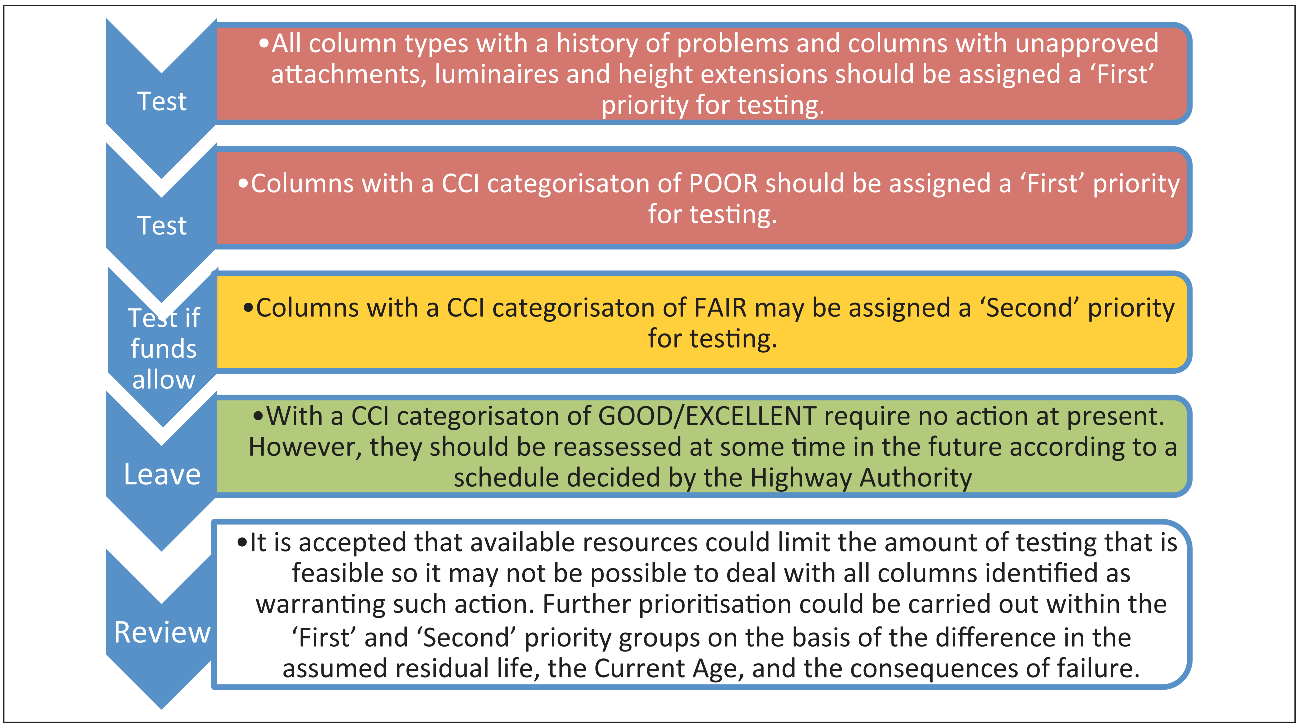


Figure 12.3: Test procedure prioritisation

Any column with an unapproved attachment, luminaire or height extension for which the column is not designed must be given a 'First' priority for action.

All other columns are prioritised on the basis of the risk model as a 'First' and, possibly, 'Second' priority for action, or reassessment according to a schedule decided by the asset owner.

The steps in the procedure are summarised in Figure 12.3.

It is important that clear and unambiguous policies are adopted for deciding when the appropriate action is carried out. Testing can range from 'indicative' tests where appropriate, for example visual inspection and loss of section monitoring, to more involved 'strength' tests, for example static loading (Jordan, Vassie, McKenzie and Ofori-Darko, 2001).

Columns with unapproved attachments, luminaires and height extensions do not require urgent action if it can be shown that the design of the column satisfies the service criteria. For this, the original thickness of the column sections (base and shaft), away from areas where there may be corrosion or cracking, can be measured using, say, ultrasonic testing. The bending and torsional strength of the critical sections, and the wind and dead loads can be calculated to determine whether the ultimate limit state requirements are satisfied. If the requirements are satisfied, the attachments, luminaires and height extensions assumed in the analysis become approved and the column may be prioritised on the results of the risk assessment model. However, other details of the design of the column such as joints and connections may also need to be considered before approval.

A column whose attachments, luminaires and height extensions are approved as described above may be assigned a 'First' priority for action when it is assessed by the risk model. Then, further testing may be the recommended course of action, so the column could be tested twice. To prevent the duplication of testing when a column has unapproved attachments etc, the following options are available:

- The column could be tested fully to determine whether it can withstand the design loading (that is, the effect of corrosion is assessed as well as possible low wall thickness).
- If the assumed residual life calculated by the model is similar to or less than the current age of the column, the column should be tested fully. If the assumed residual life is greater than the current age, only testing to determine if the original thickness of the column is high enough to withstand the design loading needs to be carried out.

Condition indicators

Having a condition indicator for minor structures allows an assessment of the overall condition of the stock. Over time, changes in condition can be monitored and assessed by having a condition index thereby allowing strategic decisions to be made on improving the performance of an asset owners lighting assets.

The objectives of a condition index are:

- To provide beneficial and meaningful data for minor structure asset owners.
- To measure and monitor the condition of individual minor structures and stock assets.
- To have the ability to measure lighting performance at a more granular level if so desired, such as at street/town/district level.
- To develop consistent and harmonised visual inspection reporting.
- To provide guidance on the interpretation and application of condition indicators.

Condition indexation is split into two levels:

- *Column Condition Index (CCI)* – considers the condition of the structural elements of the column/catenary system as critical.
- *Lighting Condition Index (LCI)* – includes the CCI with additional non-structural and electrical parameters, obtaining a condition for the whole lighting asset.

Calculating a column condition indicator for a column/catenary/wall bracket support involves a visual inspection of the lighting unit using a standardised method of collecting defect data noted during the inspection. By collecting the defect data from visual inspections and scoring against an established range of severity and extent of defects the data can be translated into a column condition index (CCI).

The CCI is a qualitative measure of stock condition used to develop financial strategies for future maintenance and investment. The data collected is considered objective and therefore comparable from location to location and asset owner to asset owner.

It should be noted that CCI and LCI are condition- and performance-tracking tools intended to enable owners of lighting assets to make forward investment planning decisions and replacement programmes on aged assets; it is *not* intended to be a structural assessment tool.

Gathering inspection data – using the pro forma

As already mentioned, CCI relies on a visual inspection of support structures to identify visible defects. It is not within the remit of the CCI guidance to dictate inspection frequencies. Individual asset owers must determine their own inspection frequencies based on local knowledge and needs. This approach is consistent with the spirit of the national code of practice – *Well-managed Highway Infrastructure* – which advocates a risk-based approach to the inspection of highway assets.

Defect data from the visual inspection should be collected using the pro forma developed as part of the CCI guidance. The pro forma identifies three different minor structure types:

- Column/mast
- Surface mounted, for example wall mounted
- Catenary

The completed pro forma provides a record of the visual condition of any given lighting unit on a given date by recoding the severity and extent of any defects noted during the inspection, for example corrosion of base metal, spalling of concrete, breakdown of protective coating etc. The pro forma particularly seeks to understand the condition of vulnerable areas, for example the base of a column, swage joint, supporting bracket, and door opening.

The severity of a defect is measured on a scale of 1 (as new) to 5 (element is non-functional/failed). The extent of a defect is measured from A (no significant defect) to E (extensive – more than 60% of the area/length).

In addition to recording the defects at element locations of the different lighting types, the pro forma also seeks to gather information on the condition of any protective coatings and the condition of any attached ancillary items such as CCTV cameras, pollution monitors, traffic monitors etc to ensure these are also in satisfactory condition and do not pose a risk to the public.

It is recommended that during an inspection visit any attachments are photographed and the image attached to the inspection report so that they can be compared to previous visits and a record of all attachments maintained.

Such photographic documentation should be taken during the day, not direct into sunlight, and include but not be limited to aspects shown in Figure 12.4.

It is essential that those persons responsible for undertaking the visual inspections of individual lighting units and recording the defect information understand the impact a defect may be having on the structural integrity of the physical asset. They must therefore have the appropriate skills and competency. It is therefore recommended that:

An inspector has a basic understanding of structural mechanics with the ability to identify when defects potentially present a risk to the stability of a lighting unit; bridge inspectors within a local authority typically have this depth of knowledge and competency.

Similarly:

It is also essential that an appropriately qualified, experienced and competent chartered or incorporated civil or structural engineer signs off the pro forma as the approver.

An overview of calculating an LCI is given in Appendix 4.

Reporting and strategic planning

Consistent reporting on the condition of minor-structure stock is achieved using an index-based approach. Although this approach is reliant on industry-accepted deterioration curves and assumptions, development of the CCI for minor-structure stock ensures not only that the stock condition is understood but also that the rate of deterioration and transition of stock from one condition category to the next is identified, quantified and reportable.

Using test data and the deterioration model (DM) approach to more accurately assess the assumed residual life of minor structure asset results in more prudent use of resources as the deterioration is

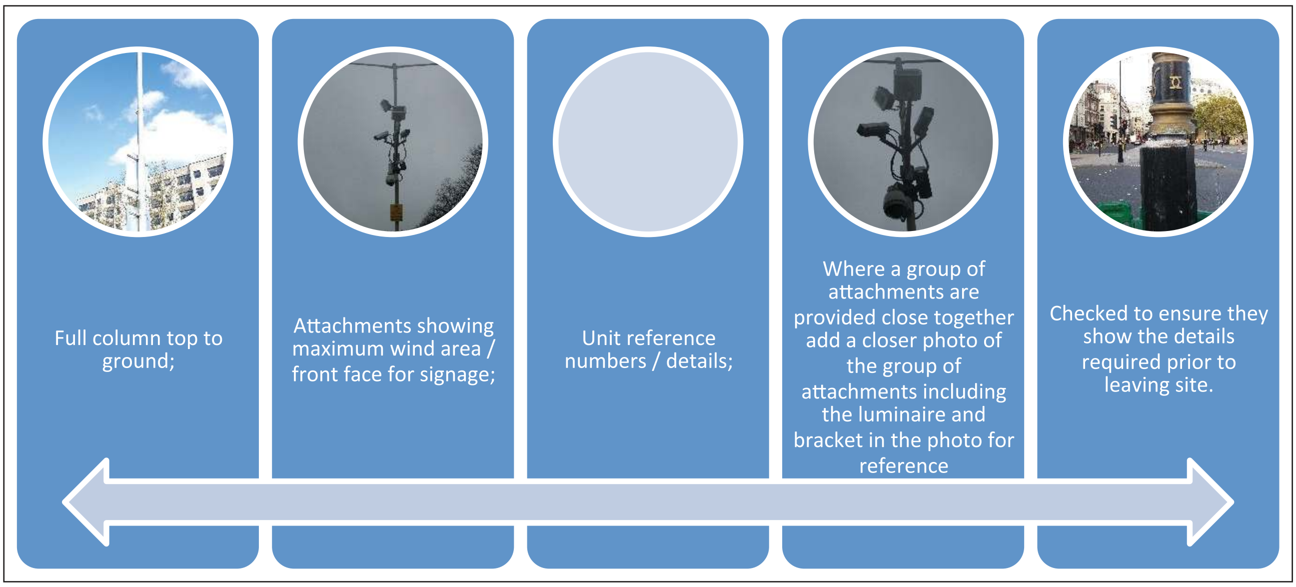


Figure 12.4: Evidential image criteria

based on actual data rather than industry assumptions.

Using either of the above approaches allows production of life-cycle plans which, when superimposed on intervention strategies, can produce a variety of investment models to suit varying criteria such as budgetary, political and social.

13. Intervention considerations and timescales

Levels of service

All local authorities set levels of service, which are non-technical statements of service objectives or outcomes, which can be easily understood by customers, and which reflect their priorities and interests. They are concerned with how the asset performs in supporting delivery of those outcomes, rather than in performance in a technical sense.

There are two types of level of service that are applied to achieve service objectives:

- Short-term service levels
  - relating to the response times for reactive maintenance (urgent and non-urgent defects), timescales for carrying out routine inspection and maintenance activities.
- Long-term service levels
  - which relate to the overall condition of assets, and overall performance in delivering the objectives.

Life-cycle planning

A life-cycle plan is a long-term strategy for managing an asset, or group of similar assets, with the aim of providing the required level of performance while minimising whole-life costs.

A life-cycle plan should give the optimal mix of inspection, testing, monitoring, maintenance, renewal and enhancement works that minimises the whole-life costs and meets the service requirements.

To meet the long-term service levels a life-cycle plan needs to be produced. This will need to consider the current state/condition of the asset and the required state/condition of the asset at a point in time in the future. A gap analysis should be undertaken to determine robust forward projections about maintenance and expenditure. It is essential to have reliable information on these factors, therefore it is important that the regular inspection and maintenance processes capture the information for the gap analysis.

A key factor in determining any forward programme of maintenance and/or renewals is the maintenance strategy to be adopted. One of a range of options could be adopted:

- Do minimum and service restrictions
  - For example, undertake reactive maintenance only to keep assets at the operational threshold.
- Corrective maintenance
  - For example, undertake reactive maintenance reinstating deteriorated elements.
- Time-based preventative maintenance
  - For example, column replacement perhaps every 30 years based upon manufacturer' data.
- Condition-based preventative maintenance
  - For example, undertake maintenance when the condition falls to a defined threshold.

Reactive (ad hoc) maintenance

A key element in developing a maintenance strategy is to set intervention levels – at which point a repair and/or replacement is made – and the timescales within which such intervention should be made.

There is a clear distinction between an investigatory level and an intervention level. An inspector will identify and record defects, some requiring immediate intervention, and others requiring further investigation before deciding what may need to be done.

Investigatory levels, together with procedures for dealing with such identified defects/issues, also need to be set. For example, an inspector may find that a lighting column has excessive movement yet there are no defects. There is therefore an identified risk that will require further investigation.

It is common practice to maintain and publish a risk register that identifies common defects and the range of risks likely to be encountered. The risk register contains an assessment of the likely impact should the risk occur and the probability of it happening. A risk matrix is then formulated so that response times are aligned to ranges of risk factor scores.

The underlying principles of how the probability and impact of risks are quantified are included elsewhere in this guidance. This encompasses the defects and their extent for all items in the register. It is important that this information is available in the event of possible future litigation.

Bear in mind that the risk register is for guidance only, and that the inspector can assess risks from first principles with the benefit of local knowledge that could result in a different risk factor from that appearing in the register. In such cases, it is essential that the inspector records the reasons for the variation.

All items for inspection which meet or exceed the appropriate defect investigatory level must be recorded, together with the assessed risk and action to be taken (including no action to be taken if applicable).

Routine inspection and maintenance

Historically, local authorities have undertaken inspections and certain maintenance activities at set intervals, for example yearly safety inspections and two-yearly bulk light change and clean. To introduce a risk-based approach to such activities a greater understanding of the assets is needed.

How to develop a risk-based inspection and testing programme is covered elsewhere in this guidance. A similar approach can be taken in respect of routine maintenance.

Planned maintenance and renewals

The guidance in respect of life-cycle planning is contained in the *Lifecycle Planning Toolkit* (issued by HMEP in November 2012). The *Ancillary Assets Toolkit* is aimed at providing users with planning-level decision support in the maintenance management of ancillary highway assets and is applicable to street lighting. This gives a high-level long-term assessment of investment needs required to meet any defined long-term service level (and/or defined condition). The toolkit enables 'What if' scenarios to be modelled, taking into consideration restrictions/funding constraints.

The current toolkit for ancillary assets is a relatively simple tool. It is similar to the highways toolkit which uses 'default deterioration curves'. The development of specific deterioration models for lighting together with the monitoring of performance trends in respect of condition will enable the toolkit to become more accurate.

This guidance gives advice in respect of deterioration modelling and how to determine 'assumed residual life' and introduces a column condition index (CCI) which will assist greatly in refining life cycle plans and determining future investment needs.

What the *Lifecycle Planning Toolkit* does not do, which is often mistaken, is determine annual work programmes on a prioritised basis.

Annual planned maintenance and value management (VM)

Value management is recognised as a core component of good asset management. Its purpose is to support the development of the annual programme enabling all stakeholders to have full visibility of how the programme is developed.

Value management is a formal process (‘multi-criteria decision analysis’) that enables engineering data and end-user considerations to be used in a transparent and objective manner for the identification, assessment and prioritisation of maintenance needs.

Various criteria (factors considered relevant to the prioritisation process for the replacement of the existing public lighting system) are identified, for example structural condition, optical performance of the existing lighting, electrical condition, crime and perception of crime, accident statistics etc, and a weighting is applied to each of them.

Prioritisation is carried out annually using updated information from the asset register which includes condition data and a prioritised programme of work is then produced. The amount of work taken forward will be dependent upon available finance.

In respect of renewals and/or upgrading, a revision to the VM tool can be made taking into consideration lighting deficiency (current illumination versus required illumination), optical performance and aesthetic appearance. Other factors could also be included such as load-carrying capacity requirements, for example the need for columns to carry other equipment such as smart technology.

14. Condition index and performance monitoring

Why is it important to have a column condition indicator (CCI) and lighting condition indicator (LCI)?

Both CCI and LCI are performance measures that can be used to determine whether the overall condition of an asset owner's stock is deteriorating or not. Asset owners who do not have luminaires can assess stock condition using CCI whereas asset owners with luminaires, signals or other such equipment may progress to LCI using the additional parameters of reliability and energy efficiency. Condition indexation may be used as a means for monitoring whether adequate funding is being provided for maintenance work. It can also assist in determining whether investment is being targeted in the right areas.

Performance measurement is an integral and important component of a good asset-management system and is key to achieving significant improvements in performance. A minor structure stock with a 'poor' condition indicator may need suitable intervention in the near future including refurbishment, repair or replacement and need funding and programming.

By monitoring the condition indicator values over time, early warnings of progressive degradation in performance can be identified and corrective action taken at an early stage.

Column condition indices and lighting condition indices are also discussed in Section 15 and Appendices 4 and 7 of this guidance

Why should I adopt LCI if I am a lighting manager?

Adopting the use of the LCI will enable an asset manager, with the responsibility for a large stock of lighting units, to have a clear understanding of the condition of the stock over time and assist in the determination as to whether the stock is deteriorating, improving or at a steady state. Understanding any changes and trends in the condition indices will assist greatly in life-cycle planning and justifying the need for investment.

The condition indices can be analysed and grouped together by street, estate, ward, area/district, town or city depending on the set up of a local authority and/or asset owner to assist in targeting investment and/or developing future investment strategies.

The condition indices can also be grouped by column type to assist in determining whether any particular type is deteriorating more than others.

It is not only public bodies that are being placed under increasing financial scrutiny to justify the resources needed to manage their lighting stock and to demonstrate best use of resources. The LCI results provide a transparent and defensible business case to present to senior managers and politicians to demonstrate where investment is most needed on the lighting network.

Having a standardised lighting condition indicator in use enables benchmarking possibilities to allow comparisons to be made to other authorities and asset owners.

In the UK, condition indices for carriageways, footways and bridges and have been in use for many years. These indices have been used by the Department for Transport to determine funding allocations between local highway authorities.

Having an easily understandable and measurable way of demonstrating lighting condition within an authority – and being able to compare this against similar condition or performance indicators for other asset types on the highway network such as bridges, drainage, footways and carriageways – supports decision making and investment priorities within the authority.

A lighting condition indicator which is an industry-recognised standard to demonstrate lighting performance, banded into easily understood excellent/good/fair/poor categories, can be communicated to the wider public, providing an openness and transparency on the lighting condition within an authority not only at a stock level but also at a town or district level.

LCI and value management (VM)

LCI scores are considered an essential component in any lighting value management (VM) exercise used to establish prioritised forward plans of work. It is recommended that LCI scores are combined with other VM criteria specific to local needs such as accident and crime statistics, prestige streets/walking routes, streets with heritage columns etc.

Deterioration modelling is a means of predicting the life of an asset based on certain information such as the physical attributes of its composition such as material type and thickness and the environment in which it operates. It then allows asset owners to track the life of an asset against the prediction and make assessments of life expectancy and replacement. Degradation modelling is a form of life-cycle planning and is also recognised as an important component of a good asset-management system. The results from any life-cycle planning procedure should feed into a VM exercise.

15. Deterioration modelling 'assumed residual life'

Risk-based asset management

As discussed previously the migration from basic to advanced asset management focuses on understanding the nature and extent of stock condition leading to an understanding of the risks posed by the asset stock condition, culminating in the development of a strategy (financial, performance and programme based) to manage the risks posed by asset stock condition.

Whereas both column condition indices (CCI) and lighting condition indices (LCI) are useful tools to understanding asset stock condition and how it is affected by investment, assumed residual life (ARL) is a useful tool to determining the direction of investment to achieve improved CCI and LCI discussed in Appendices 6 and 7.

Probabilistic approach to risk-based asset management

A combination of visual inspection and non-destructive testing methods are utilised to assess the structural condition of lighting columns and other supports. These techniques could provide a short-term indication of the health of an asset owner's overall lighting stock.

The computation of ARL modelling uses inspection and testing data, column geometry (shape), location, environmental factors (inland/coastal) and any other factors that could affect column performance such as attachments.

In the absence of material testing and exhaustive testing of every column there will exist uncertainties relating to condition, parent material strength, protective coating quality and application, etc. Probabilistic modelling accommodates these random variables to give a single, likely, outcome. It is a widely used statistical approach enabling development of ARL with basic input data. Basic data will produce crude ARL estimates but as successive test data for the same asset is collected and input into the model the ARL predictions are refined.

The approach to probabilistic modelling in determining ARL is outlined in more detail in Appendices 5 and 6.

Risk acceptance level

Whilst there are obvious boundaries to what is acceptable and not acceptable, there exists a broad zone within which different asset owners will take differing views on when to intervene on a deteriorating asset.

The Health and Safety Executive has published guidance on 'As Low As Reasonably Practicable' (ALARP). The origins of ALARP follow the legal case of Edwards v The National Coal Board. In that case, the Court of Appeal considered whether or not it was reasonably practicable to make the roof and sides of a road in a mine secure. The Court of Appeal held that

> *"...in every case, it is the risk that has to be weighed against the measures necessary to eliminate the risk. The greater the risk, no doubt, the less will be the weight to be given to the factor of cost."*

and

> *" 'Reasonably practicable' is a narrower term than 'physically possible' and seems to me to imply that a computation must be made by the owner in which the quantum of risk is placed on one scale and the sacrifice*

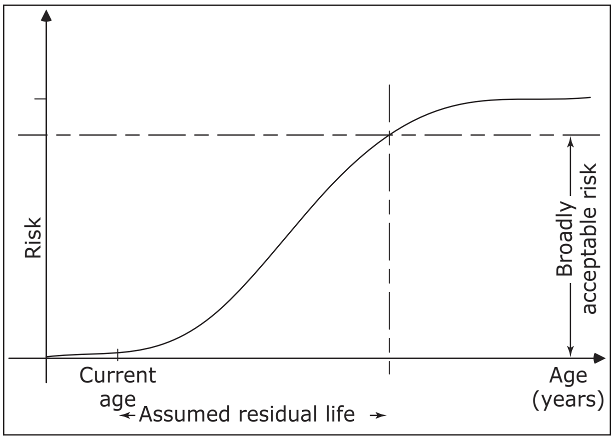


Figure 15.1: Lighting asset risk profile over time

involved in the measures necessary for averting the risk (whether in money, time or trouble) is placed in the other, and that, if it be shown that there is a gross disproportion between them – the risk being insignificant in relation to the sacrifice – the defendants discharge the onus on them."

Accordingly, there remains a zone within which risks may be considered subjectively acceptable when the consequences of failure is weighed in light of other factors such as vehicular/pedestrian count, vehicular speeds, highway network hierarchy, railway environment and suchlike.

The comparison of ARL and acceptable risk is demonstrated graphically in Figure 15.1.

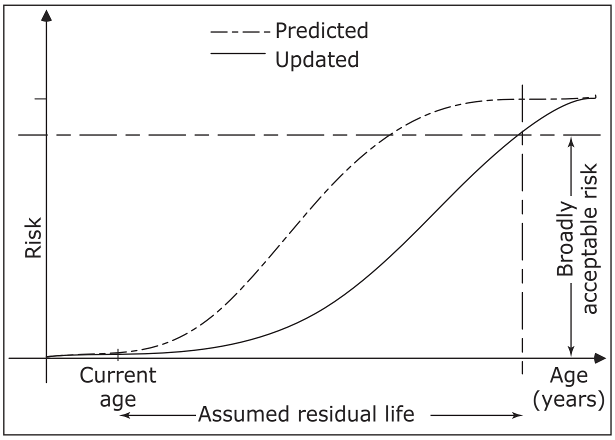


Figure 15.2: Updated lighting asset risk profile over time

Using this profile in conjunction with the minor structure failure consequences, the risk profile of the lighting assets is calculated.

This profile, in line with the latest collected data, may be updated to incorporate additional or latest information relating to specific assets arising from test reports, visual inspection or de-commissioning reports.

The updated model may well move the predicted ARL as local factors, deterioration rates and other conditions are better understood, as demonstrated in Figure 15.2.

Selection of the minimum broadly acceptable failure rate is one of the most important steps to determining the ARL of

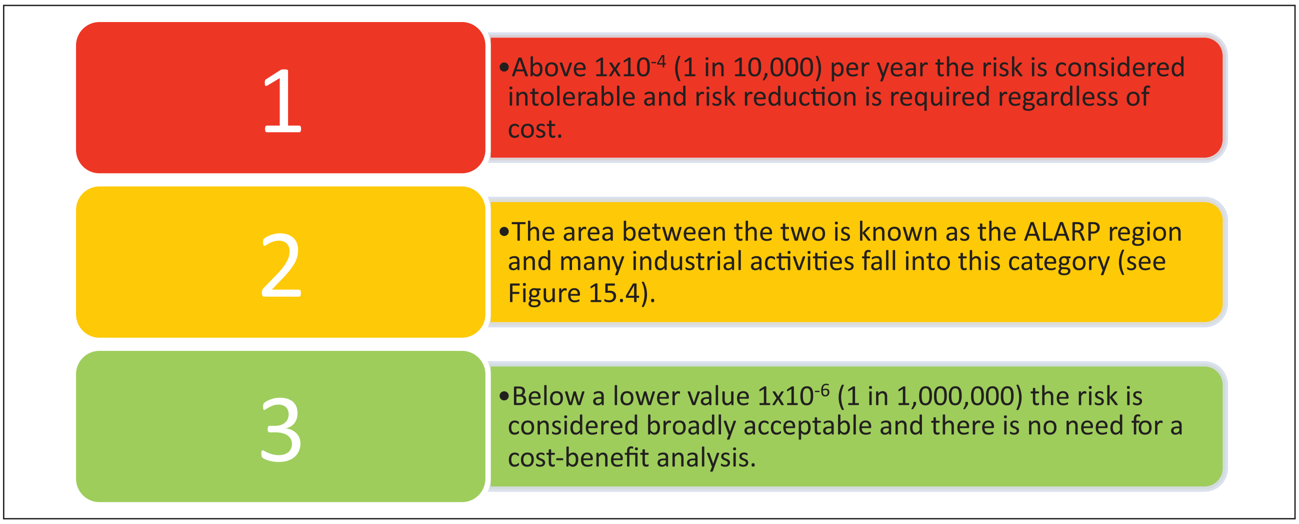


Figure 15.3: Risk categories

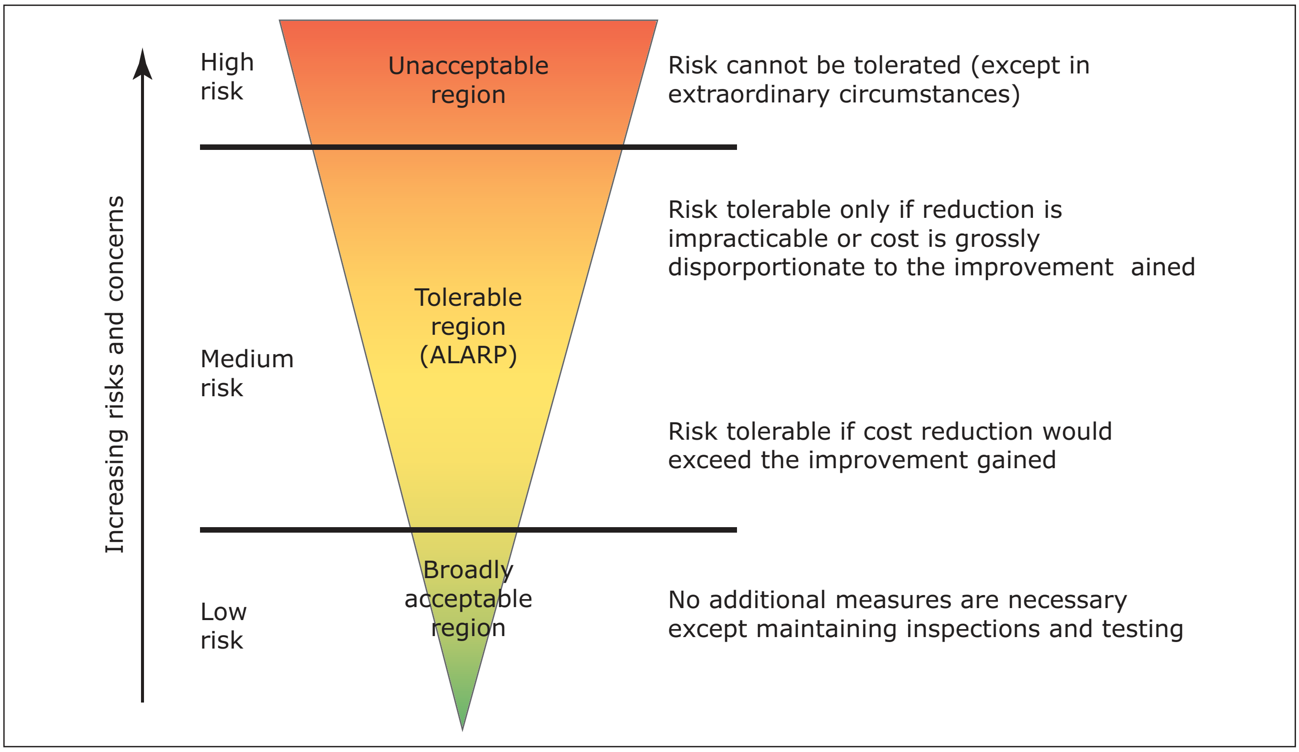


Figure 15.4: As Low As Reasonably Practicable principles (ALARP)

a lighting asset. The ‘as low as reasonably practicable’ or ALARP principle may then be applied to select the appropriate acceptable risk associated with failure of an asset.

In simple terms, a level of risk is considered tolerable or ALARP if the cost of risk reduction is grossly disproportionate to the improvement gained, ie some from of cost-benefit analysis is needed. The HSE has produced guidance on this subject such as ALARP ‘at a glance’ available on the HSE web site (www.hse.gov.uk). Three categories are described as indicated in Figures 15.3 and 15.4, where the risk levels relate to those imposed on the public by a third party.

These are only for indication and it is for the asset owner to consider their own risk categories based upon their tolerance for risk. The risk of a fatality or injury from the collapse of a lighting column must be judged against these, or similarly defined, criteria. The risk will increase with time in the absence of any maintenance of the minor structure stock.

Based on inspection/test costs and their required frequency it will be evaluated if category 3 (risk below 1×10^{-6}) or category 2 (ALARP region) is more appropriate for minor structures.

16. Glossary

Asset	An asset is a singular item of equipment such as a column, and stock is the group term for assets.
Asset management	Asset management refers to a systematic approach to the governance, management and realisation of value from an asset/stock over its whole life cycle.
Attachment	An item attached to a minor structure, such as a luminaire, banner or sign.
Base plate	A plate fixed to a planted column below ground level to prevent the column sinking into poor ground.
Bracket fixing	The connecting part on a column for securing a separate bracket. It may be of the same or a different cross section from the column.
Bracket gusset	An optional small triangular plate welded between the vertical and horizontal bracket tube to provide additional stiffening.
Bracket outreach	The horizontal distance from the centre point of the light source within the luminaire to a vertical line passing through the centre of the cross section of the column at the ground level.
Bracket projection	The horizontal distance from the point of entry to the luminaire to a vertical line passing through the centre of the cross section of the column at the ground level.
Capital	One-off costs usually associated with the provision of an asset or the refurbishment of an existing asset (cf *Revenue*).
Column base	That portion of the column from ground level to a short distance above the top of the base compartment and door, terminated by the shoulder for tubular columns.
Column condition index (CCI)	A score assigned to a minor structure indicating its current condition: excellent, good, fair or poor.
Column root	That portion of a planted column below ground.
Column shaft	That portion of the column above the column base and above the shoulder for tubular columns.
Competence	Indicates sufficiency of knowledge and skills that enable someone to act in a wide variety of situations.
Competency	Related abilities, commitments, knowledge, and skills that enable a person (or an organization) to act effectively in a job or situation.
Condition index	A system for asset-condition rating based on a methodology for the assessment of the condition of an asset relative to other similar assets determined in a consistent and repeatable manner.
Current age	The present age of the lighting column expressed in years.

Deterioration	The process of becoming worse/impaired.
Deterioration modelling	An integral part of asset management used to predict future asset condition and to establish funding requirements.
Elbow	The connection or bend at the change of direction of the substantially horizontal bracket arm and the vertical bracket tube.
Enhancement works	The undertaking of works to reduce the deterioration of an asset.
Flange gussets	Optional small plates, generally of triangular form, welded between the column shaft and the flange plate to assist in the distribution of load.
Flange plate	A plate, with an opening for a cable entry, attached rigidly to a column to enable surface mounting to a concrete foundation or other secure structure.
Indicative test methods	Testing serving as an indication of the asset condition.
Level of service	The delivery of a service compliant with standards and timescales agreed through consultation.
Life cycle	The series of stages that take place in an asset from its beginning until the end of its usefulness.
Life-cycle plan	A process to assist with the planning, projections and decisions.
Lighting condition index (LCI)	A index that denotes the performance of a lighting installation in terms of its overall condition and performance.
Luminaire	A unit mounted on a minor structure that produces light.
Luminaire fixing	The connecting part on the end of a bracket or at the top of a column for securing the luminaire. It may be the end of the bracket or the column itself, or an additional part with the same or a different cross section from that of the column or bracket. Frequently known as a luminaire spigot.
Non-destructive testing	A process of testing a minor structure without causing damage to it or affecting its performance ability.
Minor structure	A structure that supports a luminaire, sign, signal, CCTV.
Monitoring	A process of repeated testing undertaken as regular times, for example annually.
Pinching screws	Screws used to secure the bracket on to the bracket fixing by means of tightening a series of screws. On occasions bolts can be used, particularly in the case of older columns.
Planting depth	The length of the column below the intended ground level.

Quantitative test methods	A test methodology producing a measured value.
Residual life	The period remaining until the asset will next require reconstruction, rehabilitation, restoration or renewal.
Revenue	On-going operational costs of an asset (cf *Capital*).
Shoulder	A change in diameter of the column shaft typically at the top of the base compartment on tubular columns.
Spigot	See luminaire fixing.
Stock	The group term for a collection of assets.
Support	A minor structure supporting an attachment.
Test house	A competent organisation undertaking the assessment of the asset condition.
Visible cracks	These are cracks of a width greater than 0.1mm.

17. References

An Introduction to Highway Law or in *The Highways Act 1980 with annotations by Charles Cross and Stephen Sauvain*

British Institute of Non-Destructive Testing (BINDT)

Code of Practice *Well-managed Highway Infrastructure*

Construction (Design and Management) Regulations 2015

Construction Fixings Association (CFA) guidance note: *Anchorage Systems for Seasonal Decorations*

Department for Transport

- Departmental Standard BD 26/99 Design of Lighting Columns

Highways Act 1980

Highways Maintenance Efficiency Programme (HMEP)

Chartered Institution of Highways & Transportation (CIHT) document *Well Managed Highway Liability Risk*; available at www.ukroadsliaisongroup.org/en/guidance/highway-risk--liability.cfm

HSE

- ALARP guidance as published on their web site www.hse.gov.uk/risk/theory/alarpglance.htm

Institution of Lighting Professionals

- PLG07 *High masts for lighting and CCTV* (2013 edition)
- TR22 *Managing a vital asset: Lighting supports* (superseded)

Office of Rail and Road (ORR) operates a Risk Management Maturity Model (RM^3)

Standards

- BS EN 40 *Lighting columns*
- PD 6547:2004+A1:2009 *Guidance on the use of BS EN 40-3-1 and BS EN 40-3-3*

Transport Road Laboratory Limited (TRL)

- PR/IS/07/02 *A Risk Management Strategy for Lighting Columns*
- PR/IS/25/01 *Non-Destructive Testing of Lighting Columns*, Final Report, Unpublished

Westminster City Council

- Column degradation modelling/condition index

18. Bibliography

Standards

- BS 729:1971 *Specification for hot dip galvanised coatings on iron and steel articles*. (Superseded, Withdrawn)
- BS 2569-2:1965 *Specification for sprayed metal coatings: Protection of iron and steel against corrosion and oxidation at elevated temperatures*
- BS 3900 *Methods of test for paints*
- BS 5493:1977 *Code of practice for protective coating of iron and steel structures against corrosion*
- BS 5649 *Lighting columns*
- BS 7079 *Preparation of steel substrates before application of paints and related products*
- BS 7773:1995 *Code of practice for cleaning and preparation of metal surfaces*
- BS EN 60529:1992 *Specification for degrees of protection provided by enclosures (IP code)*
- BS EN ISO 1461:2009 *Hot dip galvanized coatings on fabricated iron and steel articles. Specifications and test methods*

ISO Standards

- ISO 31000:2018 *Risk management – Guidelines*
- ISO 55000 *Asset management – Overview, principles and terminology*
- ISO 55001 *Asset management – Management systems – Requirements*
- ISO 55002 *Asset management – Management systems – Guidelines for the application of ISO 55001*

Institution of Lighting Professionals

- Technical Report No 9 *Protective Coatings for Steel Lighting Columns* (1980) (Withdrawn)
- Technical Report No 18 *The Planned Replacement of Lighting Columns* (1988) (Withdrawn)

Galvanizers Association

- Hot dip galvanizing corrosion map; www.galvanizing.org.uk/corrosion-map/

Department for Transport

- TRMM 2/88 *Trunk Road and Motorway Structures – Records and Inspection*

Other

- Lighting Column Technical Forum
- *Health and Safety at Work etc Act 1974*
- *New Roads and Street Works Act 1991*
- Routine Maintenance Manual for Trunk Road and Trunk Motorways (Withdrawn)

Appendix 1
Non-destructive testing methodologies

Introduction

The following inspection and testing approaches/systems are provided and explained for general information. Those commissioning such inspections and testing operations need to assure themselves that the companies and their operatives have the required competencies to undertake the work and that the inspection approach/test operation(s) are fit for the intended purpose as discussed within this guidance and will not cause damage or adversely affect the condition of the asset.

Indicative inspection and test methods

Visual inspection

Visual inspection is a very effective way of determining whether minor structures have certain serious defects. The presence of holes around bases or cracks at door openings are often reasons why minor structures are replaced without the need for further testing. However, it is not possible to rely solely on visual inspection.

Many defects that may reduce the strength of minor structures significantly and cause them to collapse – for example corrosion of the internal and external surfaces below ground, corrosion of the internal surfaces at hot-swaged joints, and fatigue cracking – are not visually apparent. It will therefore be necessary to consider other inspection and testing techniques to understand the condition of the minor structure as the areas of concern may be hidden under protective coatings or just not visual due to the design of the structure.

Therefore, visual inspection is suitable only for the initial general inspection of large areas where major cracking, damage or visual corrosion can be expected, as demonstrated in Pictures A1.1, A1.2 and A1.3.

Specifications provided by authorities indicate that visual inspections can assess the following (although obviously only when they are visible):

- Corrosion, damage to the protective treatment and cracking at the flange plate (flange-plated columns);
- Corrosion of holding-down bolts (flange-plated columns);
- Missing or loose holding-down bolts (flange-plated columns);

Pictures A1.1, A1.2 and A1.3: Corroded flange plate; cracking of paving surrounding embellishment kit; corroded column base.

- The presence of water in the base compartment and corrosion on internal surfaces;
- Number, type and details of attachments, with photographs;
- Corrosion and damage to the protective treatment below ground level (planted columns);
- Corrosion and damage to the protective treatment from ground level to the door opening;
- Corrosion and damage to the protective treatment and cracking at the door opening;
- Corrosion and damage to the reinforced strips in steel columns with reinforced door openings;
- Missing spacing plugs in prestressed concrete columns;
- Cracking of paving/concrete surrounding an embellishment kit, possibly indicating column movement and corrosion of the root under the embellishment kit;
- Corrosion and damage to the protective treatment at the base of the shaft;
- Corrosion and damage to the protective treatment at the welded joints between the sections of columns;
- Corrosion and damage to the protective treatment and cracking at the top of the shaft where it is connected to the bracket (especially cracking of concrete columns);
- Corrosion and damage to the protective treatment and cracking at the bracket where it is connected to the shaft (especially cracking of concrete brackets);
- Missing bolts at the bracket connection;
- Corrosion and damage to the protective treatment and cracking along the bracket and near the luminaire (especially cracking of concrete columns and brackets);
- Any impact damage;
- General condition of the protective treatment, including staining, blistering and flaking.

It is possible to carry out a simple visual inspection every time a column is visited for any maintenance operation. Good eyesight and good lighting are required, and optical aids are beneficial. The bracket can be inspected when a hoist is used to check the luminaire.

Attention should be paid to areas where damage to the protective treatment may accelerate corrosion; reparation work should be considered in these areas. Those who are painting minor structures should be encouraged to report defects, as they can be less easy to detect if painted over.

On some minor structures, corrosion on the internal surfaces of the root and at the swaged joint can be assessed with access through the door opening by using an endoscope. The extent of any flooding of the root should be assessed after a period of wet weather.

Any distortion of the minor structure due to impact damage will increase the bending and torsion moments at some critical sections, and reduce the strength of the damaged sections.

The advantages of a visual inspection are that:

- Inspections of the base section near and above ground level are low cost;
- It can identify the need for maintenance, for example painting;
- It can identify serious defects rapidly;
- It can identify unauthorised attachments;
- It can identify some defects which require columns to be replaced without further testing;
- It does not damage the minor structure.

The disadvantages are that:

- Defects that reduce the strength of the critical sections significantly may not be identified;
- It is difficult and costly to inspect below ground, internal surfaces and brackets;

- Assessment is subjective, and may differ according to the weather (that is, light level and wetness of surfaces);
- It is not possible to estimate the strength from observations.

Aural/tap test on steel columns

A few authorities have used aural/tap testing to determine the condition of the base and shaft of steel columns. It is claimed that a skilled operative can determine an indicative condition of the asset by listening to the 'ring' from the tap. A dull thud can be heard when a section thinned by corrosion is lightly tapped with a hammer, whereas uncorroded columns can ring clearly when tapped. Also, the amount of debris heard falling inside the column following the tap gives an indication of the level of internal corrosion. In extreme cases, corrosion is so bad that the asset dents or perforates.

Some authorities have commented that the variation in the tone of the thud from one column to another is indistinguishable. The method may be effective on columns with severe corrosion where rust has formed in layers. It may be less effective on newer columns with thin sections designed to BS 5649 when comparatively little corrosion may reduce their strength significantly.

The advantages of the aural/tap tapping are that:
- It is very low cost;
- It can detect serious defects rapidly; and
- It can detect some defects which require columns to be replaced without further testing.

The disadvantages are that:
- It may be ineffective near the door opening and at the connection between the base and shaft;
- It cannot detect all defects below ground, for example corrosion deep in the root of columns in poorly-drained ground;
- It cannot detect corrosion at hot-swaged joints;
- Assessment is subjective, therefore significant corrosion may not be detected;
- It may damage the protective treatment;
- It is not possible to estimate the strength from the results.

Ultrasonic testing to measure section thickness

Ultrasonic testing is used by many authorities to measure the wall thickness of steel columns. The testing can be carried out by competent in-house staff or by test organisations.

This is the most commonly used method for measurements of remaining wall thickness on minor structures. The test involves the use of a hand-held probe placed on a clean surface which measures the metal thickness to determine wastage or corrosion, accurately, quickly and, in general, without the need to remove surface coatings.

Various ultrasonic instruments are available, each with its own claim over rivals. A number of instruments take measurements using the multiple echo method, and these will ignore any protective coatings and will only give measurements relating to the actual metal thickness; see Figure A1.1. The timing

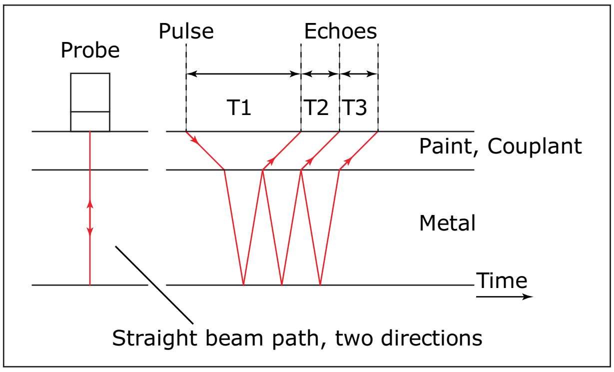


Figure A1.1: Multiple echo single probe between any two successive back wall echoes gives a true indication of thickness of the metal only. T2 will equal T3 as shown in Figure A1.1 for only true back wall echoes. In most cases this can eliminate the need to remove the protective coating, although some surface preparation will still be required to remove surface rust and dirt in order to obtain a good coupling between the probe and material surface.

Consideration must be given to the choice of probe used. Probes are available which utilise one or two crystals; some have hard faces and others incorporate membranes thus allowing a soft compressible face which can be used on curved surfaces. It should be noted that some probes/instruments will only measure/operate in a range of thicknesses. There is no point in using a system that can only measure to 5mm minimum if the base thickness is 3.5mm.

The method of testing requires as many measurements to be taken as is reasonably practicable and cost effective. A general method requires measurements to be taken on four faces of the column (referenced to the access door). These measurements are taken at 100mm intervals from ground level to 300mm above ground level in hard ground and from 300mm below ground level to 300mm above in soft ground. A check reading is also taken just below the column's shoulder to obtain an indication of the original wall thickness.

The results obtained from this type of inspection will indicate the amount of remaining wall thickness at the points of test. This information should then be referred to a structural engineer for evaluation. In general, it will be possible for the structural engineer to provide a percentage corrosion rule of thumb, based upon the original wall thickness, which will indicate if the minor structure is sound or needs to be removed. There will be a grey area between these two conditions where advice will need to be sought. This rule will obviously be based upon a standard minor structure with only a bracket and lantern fitted.

After considering authorities' specifications it is concluded that in order to determine the integrity of the full column, the wall thickness should be measured at the following locations:

- From 300mm below ground (maybe lower) to ground level (in poorly drained ground only);
- From 100mm below ground level to ground level (in well-drained ground);
- From ground level to the bottom of the door opening;
- At the door opening, including the four corners of the door opening;
- At hot-swaged joints and changes in section.

It is recommended that the minimum and maximum thicknesses in these areas are recorded. In addition, the thickness should be recorded at no less than four points around the circumference at the height of the minimum thickness. Further measurements could be taken in order to determine the strength of each critical section accurately, but this is impractical. The cost of the structural analysis would be significant. The weakest section may not be where the minimum thickness is measured. Therefore, it is proposed that the minimum thickness is compared with the minimum wall thickness (uniform thickness around the full circumference) that satisfies the chosen service criteria. If the section is severely corroded, ie all the thicknesses around the circumference are low, the column should be replaced, even if there is sufficient wall thickness to satisfy the service criteria.

A range of ultrasonic measuring devices has been used or could be used to test minor structures. They may use compression waves, shear waves or surface waves. The devices most

Picture A1.4: Ultrasonic testing

commonly used on minor structures are small hand-held compression wave probes that can be placed against the external surface. Some probes measure the wall thickness from the frequency of the standing wave at resonance. Most probes used on columns measure the wall thickness from the time it takes for waves to travel through the metal. Either the time for the first echo to return from the internal surface is measured, or the time between multiple echoes as waves are reflected back and forth from the internal and external surfaces of the metal. The latter measures only the thickness of the metal. The former includes the equivalent thickness of any protective treatment.

Compression waves are low in amplitude, and there must be good contact between the probe and the surface of the column if waves of sufficient amplitude are to be transmitted into the test section so the thickness can be measured. A liquid couplant is used, but problems arise if there is an air gap in the transmission path, for example the protective treatment is damaged or there is surface rust. Some devices can be ineffective on minor structures with only slight rust or loose paint on the external surfaces. They may operate satisfactorily with good preparation, but this is costly and often impractical.

There are further problems associated with the way the waves are reflected at corroded surfaces. They can be highly dispersed so only a small amount of energy may return to the probe. The thinnest section may not be measured if there are peaks and valleys at the corroded surfaces.

There is much debate about what type of device is the most suitable for testing minor structures.

Devices that monitor the first echo are slightly less dependent on the coupling and the reflection from corroded surfaces than multiple echo devices, but care is needed to interpret the results as the measured thickness includes the equivalent thickness of the protective treatment.

It is claimed that probes with a separate transmitter and receiver rather than a combined probe can focus the waves to increase the amplitude of the reflected wave. Also, some test organisations use probes with more specialised signal conditioning equipment and, by monitoring the A-scan display, changes in the characteristics of the reflected wave can be used to assess the thickness of reasonably large areas. When the thinnest sections have been identified, their wall thickness can then be measured accurately. Some devices can measure the wall thickness remote from the probes. Some may be able to measure the wall thickness at welded joints where the parent metal has been thinned by the welding process. However, the more sophisticated test methods may not be cost effective on minor structures.

Normally, the root must be excavated to measure the wall thickness below ground. This is costly in hard surfaces (ie concrete, paving, bituminous surfacing), especially as it is necessary to excavate the full circumference in order to determine the extent of any corrosion. Measurements have been taken from the inside of some columns, but the method is unlikely to be successful when there is severe internal corrosion. If it is not possible to measure the wall thickness from inside the column, it is not possible to take measurements where there is a concrete foundation.

Whatever type of device is used, any surface preparation that is required to take a measurement increases the testing time. Any damage to the protective treatment caused during the preparation should be repaired, whether it is above or below ground.

Ultrasonic testing has a number of pitfalls and is only as good as the number of readings taken. Obviously, measurements are only valid for each probe location, which is also reliant on a good coupling being achieved between the probe and the surface under test; preparation of the surface is therefore important. When coupling cannot be achieved the operator may move the probe to a 'better' area and thus miss the critical area of measurement.

It should be noted that ultrasonic testing will only measure one thickness of metal; see Figure A1.2. The ultrasonic signal is reflected by the 'internal' joint face rather than passing through to the 'rear' wall. The thickness measured only relates to distance 'B' rather than 'A'. It is therefore not suitable for measuring through embellishment kits or at construction lap joints as it will only record the thickness of the top metal layer.

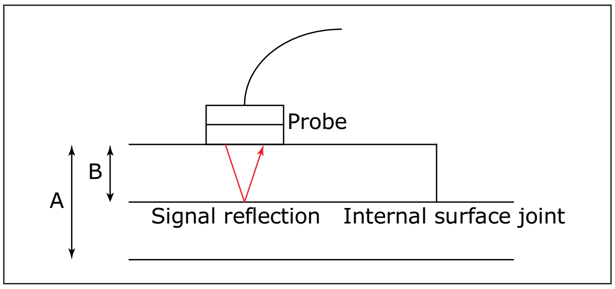


Figure A1.2: Measurement at lap joint

A major concern with ultrasonic testing is that the measurements are very dependent on the operator. Clearly, a key requirement is that measurements are accurate on deteriorated minor structures with corroded external and internal surfaces. Devices with a digital display may give no reading at some locations. When it is difficult or not possible to take a measurement, the operator may move the probe to a location where a measurement can be taken, so the thickness of the critical sections where there is severe corrosion may not be recorded. The situation is not helped when the operator is encouraged to take the measurements quickly in order to maximise the number of minor structures that are tested.

The advantages of ultrasonic thickness measurement are that:

- The cost of most test equipment is reasonably low;
- Little access is needed for testing above ground;
- The cost of testing above ground is low;
- Successive tests can give an indication of the variation in condition of minor structures over time, although small variations in the test locations may be significant.

The disadvantages are that:

- Thorough surface preparation is required that may damage the protective treatment;
- It may not be possible to locate and measure the thickness of the weakest sections, or the measurements may be inaccurate;

- Devices with a digital display may give no reading if the section is severely corroded;
- Specially-trained staff using sophisticated conditioning equipment may be needed to measure the thickness of severely corroded sections;
- Excavation is needed to determine the condition of minor structures below ground, so the cost of testing below ground is high;
- If it is not possible to take measurements below ground from the inside the minor structure, it is not possible to measure the wall thickness below the level of a concrete foundation;
- Detailed calculations are needed in order to determine the strength of the critical sections;
- Other test methods must be used to assess the effects of cracking.

Relative loss of section monitoring

Relative loss of section (RLS) tests steel minor structures for corrosion in the key areas of concern where minor structures are known to corrode, including the root, base and swaged joint sections These are considered the most vulnerable areas at which, if corrosion is not prevented, sudden catastrophic failure of the minor structure may occur.

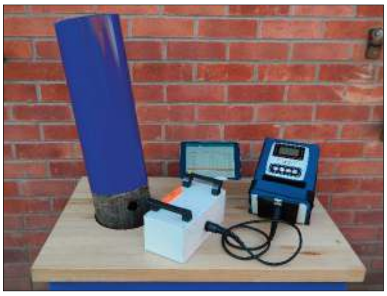
A transponder head is placed a fixed distance from the minor structure and generates a very low frequency shaped magnetic field. The only surface preparation required is to ensure that the fixed distance is maintained when measurements are taken. The magnetic flux is distorted by sound contiguous core material and metallic conductive protective treatments. Distortions to the magnetic flux are processed and related to the surface profile and the mean thickness of the material. The readings indicated are 'loss of section' units (LSUs) which give a guide value for each test section relative to a RLS value at a reference section. The reference section is chosen where no or little damage is expected and the thickness of the material can be measured by, say, ultrasonic testing. The reference value varies with the minor structure diameter, the material and the protective treatment – that is, whether the steel is plain, galvanised or metal sprayed.

It has been shown that the LSUs measured by the device are approximately equal to the bulk average percentage loss of section relative to the reference section. The readings are slightly affected by galvanised coatings, but this is considered to be insignificant in respect of the classification of minor structure condition.

The system uses various transponder search heads which generate low frequency shaped magnetic fields, a form of eddy current. The system compares loss of section in vulnerable areas of the minor structure to a reference section on the same minor structure. The instrument should be calibrated each day on a dedicated calibration steel plate.

The standard search head

The standard head monitors an area approximately the full width of the head, 200mm, and from 100mm above to 100mm below the base of the head. Therefore, when placed on the ground, the instrument can monitor a 100mm length of minor structure below ground without the need for excavation. The standard head can monitor lengths of column above ground, provided they are away from changes in section. The standard head detects sound metal up to around 100mm from the head, so half of the circumference can be monitored if the radius of the column is small, but it is normal to take measurement at four points around the circumference.




Pictures A1.5 and A1.6: RLS equipment

The mini search head

The mini head monitors an area approximately 65mm wide and from 50mm above to 50mm below the base of the head. Therefore, when the base of the head is at ground level, the loss of section up to 50mm below ground level can be determined, without the need for excavation. It is assumed that all significant loss is in the zone below the base (that is, below ground level when the base is on the ground), but this assumption can be assessed by moving the head vertically.

The mini head can be used in conjunction with the standard head to locate the most corroded sections of columns more accurately.

The provisional condition criteria suggested by test organisation for the RLS meter with the standard head are listed in Table A1.1.

The probe search head

This can be used to detect corrosion on the full depth of the root section of the column by inserting the probe down the inside of the column, and can be used where the standard or mini search heads cannot access the column at ground level due to embellishments, ornate columns or external obstructions.

Table A1.1:	
Average LSU (%)	**Recommendation**
>50%	Immediate removal or make safe
25 to 50%	Consult a structural engineer
<25%	Re-test on advice of test house and column degradation model

Swaged joint analyser (SJA)

This is designed to detect and quantify any internal corrosion at the bottom of the shaft at and around the swaged joint connection by comparing the relative section of a reference area above the joint. It may also be used to assess the relative thickness between different test areas of the same tubular section. The test area extends approximately 10mm × 10mm and up to a total overlapping thickness of 20mm to enable detection of very localised corrosion around and just above the swaged joint connection. It detects corrosion not otherwise visible as it originates on the internal surface.

Test results

Using the test results obtained, loss of section units (LSUs) are reported as an estimated bulk average percentage loss of section relative to a reference section. The data is then used to classify the column depending upon the asset owner’s appetite for risk, with a corresponding recommendation given regarding replacement or required re-testing of the column within a recommended time frame.

It is concluded that loss-of-section monitoring with the standard head is an effective indicative test for determining whether a minor structure is corroded, especially near ground level. Generally, the test results have compared favourably with the condition of minor structures that have been inspected on removal.

Although the standard head monitors the bulk average percentage loss of section of a fairly large test section, this appears to be sufficiently accurate for minor structures with un-reinforced door openings that can suffer a significant loss of section before they are below strength. There are concerns that highly localised corrosion may not be detected by the standard head.

The RLS meter should not be used to estimate the strength of a minor structure as the minimum thickness may be significantly less than the bulk average thickness that can be derived from the readings.

The advantages of loss-of-section monitoring with an RLS meter are that:

- It is low cost;
- The testing time is short;
- Minimal surface preparation is required that should not damage the protective treatment;
- It can detect highly localised corrosion causing total loss of section;
- With the standard head, the condition of minor structures up to 100mm below ground can be monitored without excavation;
- With the standard head, the condition of minor structures up to 100mm below concrete foundation can be monitored without excavation;
- Surface rust and the ground conditions do not affect the results;
- Limited access is required;
- Successive tests give an indication of the variation in the condition of minor structures with time.

The disadvantages are that:

- The standard head is most effective near ground level, and is ineffective near changes in section and door openings;
- Columns with embellishment kits cannot be tested;
- It may not detect all defects below ground, for example corrosion deep in the root of a column in poorly-drained ground;
- The bulk average percentage loss of section is measured, not minimum thickness.

Ultrasonic testing using guided waves

Ultrasonic guided waves are used for the remote detection of corrosion in pipelines and other tubular structures. The system was developed in Germany specifically for testing columns, and has been used in that country since 1999 and in the UK since 2001.

A device, incorporating a single ultrasonic probe, is attached to the column by a flexible strap that can be adjusted to suit the diameter. Long wavelength ultrasonic waves are induced in the column by the probe and propagate along its length in one direction. For example, the probe may be placed on the base section so the waves propagate into the root, or the probe may be placed on the shaft so the waves propagate towards a hot-swaged joint. The waves are reflected at corrosion sites or at discontinuities such as the cable entry slot, door opening or the end of the column.

The probe is moved around the circumference of the column by guide wheels that are operated by an electric motor. If the door opening is at a low level, a ring can be fitted to enable the guide wheels to move around the column. Separate rings are available for each column diameter.

The reflected ultrasonic waves are monitored every 2mm of travel around the circumference so that a complete profile of the column is built up. A graphical display is produced that shows the amplitude of the waves reflected from different areas of the column. Almost all of the waves may be reflected where there is severe corrosion, so corrosion further away from the probe is not detected. However, this should not be significant, as the weakest section of a column is normally where the corrosion is most severe. Where there is minor corrosion, only parts of the waves are reflected, so corrosion further away can be detected.

The amplitude of the guided ultrasonic waves is much greater than that of compression waves. The column needs to be reasonably clean, but no couplant is required between the probe and the column.

Ultrasonic testing using guided waves is an effective indicative test for detecting corrosion in the root of columns without the need for excavation and detecting corrosion at hot-swaged joints. Attempts are being made to relate the amplitude of the reflected waves to the loss of section, but qualitative assessments of the severity of corrosion are being made at present.

Test results have compared favourably with the condition of columns that have been removed for replacement.

The advantages of ultrasonic testing using guided waves are that:
- Minimal surface preparation is required that should not damage the protective treatment;
- It can identify the exact location of corrosion in the root of columns without excavation and at hot-swaged joints;
- It can identify highly localised corrosion quickly;
- Limited access is required.

The disadvantages are that:
- At present, it is difficult to determine the wall thickness or loss of section from the results;
- The cost of testing is high for an indicative method, although excavation is not required.

Phase angle technique

Electronic phase angle technique is an adaptation of a non-destructive test utilised within the oil and pipeline industry for use on lighting columns.

A stimulus coil energised by an alternating current is placed around the lighting column into which circulating eddy currents are induced, which in turn creates a magnetic field inside the lighting column.

A sensor coil/transducer located on the end of a probe is inserted into the lighting column via the access door allowing access below ground level to detect and measure the magnetic field inside the lighting column. The phase angle between the stimulus current and the measured signal is compared by an electronic processor unit. Tests have proven that there is a direct correlation between the thickness of the lighting column wall and the measured phase angle, enabling the thickness of the wall to be recorded

Where corrosion, defects, holes or loss of section exists within the wall of the lighting column, the wave forms created by the eddy currents become complex, distorted and harmonic and interact with the stimulus waveform that is detected by the probe. Processing the waveform produces a signal that can be compared to the stimulus signal to determine the percentage of phase shift caused by the waveform propagation delay, which is represented by a meter. Tests have shown that accuracy to be about 5% of the scale reading (0.15mm for a typical 3mm column wall thickness)

The induced eddy currents flow down to depths of more that one metre below ground level (typical of most lighting column root sections) and above the shoulder and swage area past any defects in the tube section. Provided a signal can be measured, then its phase angle can be determined enabling assessments to be made.

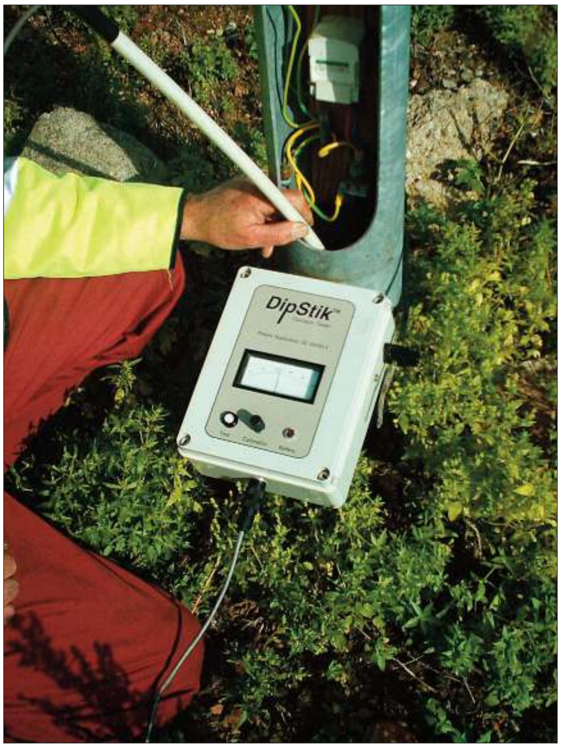
Sensor coil/transducers of differing characteristics can be used to either localise the detection area or to enable a general area of assessment eliminating the practical issues of obstructions such as cables within the lighting column

Calibration is achieved using a known good section of the lighting column wall. The stimulus current is adjusted in frequency and quantity to enable lighting column of different diameters, wall thickness and metals (permeability) to be measured.

The results can provide a clear indication of loss of section and by careful positioning of the sensor can provide an accurate measurement of the thickness of the lighting column

The advantages of phase angle technique are that:

- Results are repeatable and accurate;
- No surface preparation is required;
- It can be used inside the root section of the lighting columns without excavation;
- It can identify corrosion quickly and limited access is required;
- It is effective in the base, root and swaged areas.



Pictures A1.7 and A1.8: Phase angle testing

The disadvantages of phase angle technique are that:

- It cannot be used on lighting columns where the base has been filled
- It cannot be used on concrete or cast lighting minor structures

Corrosion-protection thickness measurements

A number of hand-held gauges are readily available that measure the thickness of protective coatings. These can be magnetic or electronic and will work on coatings on a ferrous metal. They rely on the reduction in magnetic pulling force as the thickness of the corrosion protection increases. A simple magnetic gauge (referred to as a 'banana' gauge) measures the total thickness of coating layers at any point on the steel column. Electronic gauges carry out the same operation but with greater accuracy. Some can measure both the thickness of paint over the galvanising and also the total thickness of paint and galvanising over the steel. Some electronic gauges collect the readings in memory, provide statistics and have the facility to transmit them to a printer or computer at a later time.

Comparative deformation measurement

The testing of the lighting column through comparative measurement of deformation is a low-cost alternative to the static load test.

Comparative measurement of deformation tests the stability of the column through a comparative measurement of deformation of the column/support cross-section. By establishing two comparative pressure measurements with opposing forces, the deformation of the cross section is measured and evaluated.

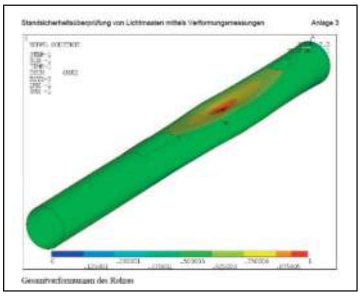
A corresponding comparison measurement is performed on the same column at about 300mm beyond

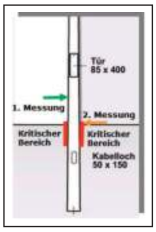
Pictures A1.9 and A1.10: Deformation measurement

the first measuring point, and from the data collected the column stability can be determined. Alternatively, the initial measured value of the reference column can be used with knowledge of a similar type of column.

The column is deformed by the diametrically opposed forces not only at the point of force introduction, but also in the next. A defective spot, such as rust, can be located in the deformation area of the force introduction point below the visible area in the ground, which could influence the deformation behaviour. The comparison with the reference measurement immediately shows that less strain could be resumed with equal deformation.

This method lends itself testing a list of existing types of minor structures and to distinguish them in terms of diameter and wall thickness. The system specific measured values allow an accurate classification of the measured values to the reference measurement and hence the condition determined.





Figures A1.3 and A1.4: Deformation measurement

Quantitative (strength) test methods

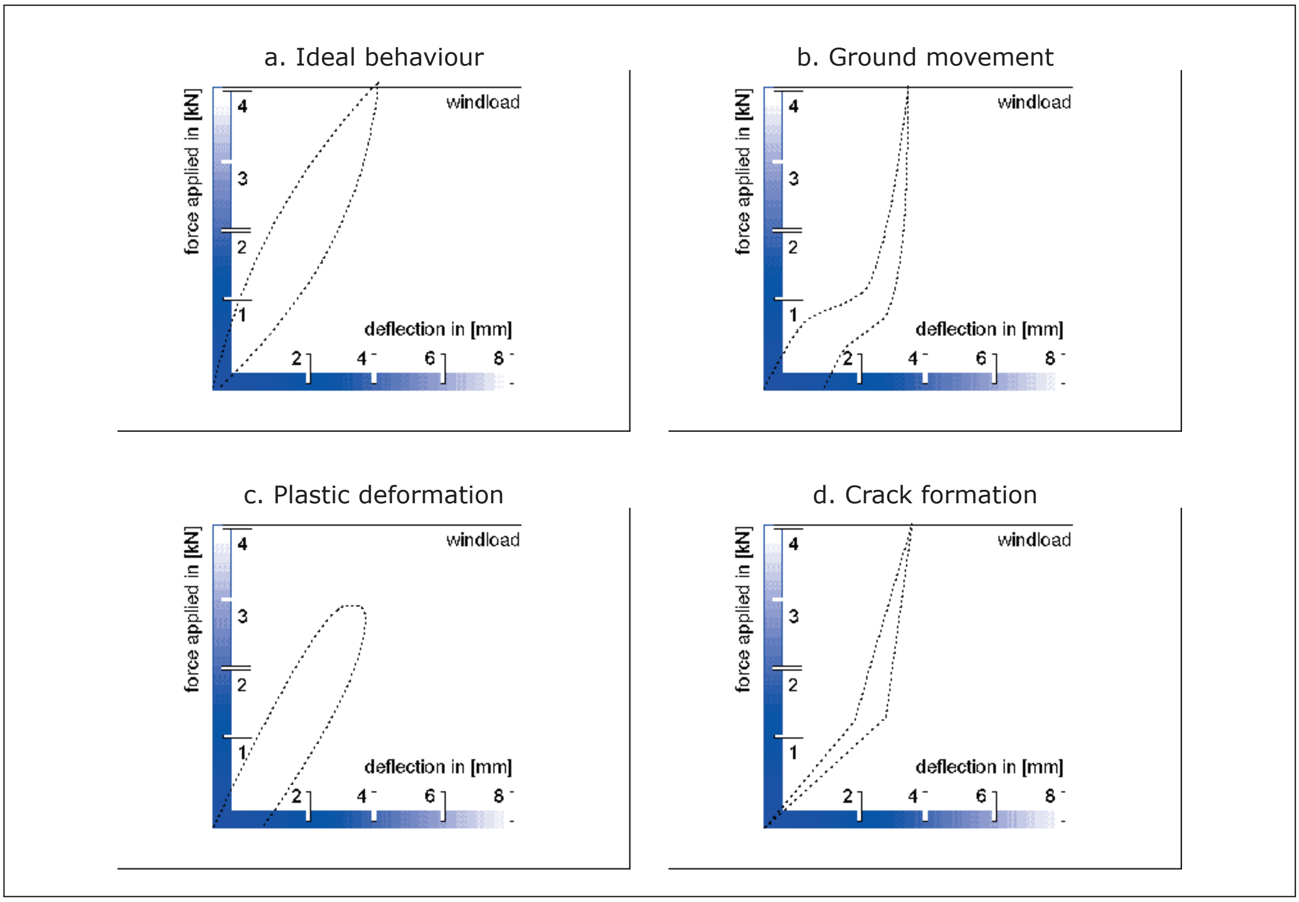
Static loading

A static load is applied to the column that simulates the bending moments induced by wind and dead loads. The condition of the column is assessed from the variation in its deflection as the applied load is increased.

Many authorities prefer static loading as it assesses the strength of the critical sections of a column directly without the need for a full structural analysis. The effects of deterioration due to both corrosion and cracking, above and below ground, can be assessed. The condition of the holding-down bolts of flanged minor structures can also be assessed.

In order to calculate the bending moments and the foundation overturning moment induced by the wind and dead loads, data on the base, shaft, bracket, luminaire and attachments of the column is required, including an estimate of the yield strength of the steel. The authority may request that an allowance be made for different attachments or luminaires.

The bending moments induced by wind loads are calculated from the details of the column. The magnitude of the test load and the height at which it is to be applied are then determined. It is possible to identify a height at which the test load should be applied such that the bending moments induced by wind loads and the test load are similar at all critical sections from the base/shaft connection to ground level. It may not be possible to apply the load at this height on all tall columns because this may exceed the maximum height that can be achieved with the test equipment. Therefore, on tall columns, the moments may be equated at a certain



Figures A1.5 a, b, c, d: Deflection results for different behaviours

critical location, for example at the connection between the base and the shaft. The bending moments induced by the test load below this critical location, for example at ground level, may then be higher than those induced by wind loads, and checks must be made to ensure that the strength of the column is not exceeded during the test.

The test load is applied to each column four times. The first two tests push and pull the column in the direction of minimum strength, ie perpendicular to the door opening. The second two tests push and pull the column in a direction ranging from 45° to 90° to the first two.

The static system's points of support remain unaltered during the entire testing procedure. This ensures that all parts of the column that bear a risk and the foundation are tested in their actual condition.

If the column exhibits ideal behaviour then a structural certificate is provided which gives a recommended re-test date which is dependent upon the precise results obtained.

This method of testing identifies the following:

Ideal behaviour

The column is structurally sound; column and foundation show no damage characteristics; see Figure A1.5a.

A test load derived from calculations based on BS 5649 or BS EN 40 as applicable was applied to the column to induce bending moments that simulate physical working conditions. The magnitude and point of application of the force applied are calculated to ensure all critical sections of the column are subject to a moment which equates to at least the design wind load, whilst not exceeding the ultimate elastic bending strength of the structure. A plastic deformation, which indicates loads exceeding the yield point (elastic limit), was not detected.

Ground movement

The column reached the wind load but ground movement was detected; the foundation or the ground does not provide a sufficient support for the column. The column should be re-tested once the ground/foundation conditions have been rectified; see Figure A1.5b.

According to the existing foundation/ ground conditions it was impossible to stress the column as described under the heading 'Ideal behaviour', above. The column achieved the wind load + 10% margin of safety without exceeding the yield point (elastic limit).

Measures to reconstruct the foundation, respectively to the ground conditions, are necessary. After that the column should be tested again within the next two years.

Or

The column did not reach the wind load, due to ground movement.

Short-term measures to reconstruct the foundation, respectively to the soil conditions, have to be carried out, because the stability cannot be guaranteed. After the reconstruction measures, the column should be tested again to assess material defects.

The column did not reach the single wind load, due to ground movement. Stability is not given. Short-term measures to reconstruct or to maintain the foundation respectively to the ground conditions are necessary.

Plastic deformation

The test load was not reached due to material defects. Due to damage (rust, corrosion or damage) the column reached its yield point before the test was completed and the column is structurally unsound; see Figure A1.5c.

The column does not comply with the requirements as shown above. The column did withstand the wind load, but plastic deformation was detected (exceeding the material's yield point). The column has to be removed as soon as possible.

Materials defects

The wind load was not reached due to material defects; any cracking within the column was identified; see Figure A1.5d.

The column does not comply with the requirements as shown above. The column did not withstand the wind load. Plastic deformation was detected (exceeding the material's yield point). The test load was

Pictures A1.11 and A1.12: Static load testing not reached. Stability is not given. The column has to be replaced immediately.

Dependent on the point of application of the test load and the height at which the bending moments induced by the test load and wind loads are equated, the bending moment induced by the test load at ground level may range from 1.1 to 1.7 times that induced by wind loads. Whereas overloading by a factor of 1.7 appears to be too high, it is appropriate to overload certain critical sections for two reasons:

- To allow for further deterioration due to corrosion and/or cracking before the next test;
- To ensure that severely corroded columns fail the test.

Some sections of columns may be severely corroded but have a bending strength greater than the bending moments induced by wind loads. In this case, columns with large holes (for example the same width as the door opening) may pass a static loading test, but some authorities may prefer to replace those with such severe corrosion.

There are also reasons why overloading is inappropriate:

- The foundations may be overloaded, causing excessive foundation movement;
- Columns with sections, where some corrosion is possible before the strength of the section becomes less than the moment induced by wind and dead loads, may be damaged and fail the test.

The testing organisation stops the test if excessive foundation movement is detected, ie the ground resistance moment exceeds the overturning moment induced by the test load. Therefore, excessive foundation movement can prevent the application of the full test load, and therefore a full assessment of the condition of columns. For this reason, some authorities do not permit static loading in soft ground.

To check that the column material is not damaged in the test, the testing organisation ensures that the bending moment induced by the test load does not exceed the elastic strength of the critical sections calculated using the original wall thickness. However, corroded critical sections may have an elastic strength that is higher than the bending moment induced by wind loads but less than that induced by the test load. A column with such sections would fail the test, as non-linear behaviour would be observed when the metal yielded before the full test load was applied.

Taking all factors into account, it is important that the amount of overloading is not too high so that sound columns do not fail the test, but it is preferable that it is high enough to fail columns with large holes due to corrosion even if they have sufficient strength. At the time of publication, there is no evidence to suggest that large numbers of sound columns have failed the test because they have been overloaded. Furthermore, the test equipment being introduced can apply the load at heights up to 5m, so in future the number of columns that may be overloaded and the extent of any overloading should be less than in the past when columns were loaded at lower heights.

General

If there are serious concerns about overloading, the static loading test could simulate the bending moments induced by wind loads at the lower critical sections of the column only. The higher critical sections could be tested using other methods.

The advantages of static loading are that:

- The strengths of certain critical sections are assessed directly without the need for a full structural analysis;
- The test assesses the effects of both corrosion and cracking at sections where the bending moments are correctly simulated;
- Columns with embellishment kits can be tested;
- The condition of columns below ground is assessed without excavation, including corrosion deep in the root in poorly-drained ground;
- The method can determine the condition of the anchorage and holding-down bolts of flange-plated columns.

The disadvantages are that:

- If the design moments are exceeded, excessive foundation movement may prevent the assessment of the condition of columns;
- Columns with corroded sections that have sufficient strength may fail the test;
- It is difficult to monitor the change in the condition of columns from successive tests;
- When columns fail the test, the cause of failure and the location of the weakest section may not always be identified;
- Reasonable access is needed for the test equipment.

Loss of section monitoring to assess section thickness

A device called a swage joint analyser (SJA) that operates on a similar principle to the relative loss-of-section (RLS) meter has been designed specifically to compare the thickness of an area measuring 20mm by 20mm near hot-swaged joints with the thickness of a reference location of the same area. It can also compare the thickness of any two points on a column.

Loss-of-section monitoring with the SJA is an alternative to the ultrasonic testing described previously. Like the RLS meter, minimal surface preparation is required and the approximate bulk average percentage loss of section is measured. However, the SJA monitors a test area that is sufficiently small to identify localised corrosion and, if necessary, a partial factor could be used to estimate the lowest possible minimum thickness. It is claimed that the SJA is totally unaffected by the protective treatment, including galvanising.

In order to determine the integrity of the full column, the wall thickness should be measured at several locations around the swage. It is recommended that the minimum and maximum thicknesses at these locations are recorded. In addition, the thickness should be recorded at no less than four points around the circumference at the height of the minimum thickness so that the thicknesses can be compared with the minimum wall thickness (uniform thickness around the full circumference). An RLS meter with a mini head can be used to locate the most corroded sections.

The advantages of loss-of-section monitoring using the swage joint analyser are that:

- Minimal surface preparation is required that should not damage the protective treatment;
- Surface rust does not affect the results;
- The cost of testing above ground is low;
- It is effective to assess corrosion from the internal surface of swaged joints;
- Little access is needed for testing above ground;
- Successive tests give an indication of the variation in the condition of columns over time, although small variations in the test locations may be significant.

The disadvantages are that:

- It is difficult to locate the thickness of the weakest sections;
- It is not suitable for assessing corrosion below ground level;

- If it is not designed to take measurements inside the column;
- The bulk average percentage loss of section is measured, not the minimum thickness, although the test area is small;
- Detailed calculations are needed in order to determine the strength of the critical sections;
- Other test methods must be used to assess cracking.

Magnetic particle inspection to detect cracking

Magnetic particle inspection is an established method that indicates the locations of surface cracks and, possibly, near-surface cracks in ferromagnetic materials. Therefore, it can be used on steel columns that are not austenitic stainless steel. The technique uses the principle that magnetic lines of force (flux) are distorted by the presence of a flaw. The flaw is located by fine iron particles that are attracted to the leakage of flux.

The test section is cleaned, and it may be sprayed with contrast-aid paint. An electromagnetic or permanent magnet is placed on the section to produce a magnetic field between the pole pieces. Leakage flux is produced by cracks between the pole pieces, provided they are orientated from around –45° to +45° to line between the pole pieces. After the test area has been magnetised, magnetic ink containing finely divided ferromagnetic particles is applied. The particles have a high permeability so that they produce an indication with small leakage flux. Sufficient ink must be applied so the particles can move over the magnetised surface and collect where there is flux leakage. To detect cracks in all directions, the above test must be repeated with the magnet orientated through 90°. Good light is needed to see the coloured magnetic particles and measure the crack length. Fluorescent inks may be viewed with ultraviolet light.

The method can detect cracks under thinly-painted surfaces: up to 1mm is claimed by one equipment supplier. It may be unsuitable for heavily-corroded sections. Surface irregularities, scratches and surface roughness tend to cause stray surface leakage flux that produces a background indication that can mask real flaws. Crack depth cannot be measured.

Authorities have used magnetic particle inspection to detect fatigue cracking at door openings and, occasionally, cracking at welded joints and near ground level. Few problems have been reported, except difficulties where repaired cracked sections still give indications.

The advantages of magnetic particle inspection are that:
- It is low cost;
- Limited access is required;
- Cracking can be detected under thin protective treatments;
- Successive tests give an indication of the variation in the condition of columns with time.

The disadvantages are that:
- Thorough surface preparation is required;
- It may be unsuitable on heavily corroded or irregular surfaces;
- It may not detect near-surface cracks;
- Crack depth is not measured;
- Detailed calculations are needed in order to determine the strength of the critical sections;
- Other test methods must be used to assess corrosion.

Ultrasonic testing to detect cracking

The detection of cracks and other defects at welds by ultrasonic testing is a well-established technique that has been used in many industries. It is a highly skilled task that is carried out by specially trained staff.

Most crack detection in steel structures is by shear waves. Probes may be used in various configurations, depending on the application. However, for many applications, a probe is placed on the external surface of the test section and the half-skip position and full-skip positions are determined. The half-skip position is where the wave is reflected directly from the base of the weld on the inner surface. The full-skip position is where the wave is reflected back from the upper surface of the weld after reflection from the inner surface of the section. The weld is inspected by scanning the probe between the half-skip and full-skip positions along its full length. Defects produce echoes that indicate their type, location and size. Unlike magnetic particle inspection, ultrasonic testing can detect cracks anywhere in the section, including the internal surface.

Some authorities have used ultrasonic testing to detect cracking at welded joints in columns. At least one test organisation has used the method to detect cracking at hot-swaged joints. It is possible that fatigue cracks can propagate from corrosion sites, but a ductile failure at high strain is thought to be more likely to occur at sections away from welded joints, rather than fatigue cracking. Ultrasonic testing can determine the cracking of the welds in the flange plates of columns, and cracking of the studs securing the columns to the foundation. However, there can be a difficulty in interpreting results from examination of the shaft-to-flange weld where the shaft passes through the flange, due to the joint detail and the proximity of two fillet welds.

The advantages of ultrasonic testing are that:
- Internal cracks can be detected;
- Limited access is required;
- Successive tests give an indication of the variation in the condition of columns over time.

The disadvantages are that:
- Specially-trained staff using specialised conditioning equipment are required, so the cost may be high;
- Thorough surface preparation is required;
- It is difficult to determine the size of defects;
- Detailed calculations are needed in order to determine the strength of the critical sections;
- Other test methods must be used to assess corrosion.

Further comments on 'strength' test methods

It is not possible to recommend one 'strength' test method that is suitable for all types of column. The methods have not been assessed fully because of limited opportunities to observe testing, and to compare test results with the true condition of columns and test results from different test methods.

Static loading has some clear advantages over other methods, provided foundation movement and damage to the column are prevented. It can assess the effect of corrosion deep in the root in poorly-drained ground, the anchorage of flange-plated columns, and testing can proceed if there is a concrete foundation. However, the test does not determine the extent of any deterioration, so it is difficult to monitor the rate of damage and estimate the service life.

Ultrasonic testing requires careful surface preparation, but it may be ineffective on severely corroded columns, and the weakest sections may not be identified. Loss-of-section monitoring requires little surface preparation, and loss-of-section monitoring with the swage joint analyser (SJA) may be an alternative to ultrasonic testing, especially if used in conjunction with the standard or mini transponder heads. However, ultrasonic testing or testing with an SJA below ground requires excavation, and is not possible where there is a concrete foundation. It may be possible to monitor the rate of damage with some forms of ultrasonic testing and loss of section monitoring.

It is concluded that each method has its merits, and it may be preferable to use several methods.

Testing of anchorages

Column anchorage/hold-down bolt testing

The assessment of flange-plate column and mast anchorages is covered in detail in ILP Professional Lighting Guide 7 *High Masts for Lighting and CCTV*. The requirements are summarised as follows:

- It is important that any levelling nuts bear uniformly on the underside of the flange and that all clamping nuts are tightened in accordance with the manufacturer's instructions; this should be checked using a torque wrench.
- Any missing locking or hold-down nuts must be reported; all nuts should have at least three turns of thread visible above the locking nut.
- Testing of studs should be in accordance with the requirements of BS 4124:1991 and BS 5996: 1993; an A-scan ultrasonic flaw detector should be used to carry out the testing procedure on each high mast stud and then to compare the results obtained which should then show any defects.
- If there is no variation between the studs and the readings are unambiguous, then the studs should be considered satisfactory.
- If all the readings appear sub-standard the studs are to be considered corroded and in need of replacement.

Test procedures[1]

Test procedures for either preliminary or proof tests should be carried out in accordance with the CFA guidance note by suitably competent personnel (other than the actual installer of the anchors tested). Prior to carrying out any tests on anchors the tester should examine carefully the structure surrounding the anchor position and note any conditions giving rise to concern that the anchorage may not sustain the required load. Such conditions will include deterioration of masonry units or mortar joints and damage such as cracks across masonry units or in mortar joints. Where a tester is concerned that the structure may not be sufficiently strong then that concern should be included in the test report and it made clear that any test results (even positive ones) do not imply that the structure can take the loads.

Test reports

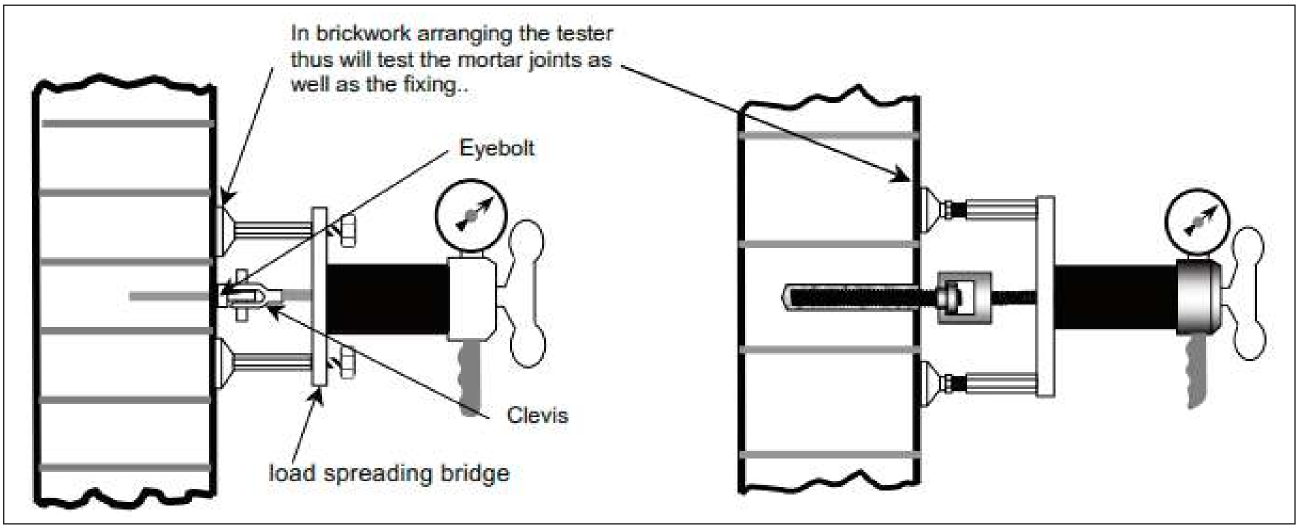
The Construction Fixings Association Guidance note *Procedure for Site Testing Construction Fixings – 2012* contains guidelines for site testing of anchors and the recording of results. Test results should be formally recorded and retained with documentation relating to the project.

Test equipment

Tests and inspections should be carried out using a test meter with a gauge calibrated within the last twelve months to an accuracy of <2% full scale deflection. It is important to use test equipment that applies the load to the anchorage in the most appropriate manner.

Eyebolts should always be loaded individually.

1 Construction Fixings Association (CFA) Guidance note *Anchorage Systems for Seasonal Decorations*



Figures A1.6: Eye bolt testing equipment.
Left: Suitable arrangement for testing eyebolts; Right: Suitable arrangement for testing individual studs

Individual stud anchors used in groups of four to fix square plates should also be loaded individually at the time of installation but at the time of regular inspections square plates may normally be loaded as a group via the eye, see above illustration. However, if there is any evidence of damage to or deterioration of any of the four anchors fixing a plate then all four anchors should be tested individually. Possible problems may include bending of the studs, bending of the plate, cracking of mortar joints or apparent looseness of the anchors or nuts.

Testing the studs individually avoids the possibility – which exists when the plate is loaded via the central eye and if the plate is stiff – that only two studs on a diagonal are actually loaded, thus doubling the test load on those anchors and meaning that the anchors on the other diagonal are untested, or that possibly just three of the four anchors are loaded with a proportionate increase in test load and in this case one anchor of the four not being tested.

Test rig dimensions

Test rigs should be arranged such that the reaction loads are taken sufficiently far from the anchor so as not to influence the result.

In the case of anchors set in concrete a distance of 50mm from the anchor centreline to the closest support of the test rig is sufficient (as proof loads only are being applied). In the case of individual anchors (for example eyebolts) set in brickwork, this means ensuring the feet of the bridge do not rest on the masonry unit being tested.

If stud anchors are tested individually, with the plate in situ, by spanning across the corner of the plate, care must be

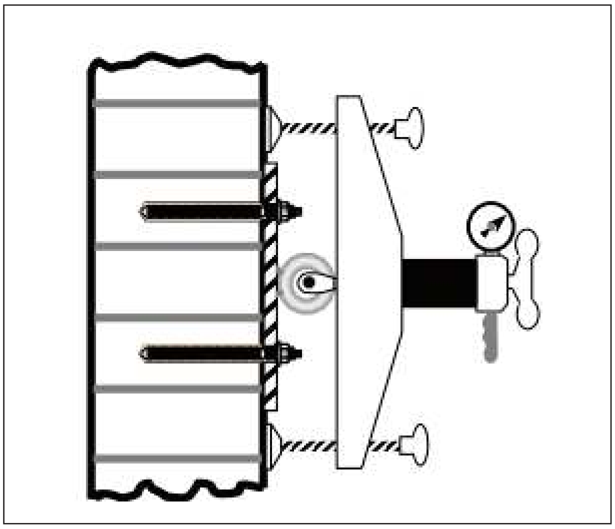


Figure A1.7, Bridge large enough to span the plate to test all 4 fixings via the eye taken to ensure that the feet of the bridge do not sit on the plate or on the same bricks as the fixing under test. Alternatively, the plate may need to be removed.

The procedure for the testing of anchorage is detailed within Appendix 1 of the Construction Fixings Association (CFA) Guidance note *Anchorage Systems for Seasonal Decorations*. The requirements are the same for all anchorages no matter what they support, including wall-mounted luminaires, flange-plate columns, catenary lighting systems etc.

Other test methods

Corrosion potential and current monitoring

The purpose of these tests is to detect and measure corrosion occurring in the root of columns (Rose, 1997). Some excavation is required for columns in a paved or asphalt area, but not for columns in soil or grass.

The corrosion potential determines the corrosion state of the root or in simple terms whether or not the steel is corroding. The corrosion depends primarily on the conductivity, pH and oxygen content of the soil. Contamination of the soil with aggressive anions such as chlorides from de-icing salt will increase the risk and rate of corrosion. Contamination from dog urine is thought to have similar effects. Corrosion cells can be formed at places where differences in oxygen content and pH occur. The oxygen concentration, for example, is generally high near the soil surface and decreases with increasing depth. The corrosion process is electrochemical in nature and consists of two reactions, anodic and cathodic. The anodic reaction is an electrochemical oxidation where electrons are produced at the steel–soil interface and pass into the steel. These electrons are conducted electronically through the steel to the site of the cathode reaction where they are consumed in an electrochemical reduction reaction. Negative ions are produced at the steel–soil interface at the cathode and these negative ions are conducted electrolytically through the soil towards the anode thereby completing the electrical circuit. For the steel–soil situation the anodic reaction is the conversion of iron atoms to iron ions, ie the steel dissolves and its cross-section is reduced.

The cathodic reaction is usually the reduction of oxygen to form hydroxyl ions.

The cathodic reaction does not involve the steel that remains unaffected at this site. Both the anodic and cathodic reactions occur at the same rate in order to satisfy the law of charge conservation so, if the rate of either reaction can be reduced, the rate of the other reaction will be similarly reduced. The corrosion process on structures is often controlled by reducing the rate of one of the electrochemical reactions. The above discussion of the corrosion mechanism indicates that the cathodic reaction usually occurs where the oxygen content is relatively high. For a column, therefore, the cathodic reaction usually occurs on the steel near the ground level whereas the anodic reaction occurs on the column root deeper in the soil. It is for this reason that corrosion (metal dissolution) can be taking place on the root without any indication on the above ground parts of the column. Corrosion caused by differential oxygen cells of this type is common for structures partially buried in soil.

The electrochemical reactions generate a potential difference across the steel/soil interface. This cannot be measured directly but can be measured in relation to a standard reference cell. This gives a steel potential – often termed a half-cell potential (the steel in soil forms one half of the cell, the reference cell the other half). The value of the potential is used to estimate whether the steel is immune from corrosion or in an active or passive state. An active state implies that corrosion is taking place. A passive state implies that the steel surface is covered by a protective oxide film. Passive films are formed on steel in alkaline environments if aggressive ions are absent. The corrosion potential test only estimates whether corrosion is occurring; it provides no information on the rate of corrosion.

The potentials measured are influenced by the conditions in the soil notably oxygen content, alkalinity and contaminants. Potentials will also be affected by coatings, particularly galvanising or concrete on the steel surface. Hence interpretation of the results of potential measurements is not straightforward and requires specialist knowledge.

Corrosion potential measurements are quick and easy to take for columns embedded in soil. A copper–copper sulphate reference electrode is inserted in the soil adjacent to the column and connected to the negative terminal of a voltmeter. The column is connected to the positive terminal of the voltmeter. The voltmeter should have an input impedance greater than 10MΩ.

The corrosion current method aims to measure the rate of corrosion. It is based on a technique called linear polarisation resistance. It requires, in addition to the copper–copper sulphate reference electrode used for potential tests, a stainless steel counter electrode which is inserted in the soil about one metre from the column. The resulting three electrode system (column, counter and reference electrodes) is used in the following way:

- The corrosion potential is measured;
- A constant current (I) is passed between the column and counter electrodes using a potentiostat;
- The potential shift (≈E) between the column and reference electrodes, caused by the current, is measured;
- The ratio ≈E/I is called the polarisation resistance and this is inversely proportional to the corrosion current, which is a measure of the corrosion rate.

This technique provides a measure of the corrosion current at the instant of measurement. It does not provide information about any corrosion which may have taken place in the past. Furthermore, it is difficult to convert the corrosion current measurement to a rate of loss-of-section thickness because:

- The area of steel tested is indeterminate;
- The proportion of this area which is corroding tends to vary in an unknown way, since the corrosion is unusually localised rather than uniform;
- Thus, the measurement of corrosion currents provides an indication of the future rate of deterioration. It does not provide reliable information about the current condition of the column root.

To summarise, the corrosion potential and current monitoring can be used to indicate:

- Column roots that are in a passive (non-corroding state);
- The rate of corrosion for roots of columns that are corroding.

Some authorities have found the method to be less effective in practice, and they have removed columns in a satisfactory condition unnecessarily. This may be because the effect of the differences in the ground conditions along the rows of columns was far greater that the effect of the differences in column condition.

Modal analysis

Modal analysis is the study of the response of a structure to a force input that varies with time. The force input may be wind, mechanical, or even acoustic excitation and it may contain components of single or multiple frequencies. The magnitude of the response will depend on the point of excitation, the frequency content of the force input, and the modal properties of the structure. The modal properties are the resonant frequencies, mode shapes and damping values of the modes of vibration.

The mode shapes are the shapes that a structure takes when it vibrates. They are different for each mode of vibration. Most mode shapes have positions called nodes where there is no movement. The positions between the nodes where the amplitude of movement is a maximum are called anti-nodes. The mode shape of the fundamental mode of vibration of a column, whose period of vibration is required to calculate factor β, has a node at or near ground level and an anti-node at the top of the column. The second and third modes of vibration have higher resonant frequencies. They may have a node at ground level and either a node at the top and an anti-node part way along the column, or an anti-node at the top and a node part way along the column. Modes at higher resonant frequencies have more complex mode shapes.

Some modes may involve movement of the cross section of hollow structures with different numbers of nodes around the circumference. In a breathing mode, the circumference simply extends and shortens. In a translation mode, for example the fundamental mode described above, the cross section moves backwards and forwards. In an ovalling mode, the cross section literally ovals and has four nodes around the circumference. Higher frequency modes may have six, eight, ten etc nodes around the circumference.

Regular sections have orthogonal pairs of modes that are coincident with the orthogonal axes. The pairs may be reasonably well separated in frequency if there are clear differences in geometry. Symmetrical cross section modes may have modes very closely spaced in frequency.

Defects in some structures have been identified from changes in the mode shapes or the resonant frequencies, and a similar approach could be applied to columns. The hammer tapping or aural/tap test on steel columns, described in the previous section, is a simple form of modal analysis, but more sophisticated methods could be investigated. However, the difficulty in using modal analysis to detect defects is to identify the modes that are sensitive to the defects.

Work on some structures has shown that defects must be very severe to observe measurable changes in the lower-frequency modes of vibration. The lower-frequency mode shapes appear to be more dependent on the average condition of the structure rather than each defect. Therefore, a significant defect in one critical section may go unnoticed.

It has been easier to detect defects on structures from the higher-frequency modes, such as those excited by hammer testing. However, because columns have a large number of potential modes, and there are large differences in their geometry, it may be difficult to identify the sensitive modes for each and every column. The resonant frequencies may be so close to each other that modes of one type may be confused with those of another type without a large amount of instrumentation and data analysis. Even then, the mode shapes may be more dependent on the average condition of the test section than the details of the defect.

Harvey (1999) and the University of Southampton have also suggested that modal analysis could be used to determine the condition of columns, but as no highway authorities have reported using modal analysis to detect defects on columns it is difficult to draw firm conclusions.

It is concluded that the apparent success of hammer tapping on steel columns suggests that a form of modal analysis may be a good 'indicative' test. It is unlikely that modal analysis could determine the wall thickness or the extent of cracking in columns with a reasonable degree of accuracy, because of the technical difficulties and the large range of column geometries. The cost of testing may be too high.

Radiography

Radiation, in the form of x-rays or gamma rays, can penetrate materials and create an image on a photographic film. Alternatively, a modified image may be observed directly on a television screen. The thinner the material the less radiation it absorbs. Therefore variations in the intensity of the image can be used to determine the thickness of the material and any defects such as cracks.

X-rays have been used extensively to detect cracking in welds in steel structures. The methods would be ideally suited for columns if it were not for the health and safety issues of working with radiation.

Penetrant dye inspection

Crack detection in steel columns may be enhanced by penetrant dye inspection. The test surface is cleaned to bare metal and a coloured dye sprayed on which is left to soak into cracks. The surface dye is wiped off and a developer is sprayed on to the surface. The dye in the crack is absorbed by the developer to leave a coloured line on the surface. The method is not suitable for cracks clogged with dirt or paint or those that are very tight. For this reason and because of the surface preparation required, this method is probably unsuitable for columns.

Appendix 2
Understanding lighting column design

BS EN 40 Lighting columns

The aim of this section is to summarise the various aspects that should be allowed for by the designer or specifier when they are considering the requirements of a column. It looks at the impact of specifying above the requirement, perhaps specifying an extra heavy column compared to an extra light (as defined within EN 40), and the impact on the design and loading of the whole structure including the foundation. It also makes comment about 'modifying' a column such as the impact of removing a bracket and using a post-top/post-mounted luminaire.

BS EN 40 is the main standard regarding lighting columns, covering design, loadings, calculations, materials, definitions, and tolerances. It comes in seven parts, and an eighth to cover wooden structures is being considered.

The seven parts cover:
Part 1 Definitions and terms
Part 2 General requirements and dimensions
Part 3 Design and verification
- Part 3-1 – Specification of loadings
- Part 3-2 – Verification by testing
- Part 3-3 – Verification by calculation

Part 4 Concrete
Part 5 Steel
Part 6 Aluminium
Part 7 Fibre-reinforced polymer

Understanding PD 6547

Introduction

BS EN 40 is a complicated design standard so PD 6547 was developed to provide guidance on the additional information that is required to allow the design of suitable lighting columns.

It is for the designer to specify the requirements for the columns.

PD 6547: 2004 – Guidance on the use of BS EN 40-3

PD 6547 provides guidance on the information which a designer must consider and provide when specifying any lighting column. This will enable a suitable column to be designed, and covers the areas omitted from EN40.

Specifically, it requires the designer to consider terrain categories, partial safety factors, and deflection classes. It also created the concept of rationalized wind loading factors for all UK administrative areas to allow a single wind factor to be used.

It also provides guidance on foundation design, road signs and banners, and fatigue as these were not included in the EN.

Wind loading calculations

The formula for calculating the wind loading on a column under BS EN 40 is as follows:

$$q(z) = \delta \times ß \times f \times Ce(z) \times 0.5 \times p \times (Cs)^2 \times V_{ref}^2$$

δ, ß, p, Cs	Factors or numbers all fixed in EN 40
f	Topography Factor (1.0)
Ce(z)	Terrain category – based on location exposure
$V_{ref} = C_{alt} \times V_{ref}0$	Wind speed at sea level
C_{alt}	Altitude factor which takes into account altitude of the site

The formula is complicated and requires input from the specifier for factors such as terrain category, wind speed and site altitude which are all discussed in PD 6547.

Effects of altitude on wind pressure

The reference wind velocity used in the calculation of the reference wind pressure is derived from the BS EN 1991-1-4 map which is referenced to sea level.

This therefore requires adjustment to the altitude of the site where the column is likely to be installed.

The wind speed increases with altitude, with a 25% increase in speed at 250m.

It should be noted that wind pressure is the square of wind speed so there is a 56% increase in pressure between altitudes of 0m and 250m.

PD 6547 introduced a rationalised wind loading factor (RWF) which combines the effects of wind speed and altitude.

Rationalised wind loading factor – RWF

The concept here is to have a common factor representing the wind speed and altitude for a UK administrative region. This presents some difficulties due to:

- The maximum UK wind speed varies from 21.5m/sec to 29.5m/sec, a 37% difference, depending on location;
- Wind pressure is the square of speed, so 1.37 × 1.37 means an 88% increase;
- If altitude increases from 0m to 250m wind pressure increases by 56%;

This information is presented in Table A.1 of PD 6547 and is based upon research with all UK authorities to determine the maximum altitude at which they would realistically be likely to install a lighting column.

The following information is taken from Table A.1 in PD 6547 a sample of which is shown in Table A2.1.

The term 'Rationalized region' has been taken from PD 6547 which says:

As an alternative to specifying the information detailed in Clause 3 (Wind Pressure), the specifier may use rationalized wind loading factors (Rwf). These have been calculated using the 10-minute mean wind velocity specified for each administrative region of the United Kingdom and adjusted for altitude.

PD 6547 goes on to say

The use of rationalized wind loading factors should be limited to a maximum site altitude of 250m above sea level. For sites above this height expert guidance should be sought.

The use of rationalised wind loading factors should be limited to a maximum site altitude of 250m above sea level. For sites above this height expert guidance should be sought.

It will be noted that the 10-minute mean wind velocity for Essex and Staffordshire is the same yet the rationalised region is

Table A2.1 Sample of table A.1 from 6547

Administrative area	BS EN 1991-1-4 10-minute mean wind velocity (m/sec)	Rationalized wind loading region	Maximum altitude (m)	Rationalized Wind Factor RWF (N/m^2)
Greater London	22.0	Extra Light	132	350
Essex	22.5	Light	178	396
Staffordshire	22.5	Medium	226	429
Shropshire	23.0	Heavy	250	466
Durham	24.0	Extra Heavy	250	576

different; this is a result of the difference in expected maximum altitude for a lighting column.

It is important that the designer specifies the correct rationalized region. Over-specifying the RWF has a direct effect on the column design. For example, using Light instead of Extra Light adds up to 13% extra loading, not just for the column but also to the foundation.

It has been noted that in a number of authorities that Extra Heavy tends to be specified as this is considered to be 'better' that say Extra Light as required

Category I
Described as rough open sea, lakeshore, smooth flat country without obstacles
Multiplier factor at 10m = 2.78

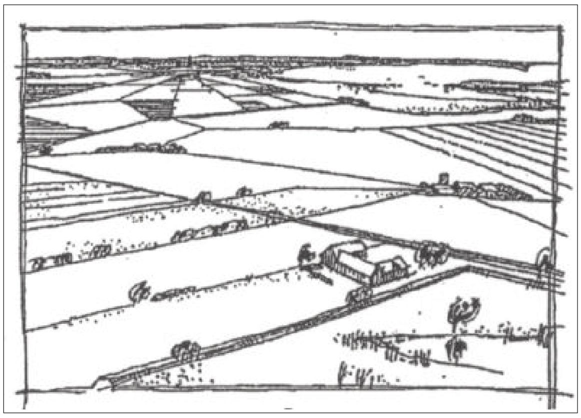


Figure A2.1: Category I

Category II
Described as farmland with boundary hedges, occasional small farm structures, houses or trees
Multiplier factor at 10m = 2.35

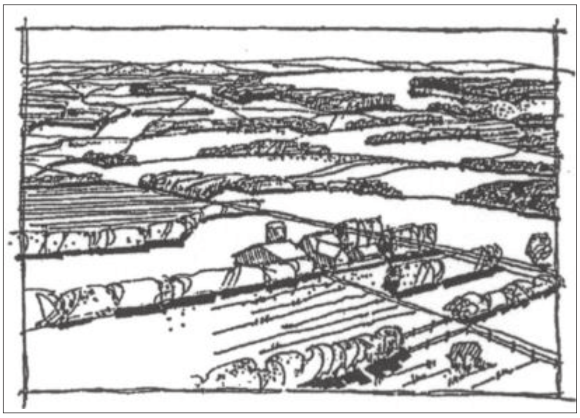


Figure A2.2: Category II

Category III
Described as suburban or industrial areas and permanent forests
Multiplier factor at 10m = 1.78

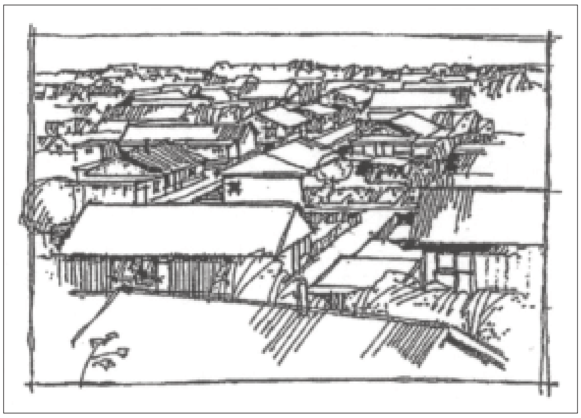


Figure A2.3: Category III

Category IV
Described as urban areas in which at least 15% of the surface is covered with buildings and their average height exceeds 15m
Multiplier factor at 10m = 1.56

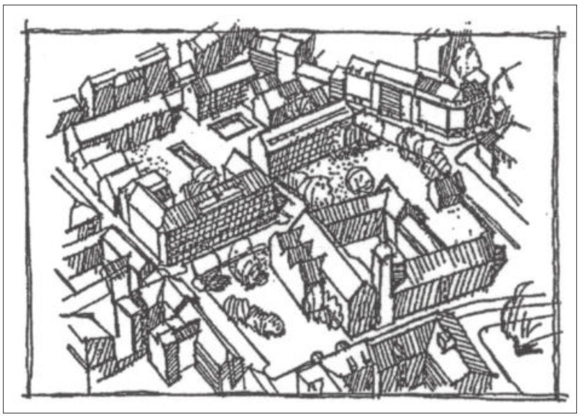


Figure A2.4: Category IV

Permission to reproduce extracts from British Standards is granted by BSI Standards Limited (BSI). No other use of this material is permitted. British Standards can be obtained in PDF or hard copy formats from the BSI online shop: www.bsigroup.com/Shop

under PD 6547 for Greater London. This approach adds up to an additional 65% extra loading over that required, and hence requires even larger foundations.

It may be noted that the most onerous wind speeds normally occur in winter and mainly in November and February.

Terrain categories Ce(z)

The terrain category takes account of the variation of exposure with respect to height above ground level and depends upon site conditions.

There are four categories as given in BS EN 1991-1-4 Annex A.1 and these are summarised in Figure A2.1 to A2.4:

In summary, the large variance between terrain category and wind pressure can be seen.

- Cat I (Figure A2.1) Ce(z) = 2.78
- Cat II (Figure A2.2) Ce(z) = 2.35
- Cat III (Figure A2.3) Ce(z) = 1.78
- Cat IV (Figure A2.4) Ce(z) = 1.56

Within the UK the following recommended terrain categories apply as being the most commonly found.

- Category II is recommended in PD 6547 for columns 8m and above
- Category III is recommended in PD 6547 for columns below 8m

However, there are some exceptions where, due to location, alternative terrain categories or special considerations must be made. These are described as follows:

Where a column is located within 5km of the coast, then the terrain category should be increased by a factor of one (50% of all local authorities have a coast line).

- Columns of 8m and greater should use Category I
- Columns less than 8m should use Category II

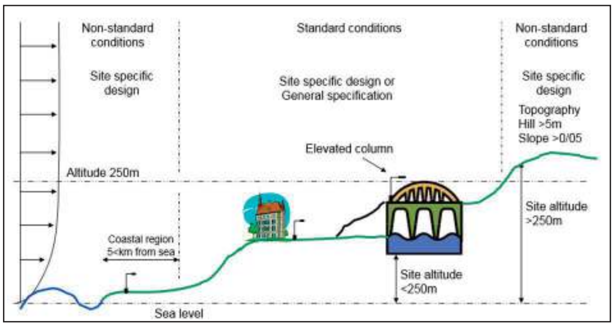


Figure A2.5: Conditions covered by PD 6547

Where columns are to be located on bridges then specialist advice must be sought and the designer must provide the height of the bridge deck above ground level.

The above information can be summarised in Figure A2.5.

The designer must use the appropriate terrain category, as using Cat II instead of Cat III adds up to 22% to the loadings, which will be reflected in the column foundation design. Terrain category is linked to column height.

Other things to specify

In addition to the above, EN 40 and PD 6547 require the specifier to advise on the following factors.

Partial safety factors

For partial safety factors on loads for which EN 40 specifies two options, Class B is normally adopted within the UK.

Horizontal deflection

This relates to the permitted deflection of the column bracket with luminaire fitted that will not adversely affect the lighting distribution; Class 3 is specified within the UK. Note however that Highways England specifies Class 2 which gives less deflection.

Foundation data

BS 5649 included a design method for foundations based on the quality of the soil defined as a Ground Factor (G) in summary as:

- *Good*: compact, well-graded sand and gravel etc;

Table A2.2: PD 6547 Check list	
Mean hourly wind speed Vref	Table A2.1 Rationalised wind load factor
Site altitude	Table A1 unless > 250m
Topography factor	1.0
Terrain category	II group A columns III group B columns increase if within 5km of sea
Rationalized wind loading	Extra Light/Light/Medium/Heavy/Extra Heavy Note – Extra Light for Greater London
Partial safety factors	Class B
Deflection class	Class 3, note HA use Class 2
Foundation data	Good/average/poor
Road signs, if required	Class A, B or C
Fatigue requirements	BD94/07 (Highways England requirement) to be replaced by CD364
Bracket, luminaire details plus other attachments	

- *Average*: compact fine sand, medium clay, loose course sand and gravel, average soils, drainage sufficient that water does not stand on the surface;
- *Poor*: soft clay, poorly compacted sand, clay with silt, poor soil; normally wet with poor drainage

The same formula has been introduced as Table 2 in PD 6547 as this is not included within EN 40; it also includes recommendations for back filling.

It should be noted that the formula uses the bending moment at ground level to inform the design of the foundation, so the designer needs to be aware that over-specifying the loads will result in bigger foundations.

Road signs and attachments

This was introduced into PD 6547 to try to get some standardisation.

With respect to attached signage, PD 6547 has three classes which include for a shape coefficient of 1.8, as follows

- Class A – area 0.3m^2
- Class B – area 0.6m^2
- Class C – area 1.0m^2

Signs are considered as being mounted at a height of 2.5m above the footway with a maximum offset of 0.3m. In addition, they are considered to have a shape coefficient of 1.8 and maximum weight of 5kg. For all signs and attachments, the most onerous orientation should be specified.

An authority should try to standardise on a single road sign attachment class.

Other attachments should be specified as required and these will include:

- Hanging baskets;
- Festive decorations;
- Banners;
- CCTV;
- Embellishment kits and decorative brackets.

For such attachments, the shape coefficients supplied by the manufacturers should be used or an assessment undertaken in accordance with BD 94/17 Annex D.

The check list in Table A2.2 should be used by all designers when considering the specification of lighting columns.

Appendix 3
Forms

The following forms are provided for information:

- Lighting inspection pro forma for mild steel columns
- Sample test reporting certificate
- Column removal proforma
- TR22 structural inspection form (SIF/001) – metal lighting column
- TR22 structural inspection form (SIF/001) – concrete lighting column

Proposed codes

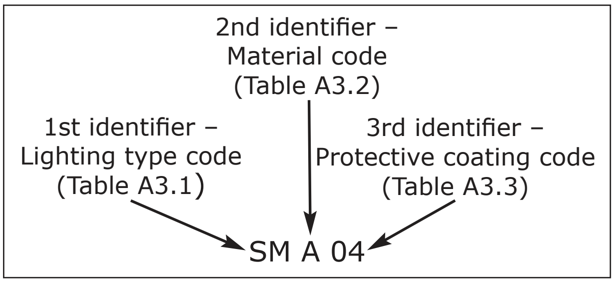


Table A3.1: Column type code		
Type	**Code**	**Description**
Column	COL	Columns – including rooted, cranked, flange plated
Surface/ Pole Mounted	SM	Surface mounted or pole mounted – including wall mounted
Catenary	CAT	Catenary lighting – including lighting sus-pended between columns on cables and lighting fixed to a cable frame where the frame is surface mounted at high level

Table A3.2: Base material code	
Code	**Description**
S	Steel
A	Aluminium
RC	Reinforced concrete
C	Composite
CI	Cast iron
W	Wood

Table A3.3: Protective coating code	
Code	**Description**
00	None/decorative coating
01	Galvanised
02	Galvanised and painted
03	Galvanised and vinyl coated
04	Protective paint only

Tables A3.4: Inspection code (proposed) – Extent codes	
Code	**Description**
A	No significant defect
B	Slight, not more than 5% of surface area/length/number
C	Moderate, 5% – 20% of surface area/length/number
D	Wide, 20% – 60% of surface area/length/number
E	Extensive, more than 60% of surface area/length/number

Tables A3.5: Inspection code (proposed) – Severity codes	
Code	**Description**
1	As new condition or defect has no significant effect on the element (visually or functionally)
2	Early signs of deterioration, minor defect/damage; no reduction in functionality of element
3	Moderate defect/damage; some loss of functionality could be expected
4	Severe defect/damage; significant loss of functionality and/or element is close to failure/collapse
5	The element or protection is non-functional/failed

Lighting inspection pro forma for mild steel columns

Street name:		Road Number:		ID:	
District/Ward:		Column height (m):			
City/Town/Village:		Heritage asset Y/N		Lighting code:	
OSE:	OSN:	Listed status:		Luminaire type:	

Type	No.	Element description	Severity	Extent	Cost £k	Comment
Columns	1	Foundation				
	2	Flange plate				
	3	Flange gussets				
	4	Holding-down bolts				
	5	Column base				
	6	Column shaft				
	7	Door				
	8	Door surround				
	9	Shoulder				
	10	Bracing (inc fixings)				
	11	Elbow				
	12	Luminaire				
	13	Bracketry (incl fixing)				
	14	Spare				
	15	Spare				
Wall mounted	16	Substrate				
	17	Anchorage bolts				
	18	Wall plate				
	19	Gusset plates				
	20	Bracketry (incl fixings)				
	21	Luminaire				
	22	Spare				
	23	Spare				
Catenary	24	Substrate				
	25	Fixing bolts/eyes				
	26	Catenary/luminaire fixing				
	27	Luminaire				
	28	Cabling				
	29	Spare				
	30	Spare				
	31	Spare				
Protective coating, ancillary items and embellishments	32	Painting system				
	33	Galvanising				
	34	Bitumen coating				
	35	Cameras				
	36	Monitoring equipment				
	37	Banners/flags				
	38	Hanging baskets				
	39	Embellishment kits				
	40	3rd party attachments				
	41	Spare				
	42	Spare				
	43	Spare				
Date of inspection						

Continued...

Lighting inspection pro forma for mild steel columns (continued)

Inspector's comments

Name: ________________________ **Signed:** ____________________ **Date:** ______________

Engineer's comments

Name: ________________________ **Signed:** ____________________ **Date:** ______________

Work required

Ref. No	Suggested Remedial Work	Priority	Estimated Cost	Action/Work Ordered

Sample test reporting certificate

Client	**Structural testing & analysis report**	*Test house*

Asset type:	
Asset reference:	

Location:		Asset description	
		Attachments	
		Photo(s) reference	
		Inspection / test date:	
		Re-test if applicable	

Inspection / assessment grid (score R, A, G)				% rating	Comments
Visual inspection	R	A	G		
NDA inspection	R	A	G		
Load condition assessment	R	A	G		
Overall comment of result					
Result	R	A	G		

Image asset reference	Asset image

Project:		Reference:	
Route:		Test house contact:	
Client:		Contact number:	
Contact:		Contact e-mail:	

Result explanation:

Declaration.

This Certificate confirms the results of the structural visual inspection / testing performed on the asset detailed in this report by [name of test house]. The validity of this Certificate is based on the information above and should any of the information change due to unnatural causes, then the Certificate will be invalidated. Such causes include vandalism, RTA damage, unapproved additional load, ground movement, groundwork subsequent to the date of structural test, which may adversely affect the column foundation support.

Column removal proforma

COLUMN REMOVAL PRO FORMA

Removal Contractor: ______________________ Date: ______________

1 General Information
Ward ______________________
Street name ______________________
Secondary location ______________________
____
Unit Number ______________________

2 Column Data
Col. type (*if known*) ______________
Col. height (m) ______________
Root depth (mm) ______________
Base circumference (mm) ______________
Base wall thickness (mm) ______________

COLUMN IDENTIFIER CODE (*tick/cross appropriate box, see Information Sheet for code)*

	Base Material					Shaft Material					Cross Section Shape					Root Type				
Code	A	B	C	D	E	A	B	C	D	E	A	B	C	D	E	A	B	C	D	E

3 Reason for Column Removal
☐ Improvement programme ☐ Load test: material failure ☐ Load test: foundation failure
☐ Vehicle Impact ☐ Other (specify ______________________)

4 Condition of Column above Ground Level
Corrosion at joints ☐ YES ☐ NO
Corrosion or cracks at door opening ☐ YES ☐ NO

CONDITION RATING
Damage Extent ☐Not significant ☐< 10% ☐10% - 50% ☐> 50%
Damage Severity ☐Not significant ☐Minor ☐Moderate ☐Severe

5 Condition of Column Root (below Ground Level)

CONDITION RATING
Damage Extent ☐Not significant ☐< 10% ☐10% - 50% ☐> 50%
Damage Severity ☐Not significant ☐Minor ☐Moderate ☐Severe

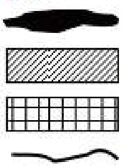
TYPE OF PROTECTIVE COATING ON ROOT (*tick/cross appropriate box, see Information Sheet for code)*

Protective Coating Code	PC1	PC2	PC3	PC4	PC5	PC6	PC7

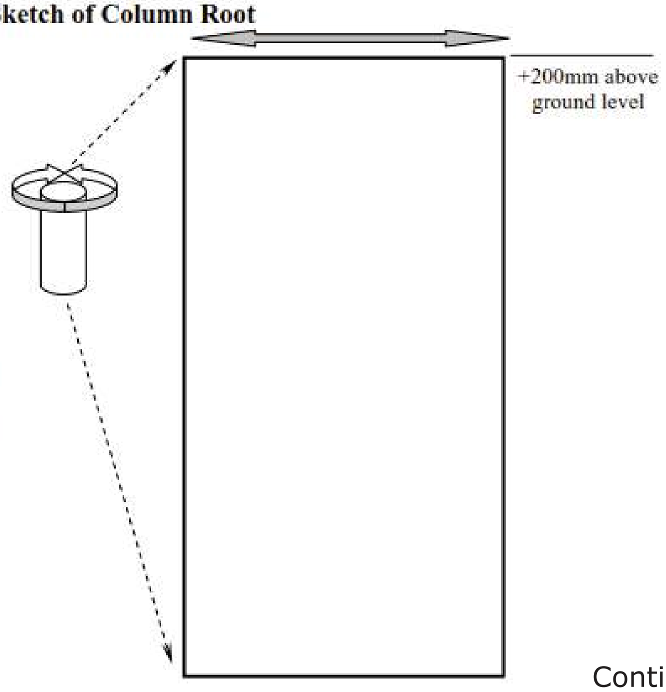
CONDITION OF PROTECTIVE COATING
Damage Extent ☐Not significant ☐< 10% ☐10% - 50% ☐> 50%
Damage Severity ☐Not significant ☐Minor ☐Moderate ☐Severe

6 Record of Corrosion on Exploded Sketch of Column Root

KEY



Perforation
General corrosion
Pitting corrosion
Corrosion bands



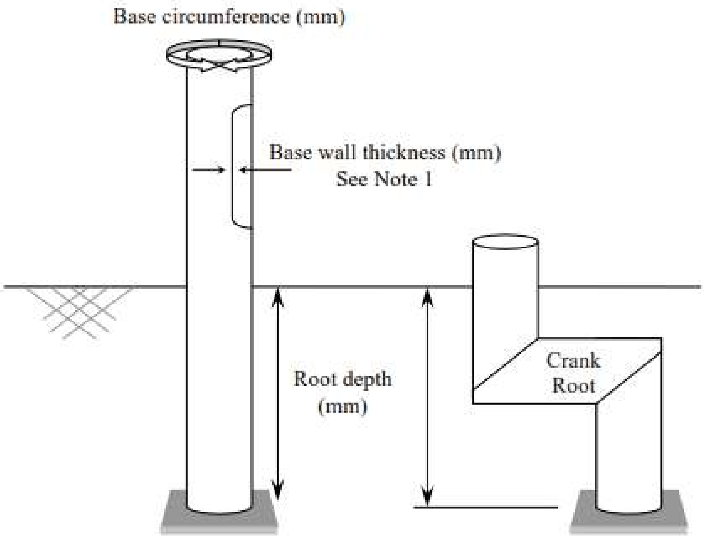
NOTE
The site engineer should approximate and record the position of corrosion features on the exploded column view opposite. The sketch is not expected to be to scale, however the inclusion of measurements describing the approximate size and position of corrosion features would be beneficial.

Continued…

Column removal proforma (continued)

COLUMN REMOVAL PRO FORMA INFORMATION SHEET

GEOMETRY



NOTES

1) Base Wall Thickness
The base wall thickness may be measured at the door opening. The thickness of the door itself must NOT be used as a substitute for the column wall thickness.

2) Base and Flange Plates
a) A base plate is frequently used on planted columns to increase the bearing area; it is not bolted onto the foundations.
b) A flange plate, normally thicker than a base plate, is bolted onto the foundations.

COLUMN IDENTIFIER CODE

Code	Base Material	Shaft Material	Cross Section Shape	Root Type (also see Note 2 above)
A	Steel	Steel	Circular	Planted
B	Cast Iron	Cast Iron	Octagonal	Planted with base plate (no bolts)
C	Concrete	Concrete	Square/Rectangle	Flange plate (bolted down)
D	Aluminium	Aluminium	Ornamental	Crank root
E	Other	Other	Other	Other

COLUMN ROOT PROTECTIVE COATING CODE

Code	Protective Coating	Additional Information
PC1	Decorative paint/none	Decorative paint or no protective coating
PC2	Protective paint	Protective paint treatments are normally difficult to distinguish from decorative
PC3	Red Oxide	Red oxide primers were widely used on cast iron columns
PC4	Aluminium spray	Grey in colour, fine particles, may be covered by paint (external)
PC5	Hot dip (zinc) galvanising	Grey in colour, larger particles, may be covered by paint (internal and external)
PC6	Bitumen	Often used to provide additional protection when only paint is applied
PC7	Other	None of the above

CONDITION DESCRIPTIONS

Extent		Severity	
Code	Description	Code	Description
Not Significant	No significant damage	Not Significant	No significant damage
<10%	< 10% of external surface damaged	Minor	General corrosion
10% - 50%	10-50% of external surface damaged	Moderate	General and pitting corrosion
>50%	> 50% of external surface damaged	Severe	General, pitting and perforations

Inspection forms

Example of a structural inspection form (SIF/001) – metal lighting column; to be used to transfer old TR22 data to the new methodology.

Area	Code	Inspection	No	Excellent 1	Good 2	Fair 3	Poor 4
	a	Number of doors					
A	b	Flange plate condition		No indication of corrosion on root or bolts	Minor internal corrosion and signs of bolt corrosion	Layers of rust within parental metal or bolts corroded	Support leaning of structurally damaged
	c	Base compartment condition		No indication of corrosion on base and protective root coating in tack	Minor internal corrosion on base and protective root coating fading	Layers of rust within parental metal on base and no root protective root coating or poor foundation	Layers of rust with parental metal, support leading or structurally damaged.
	d	Door opening condition		Sound aperture	Minor signs of corrosion	Aperture showing signs of stress. splits	Aperture split
	e	Base compartment shoulder condition		Sound weld no signs of corrosion or stress	Paint loss but sound metal work	Minor signs of corrosion	Welds showing signs of cracks and corrosion/ distortion.
	f	Internal compartment condition		No indication of corrosion	Minor internal corrosion	Layers of rust within parental metal/below ground level	Layers of rust within parental metal thin walls

Continued...

…continued

Area	Code	Inspection	No	Excellent 1	Good 2	Fair 3	Poor 4
B	g	Shaft		Sound external, no rust indication or paint in good condition	External rust spots or paint beginning to flake	25% external rust cover or paint flaking	60% external rust cover or structurally damaged.
	h	Illegal attachment (yes/no)					
C	i	Bracket type					
	j	Bracket/ shaft interface		Sound and sealed spigot attachment	Minor signs of corrosion or loose shaft joint	Missing grub screws or key way, bracket not aligned	Extensive cracking around grub screws or signs of corrosion from missing seals
	k	Elbow/web condition		Sound	Minor signs of corrosion	Signs of stress/corrosi on	Extensive stress, cracking
D		Fold down column					
	l	Hinge point		Sound hinge point.	Minor signs of corrosion around the hinge point.	Layers of rust within parental metal, corroded attachment point.	Extensive cracking/corr osion around hinge point.

Example of a structural inspection form (SIF/001) – concrete lighting column

Area	Code	Inspection	No	Excellent 1	Good 2	Fair 3	Poor 4
A	a	Number of doors					
	b	Base compartment condition		No indication of damage to root base	Minor damage or plugs missing	Signs of stress splits	Support leaning or structurally damaged
	c	Door opening		Sound aperture	Minor signs of damage	Aperture showing signs of stress splits	Aperture split
B	d	Shaft		Sound external, no damage	External damage, slight cracking	Signs of stress splits	Cracks, splits, reinforcing bars showing, areas of concrete missing
	e	Illegal attachment (yes/no)					
C	f	Bracket type					
	g	Bracket/shaft interface		Sound external, no damage	External damage, poor fitting bracket sleeve.	Signs of stress splits, poor connection	Cracks, splits, spalling, reinforcing bars showing, areas of concrete missing

Appendix 4
Overview of the procedure for calculating Lighting Condition Indicator (LCI)

This Appendix provides guidance on the computation and use of Lighting Condition Index (LCI) as a tool in understanding the condition of an asset group.

The condition of a group of assets using the CCI (Column Condition Index) is descriptive of the structural condition using visual inspection data obtained during routine inspections. However, a fuller understanding of the overall asset condition may be obtained using a Lighting Condition Index (LCI). LCI utilises visual inspection data for the following:

- *Structural* – Visual inspection data obtained during planned routine inspections recorded using 'Public lighting inspection pro-forma' (see Appendix 3). Condition is assessed visually, recording the severity (1 to 5) and extent (A to E) of deterioration to visible elements of the asset.
- *Non-structural* – Visual inspection data recording the condition of non-structural elements, for example the lens/bowl and luminaire. The inspection records whether the elements are securely fixed. This is not intended to assess light output as it is solely condition based.
- *Electrical* – Visual inspection data recording the condition of the electrical elements, noting whether wiring appears satisfactory or if the back-board is not secure.

The condition or performance ratings for each of the above functions is utilised by mathematical expressions developed as part of the CCI guidance to produce lighting condition indices (LCI) and lighting stock condition indices (LSCI). The steps involved in arriving at LCI and LSCI are summarised below:

1. Each of the three functions of condition given above is selected in turn and its condition or performance data used to produce a function condition score (FCS).
2. Next, the importance of each of the three functions will be identified and combined with the FCS to evaluate the function condition factor (FCF). Note, structural integrity will always have the highest importance of the three condition functions as this is a safety-critical function.
3. Next the FCS and FCF values are combined to produce a function condition index (FCI) which is an evaluation of each of the 3 condition functions on a scale of 1 (best) to 5 (worst).
4. Then, two different lighting condition scores are evaluated using the same 1 to 5 scale as FCI:
   - LCS_{Av} – which is an average of FCI values of the three lighting condition functions (weighted by the Function Importance Factor, FIF).
   - LCS_{Crit} which is the maximum of FCI values of the safety critical components of the lighting unit, for example foundation, base of column, support brackets and arms.
5. The LCS values are then converted to the corresponding lighting condition indices LCI_{Av} and LCI_{Crit} on a scale of 100 (best) and 0 (worst). These steps are repeated for all lighting units in the stock.
6. Finally, the LCI values are weighted according to the size of the lighting stock to calculate a lighting stock condition index (LSCI). This weighting accounts for the differing stock sizes that exist across the UK. A small authority may be responsible for only

3,000 to 5,000 lighting units (or fewer), whereas a large county council may be responsible for more than 50,000 units, and these differences need normalising. $LSCI_{Av}$ is a weighted average of the LCI_{Av}, and $LSCI_{Crit}$ a weighted average of LCI_{Crit} for all the lighting units in the stock.

Interpreting LCI and LSCI results

Both LCI and LCSI scores are provided out of 100.

A LCI value of 100 implies that a lighting unit is performing at 100% of its service potential – that is, it is operating safely, with full reliability and efficiency, emitting the correct level of lighting; a value of 60 indicates that the lighting unit has lost 40% of its service potential; while a value of 0 implies that the lighting unit is no longer serviceable.

Lighting stock condition indices (LSCI) results are likely to be interpreted as shown in Table A4.1.

Having an accurate inventory of lighting assets in the stock is essential for providing meaningful LCI results. Missing or incomplete data in the inventory has the potential to produce inaccurate results

Table A4.1: Lighting stock condition indices

LSCI range	Lighting stock condition based on $LSCI_{Av}$	Lighting stock condition based on $LSCI_{Crit}$
100 – 95 Excellent	Lighting stock is in *excellent* condition. Very few lighting units in a poor condition.	Very few safety critical issues. Represents a *very low risk* to public safety.
94 – 85 Good	Lighting stock is in a *good* condition. A low percentage of the lighting stock is in a poor or very poor condition.	A low percentage of lighting units in the overall stock may be in a poor condition. Represents a *low risk* to public safety.
84 – 65 Fair	Lighting stock is in a *fair* condition. A moderate percentage of lighting units in the stock may be a poor condition. There is potential for a further decrease in stock condition if maintenance funding is not provided. Moderate backlog of maintenance work.	Some lighting units in the stock may represent a *moderate risk* to public safety unless mitigation measures are put in place.
64 – 40 Poor	Lighting stock is in a *poor* condition. A significant number of lighting units may be in poor condition. Maintenance work historically underfunded and there is a significant backlog of maintenance work.	A significant number of lighting units in the stock may be in a severe structural condition. Some lighting units represent a *significant risk* to public safety unless mitigation measures are in place.
39 – 0 Very poor	Lighting stock is in a *very poor* condition. A significant percentage of the lighting stock may be unserviceable or close to it. Maintenance work historically underfunded and there is a huge backlog in maintenance.	A significant percentage of the lighting stock may have failed structurally or be close to it. A significant percentage of lighting units represent a *high risk* to public safety unless mitigation measures are put in place.

on the condition of the lighting stock or the results are based on a percentage significantly below the actual number of lighting units in the stock. This is particularly so once the calculation of LCI becomes automated using data from an owner's inventory.

To get the most out of the LCI functionality and achieve more granularity of the results, such as understanding the lighting condition in a town/city/district or street, it is important that the inventory has captured this location information and allocated lighting units to one or all of them.

Appendix 5
Probabilistic modelling for Assumed Residual Life

A risk-based approach to asset management utilises probabilistic structural models and a consequence analysis. Risk associated with an asset failure is defined as:

Risk = Probability of failure (P_f) × Consequence of failure (C_f)

The probability of structural failure of a minor structure is calculated using structural reliability analysis. The structural reliability analysis incorporates loading, capacity, and decoration and their associated uncertainties to calculate the probability of failure of an asset. Consequence of failure is usually calculated based on costs and/or injuries associated with failure of an asset.

A good risk-based asset-management strategy requires good quality input data. However, there is unlikely to be comprehensive asset data in the early development of such a strategy. It is common practice to develop a risk-management framework using the limited data in hand and/or expert knowledge. This limited framework and the associated models then analyse the deficiencies of the data, and asset behaviour models are identified. The identified deficiencies of the risk model will be reduced over time by collecting good quality data in the most needed areas. After all, a risk-based model's output is only as good as its input.

Probability of failure of a minor structure

Over the last few decades, use of probabilistic analysis has become common practice for determining the structural reliability of an asset.

The safety margin concept is widely used for calculating the probability of failure of a structural asset:

$M = R - S$ *Eq 1*

Hence $p_f = P(R - S \leq 0)$ *Eq 2*

Where M = failure margin
R = resistance
S = load
P_f = probability of failure.

A graphical representation of Equation 2 is shown in Figure A5.1, and the probability of failure – given that R and S are statistically independent – is defined as

$p_f = P(R - S \leq 0) = \int_{-\infty}^{+\infty} F_R(x) f_s(x) dx$ *Eq 3*

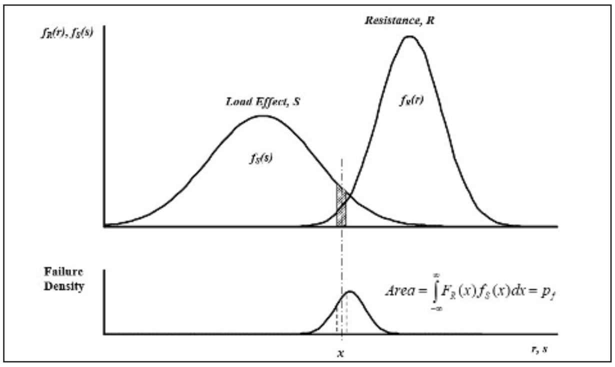


Figure A5.1: Simplified illustration of failure probability for (R-S) safety margin

Equation 2 commonly takes the form of an ultimate limit state, for example shear or flexure, and incorporates the appropriate random variables, for example material properties, dimensions, loads etc. The failure probability may be evaluated using numerical integration, Monte Carlo simulation or asymptotic methods. The asymptotic methods, for example FORM and SORM, are the most suitable for a large variety of structural reliability problems, although advances in computer hardware and software have led to an increased use of the Monte Carlo approach in recent years. Also, in many practical problems involving several random variables and complex non-linear failure margins, Monte Carlo simulation is frequently required as the primary or secondary (checking) analysis.

Consequence of failure

The consequence of a failure is usually calculated based on the costs and injuries associated with different modes of failure of an asset. The consequence of failure of a structural asset is usually determined based on structure:

- Usage;
- Location;
- Mode of failure;
- Time of failure;
- Public perception of the mode of failure.

In order to have a reliable risk-based asset-management strategy, full understanding of a structure and its modes of failure are essential for determining the consequences associated with that structural asset.

Risk-based minor structure maintenance and replacement

Other than electrical failure of minor structures, which can be easily modelled based on the manufacturer's reliability data, modelling of the long-term behaviour of minor structures requires large amount of data to enable the asset manager to accurately understand the environmental factors that cause deterioration over time.

As corrosion is usually the dominant factor in deterioration, good understanding of the minor structure environment and its corrosion protection is required. Periodic visual inspections, non-destructive tests and load tests are good tools to help the asset manager understand and quantify the uncertainties related to environmental factors and corrosion protection technologies.

Using the collected environmental and test data, the asset manager can develop a risk-over-time model for the minor structure stock. Effective use of this model to predict the future behaviour of the minor structure stock and the effectiveness of different maintenance techniques could result in lower whole-life costs of the minor structure assets.

Lighting column assumed residual life (ARL)

For a lighting column to be kept in service the risk associated with it should be within an acceptable range.

The 'current risk' and 'predicted future risk' can be assessed using the developed risk model. The period from current to when the model predicts an unacceptable risk (that is, when a column is considered at risk and should be removed from service) is called the assumed residual life (ARL). This information will assist asset managers to develop forward budget targeting to those assets most in need.

As mentioned previously in this appendix, the risk associated with a minor structure is calculated based on its probability and the consequences of failure. The probability of failure is calculated using the safety margin concept. The applied loads and the load capacity of a minor structure are used to calculate its margin of safety.

In order to be able to model a minor structure's capacity over time all the sources of uncertainties should be identified.

Material strength: There are two main sources of uncertainties associated with a column's material strength.

1. No manufacturing process is perfect and there are always differences between the actual material strength and those based on the manufacturer's data.

2. The lack of good inventory data. Many local authorities and other asset owners do not have accurate – or only limited data – in respect of the column material. This makes estimating material strength difficult other than through material testing.

Column corrosion model: Different material will deteriorate at different rates. In addition, the rate of deterioration will depend on environmental factors as mentioned in Section 6.

Deterioration of metallic materials is typically modelled using a linear mathematical expression and extrapolation over time. Published data can be used to produce theoretical deterioration models. However due to uncertainties the corrosion-rate ranges are usually quite wide.

Field data collected over time can assist in determining loss of material due to corrosion and therefore rate of deterioration. Such information can be used to re-calibrate assumed deterioration models.

Column corrosion protection system: Lighting assets are usually protected against environmental factors using a corrosion protection system. Over time, all corrosion protection systems lose their effectiveness and at some point will no longer remain effective and the column base material will corrode. The point at when corrosion of the base material starts is called the 'Corrosion Initiation Time'. Published data can be used to produce theoretical models for corrosion initiation time. However due to uncertainties the corrosion-rate ranges are usually quite wide.

Field data collected over time can assist in determining the exact time that the corrosion system fails. Such information can be used to re-calibrate assumed corrosion initiation time.

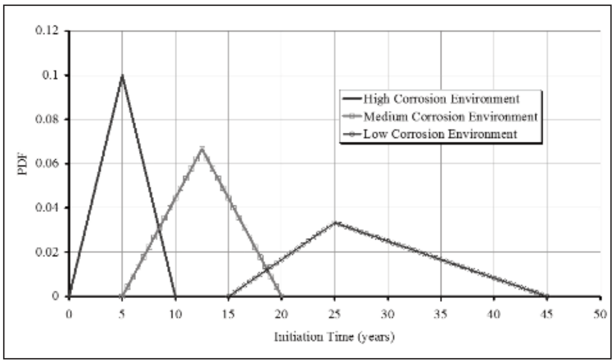


Figure A5.2 Lighting column corrosion initiation time

Using reliability analysis methods and incorporating the above uncertainties, the time-dependent probability-of-failure profile of an asset is developed.

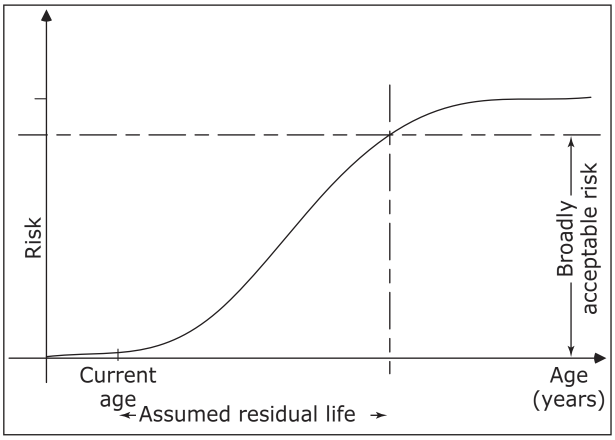


Figure A5.3: Lighting asset risk profile over time

Using this profile in conjunction with the lighting asset failure consequences, the risk profile of the lighting assets is calculated.

This profile is updated in line with the latest collected data to reflect the latest state of the lighting stock. The new data may include visual inspection, non-destructives tests and decommissioning reports.

There are a number of techniques available for updating time-dependent structural reliability. The updating techniques used in this study are:

1. *Classical statistical updating* – When more site data becomes available it is combined with the existing site data to create new variable distributions.

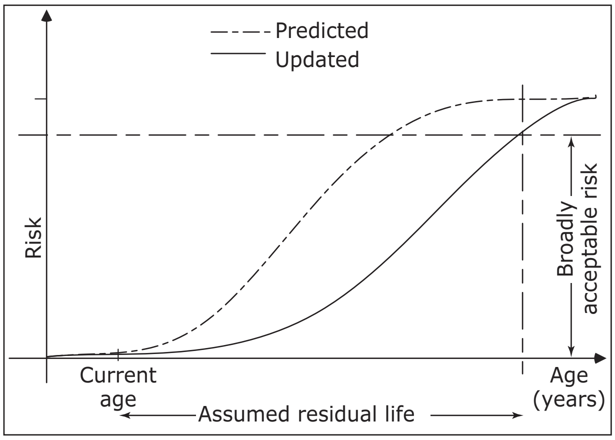


Figure A5.4: Updated lighting asset risk profile over time

Minimal engineering judgement is required in the process. The classical approach requires quantitative data, for example corrosion-loss measurements; qualitative data such as passing or failing a load test cannot be used.

2. *Bayesian updating* – New site data is used to update the prior distributions using a subjective Bayesian approach. Bayesian updating uses quantitative and/or qualitative data.
3. *Conditional updating* – If an event occurs that provides information on a lighting column condition, for example ultrasound readings and/or passing a load test, this is used as an updating condition (a rule that must be satisfied) within structural reliability analysis.

Appendix 6
Process to determine 'Assumed Residual Life' (ARL) for mild steel minor structures

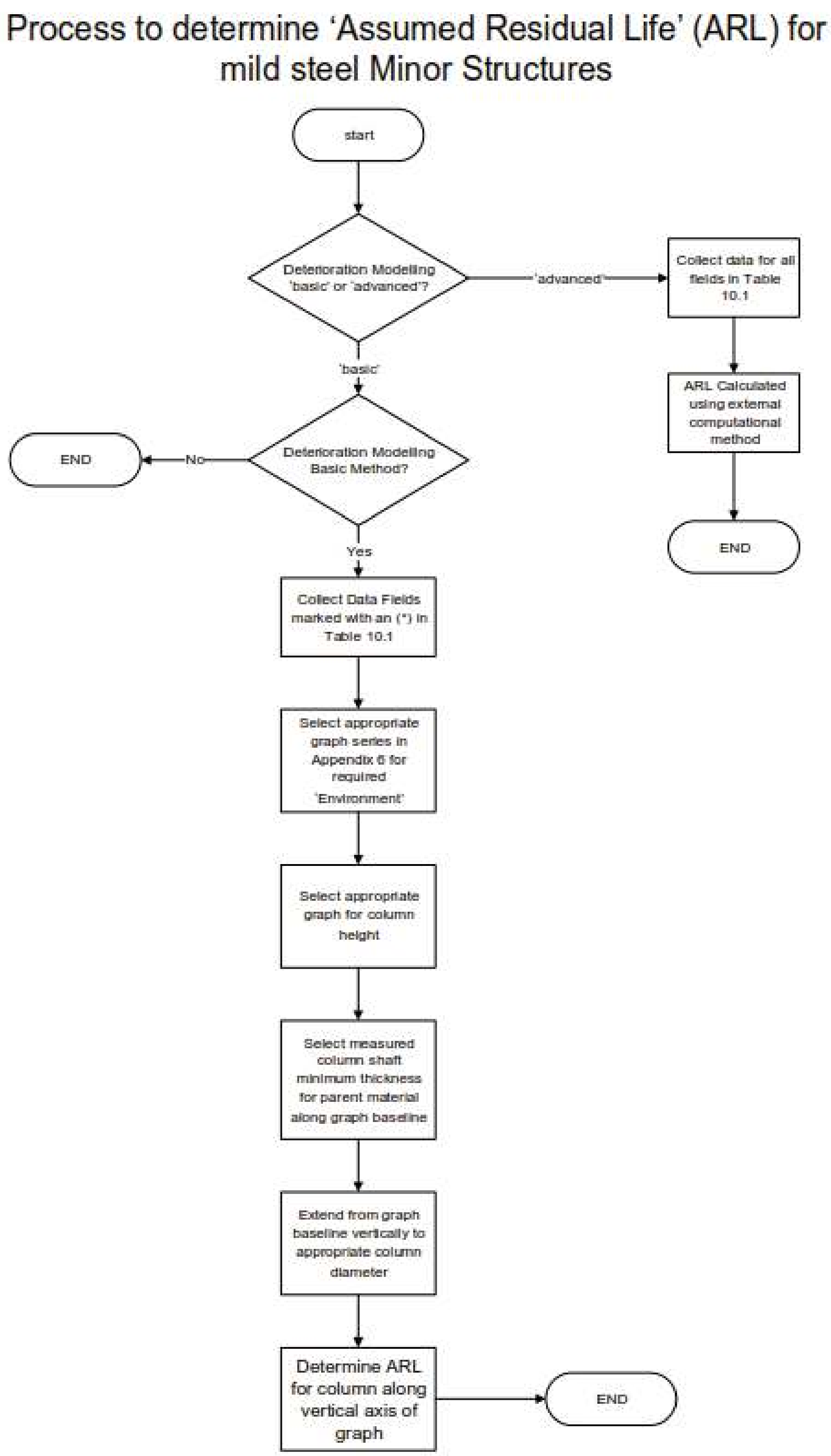


Figure 6.1:

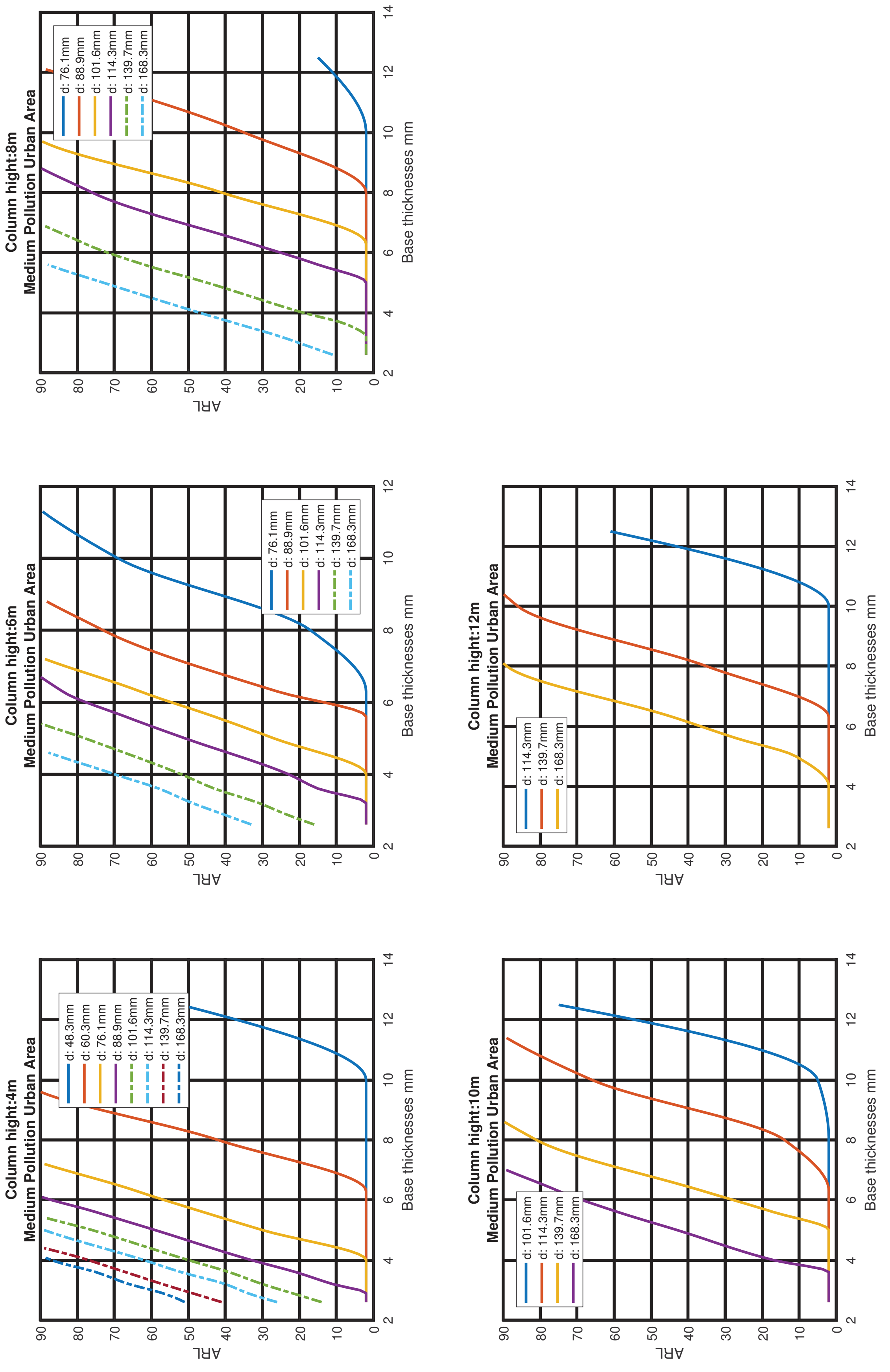

Column hight:8m
Medium Pollution Urban Area
d: 76.1mm
d: 88.9mm
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
ARL
Base thicknesses mm
Column hight:6m
Medium Pollution Urban Area
d: 76.1mm
d: 88.9mm
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
ARL
Base thicknesses mm
Column hight:12m
Medium Pollution Urban Area
d: 114.3mm
d: 139.7mm
d: 168.3mm
ARL
Base thicknesses mm
Column hight:4m
Medium Pollution Urban Area
d: 48.3mm
d: 60.3mm
d: 76.1mm
d: 88.9mm
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
ARL
Base thicknesses mm
Column hight:10m
Medium Pollution Urban Area
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
ARL
Base thicknesses mm

Figure 6.2:

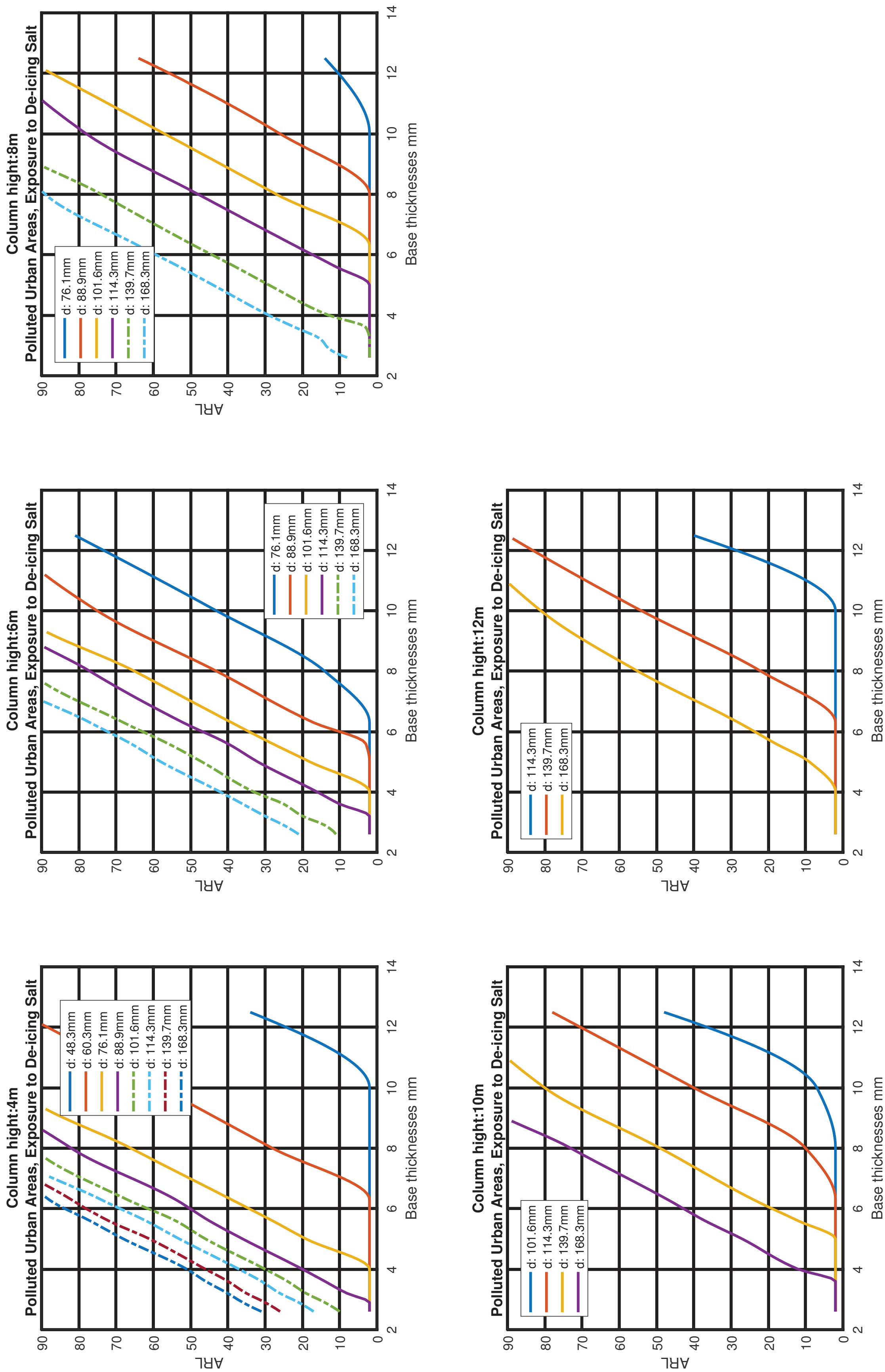

Column hight:8m
Polluted Urban Areas, Exposure to De-icing Salt
d: 76.1mm
d: 88.9mm
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
ARL
Base thicknesses mm
Column hight:6m
Polluted Urban Areas, Exposure to De-icing Salt
d: 76.1mm
d: 88.9mm
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
ARL
Base thicknesses mm
Column hight:12m
Polluted Urban Areas, Exposure to De-icing Salt
d: 114.3mm
d: 139.7mm
d: 168.3mm
ARL
Base thicknesses mm
Column hight:4m
Polluted Urban Areas, Exposure to De-icing Salt
d: 48.3mm
d: 60.3mm
d: 76.1mm
d: 88.9mm
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
ARL
Base thicknesses mm
Column hight:10m
Polluted Urban Areas, Exposure to De-icing Salt
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
ARL
Base thicknesses mm

Figure 6.3:

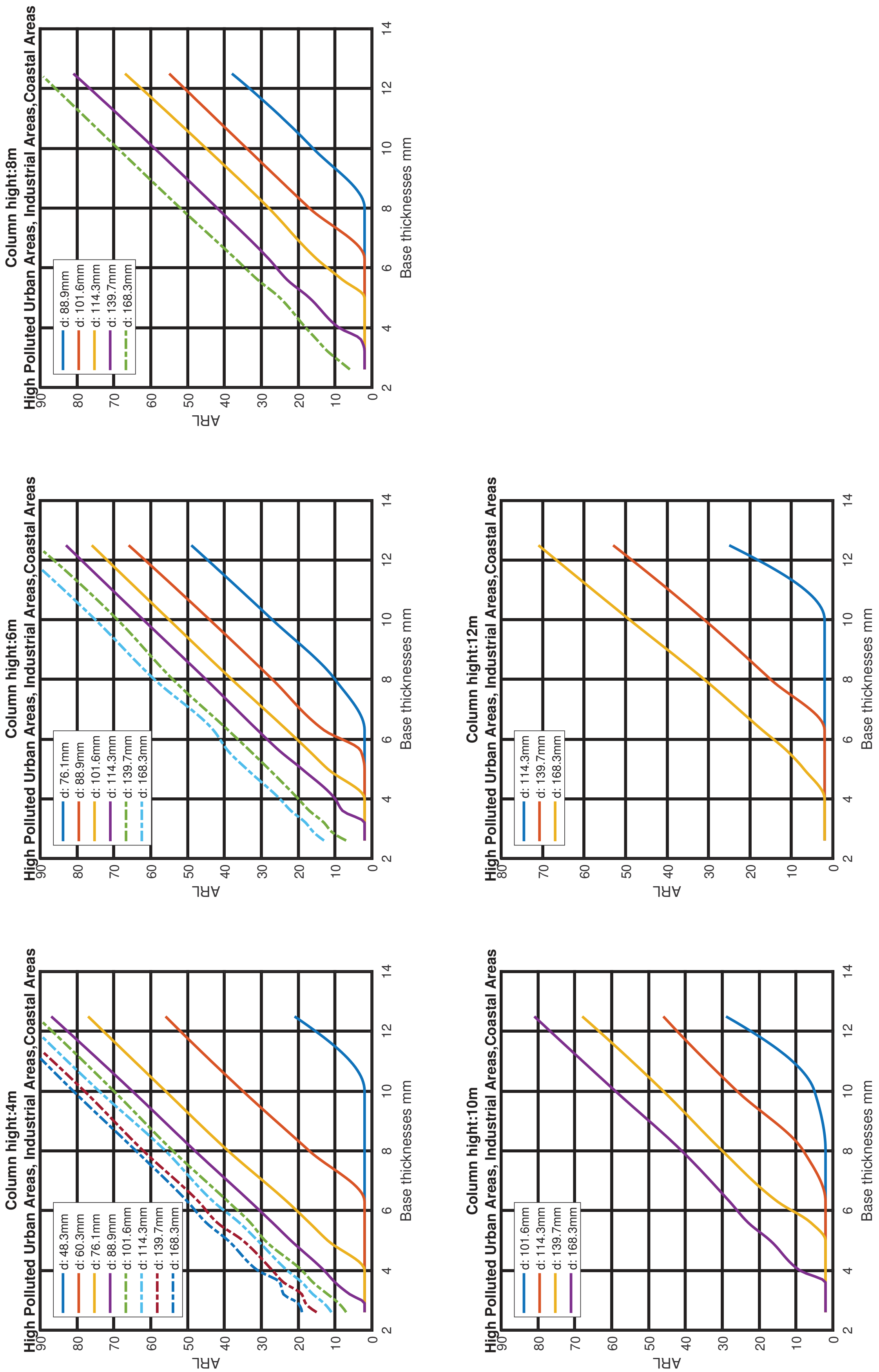

Column hight:8m
High Polluted Urban Areas, Industrial Areas,Coastal Areas
d: 88.9mm
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
Column hight:6m
High Polluted Urban Areas, Industrial Areas,Coastal Areas
d: 76.1mm
d: 88.9mm
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
Column hight:12m
High Polluted Urban Areas, Industrial Areas,Coastal Areas
d: 114.3mm
d: 139.7mm
d: 168.3mm
Column hight:4m
High Polluted Urban Areas, Industrial Areas,Coastal Areas
d: 48.3mm
d: 60.3mm
d: 76.1mm
d: 88.9mm
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
Column hight:10m
High Polluted Urban Areas, Industrial Areas,Coastal Areas
d: 101.6mm
d: 114.3mm
d: 139.7mm
d: 168.3mm
ARL
Base thicknesses mm

Figure 6.4:

Appendix 7
Process to determine Column Condition Index (CCI) for mild steel minor structures

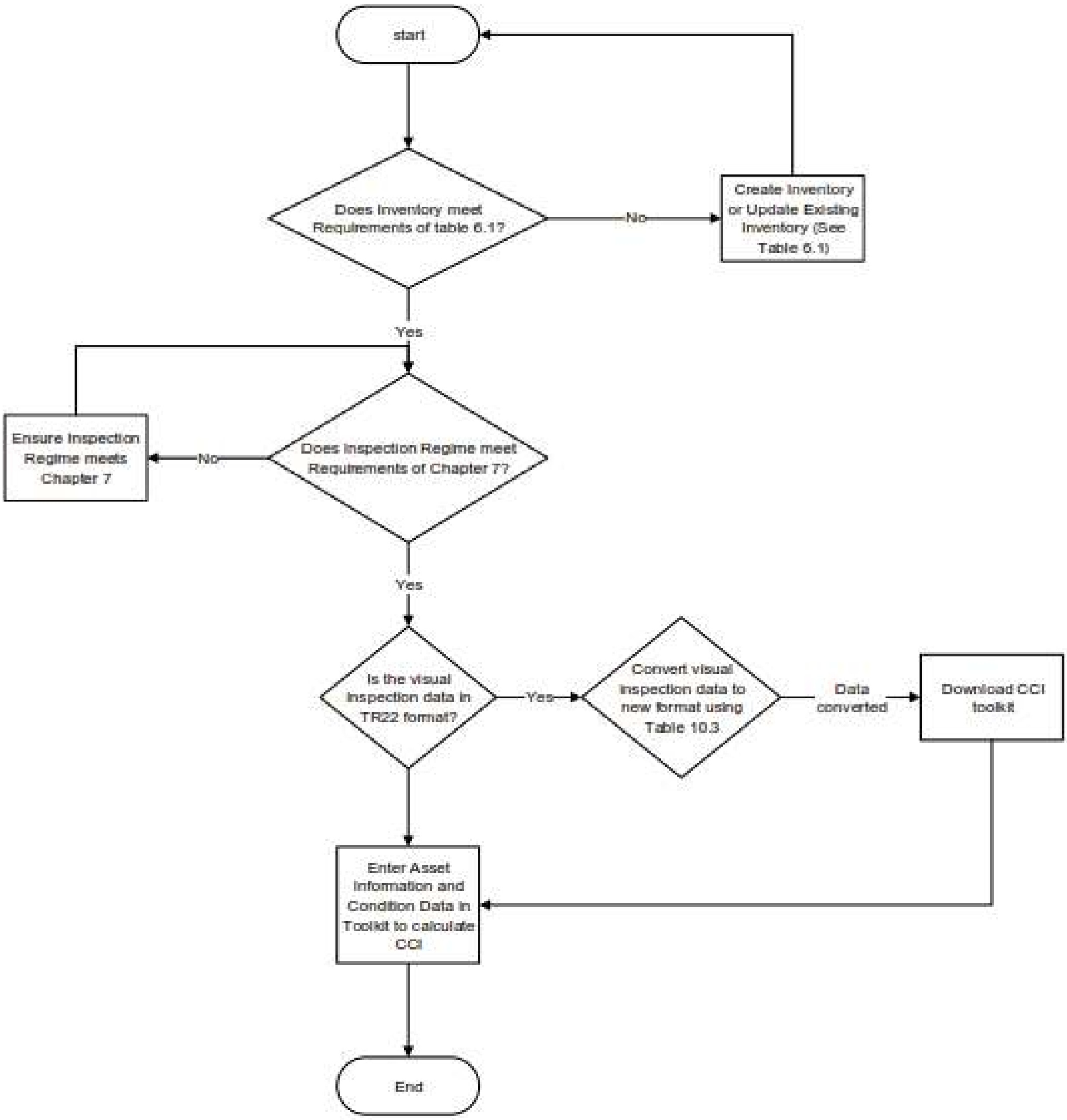


Figure 7.1:

Appendix 8
Strategies

Table A8.1: Strategies			
Regime	**Component**	**Characteristics**	**Comment**
Short-term reactive management	Data	Basic inventory and condition data on main asset types	This approach provides poor value for money and is likely to lead to increasing maintenance backlog and liability claims
	Service level	Comply with statutory requirements	
	Work planning	Condition and defect data used to plan annual works	
	Financial forecasting	Annual budgeting – last year's budget plus a bit	
Basic asset management	Data	Good inventory and condition data	This approach develops a good understanding of the assets and their condition, enabling effective targeting of resources.
	Service	Asset condition and performance standards defined	
	Work planning	Formalised procedure for identification and prioritisation of works that accounts for more than just condition	
	Financial forecasting	Simple models and engineering judgement used to produce five-to-ten-year financial forecasts	
Asset management	Data	Full asset data and data management strategy	This approach provides a robust basis for long-term financial planning; knowledge-based planning; focused on outcomes
	Service level	Outcome/customer-focused levels of service	
	Work planning	Needs identified, future deterioration modelled and the best whole-life value solutions identified and prioritised	
	Financial forecasting	Computerised tools used to model the impact of alternative long-term investment strategies on levels of service	

Appendix 9
Liability, competency, and the Construction (Design and Management) Regulations

Liability considerations

Adoption of the approach and recommendations within this guidance will provide any minor structure asset owner with a robust, auditable assurance of their stock condition against their tolerance of risk. It is important that such support structures are understood and maintained in a condition that any risks are controlled such that any residual exposure is tolerable to the asset owner.

Highways

Taking the highway sector for example:

- The *Infrastructure Act 2015* requires that all reasonable steps should be taken to ensure the continued availability and resilience of the strategic network.
- The *Highways Act 1980*:
  - Section 41 'Duty to maintain…' requires the public highway to be maintained to a reasonable standard.
  - Section 58 'Special defence in action against a highway authority for damages for non-repair of highway' provides a defence in that 'the authority had taken such care as in all the circumstances was reasonably required to secure that the part of the highway to which the action relates was not dangerous for traffic.'

The UKRLG code of practice *Well-managed Highway Infrastructure* supports this, and considers the management of the highway asset based upon a risk and asset-condition assessment process. By following this guide, the highways asset owner will be able to provide a defence in accordance with Sections 41 and 58 of the *Highways Act 1980*.

Rail

Within the rail sector, the Office of Rail and Road (ORR) operates the Risk Management Maturity Model (RM^3) which defines what excellence in risk management looks like, and allows organisations to assure themselves that their risk management is operating to an adequate level.

Liability risk

Essentially this relates to understanding the deterioration of a support structure from its normal conditions, how associated risks and hazards may be considered, as well as the owner's risk tolerance through a process based on inspection and testing.

The following references the Institute of Highway Engineers' (IHE) *Well Managed Highway Liability Risk* which has been developed to support the well managed highway infrastructure code of practice. It is recommended that this is referenced by all asset owners as it is equally of relevance to non-highway asset owners as it to those who manage highways.

https://www.theihe.org/wp-content/uploads/2019/03/LR-IHE-Risk-Liability-Guide-v3.pdf

Tony Kirby, Past President IHE advises:

> *Following on from the publication of the UKRLG document Well-Maintained Highway Infrastructure this guide [Well Managed Highway Liability Risk] seeks to provide further insight and advice on the risk and evidence-based approach to service delivery and the effective management of highway liability risk exposures.*
> *The guidance applies throughout all parts of the United Kingdom and*

particular attention has been given to ensure any specific arrangements within the devolved administrations have been identified.

Document contents

The document lays out and discusses in detail the requirements for:

1. Risk-based approach and highway liability risk management
   Provides practical advice and guidance on applying a risk-based approach to highway liability exposures and will cover how the five activities of the ISO 31000 process can be applied to highway liability risk management as well as key considerations for managing highway liability risks within a risk-based approach.
2. The law on highway liability
   Provides an overview of the fundamental legal principles relating to highway liabilities in England and Wales, Scotland and Northern Ireland. It sets out the main statutory provisions and case law which provide the core principles on which highway claims are considered.
3. Claims management
   Details the claims processes, fraud, and the importance of recording and capturing claims management information, which can support a risk-based approach to highways asset management. Specific references are made to Scotland and Northern Ireland where the claims and litigation processes differ to England and Wales.
4. Training, and assessment of competence
   Sets out the training available and competence requirements (as discussed in further detail in Section 4 of the IHE document) of the various personnel involved in delivering the highway service. It stresses the importance of maintaining training plans and records to evidence the training provided as well as the need to periodically refresh and update training to ensure the continued competence of staff.
5. Information management and record keeping
   Provides guidance on information management and record keeping critical to highway liability management as well as highlighting factors relevant to data protection and freedom of information.
6. Risk ownership when outsourcing services
   Reviews some of the issues relating to transfer of liability risk when services are outsourced. It provides a list of practical recommendations to help avoid conflict and uncertainty around risk ownership, and outlines the items to agree with contractors and include within a claims management protocol.

As with the principles of the HMEP Highway Infrastructure Asset Management Guidance, the UKRLG code of practice *Well-managed Highway Infrastructure recommends* that '*A risk-based approach should be adopted for all aspects of highway infrastructure maintenance, including setting levels of service, inspections, responses, resilience, priorities and programmes*'. This should be developed in accordance with local needs, priorities, appetite for risk and affordability.

The risk-based approach to the lighting infrastructure maintenance as discussed in Section 12 of this guidance is essentially based on an understanding of the lighting asset, the potential risks, and an appreciation of their likely significance.

It is not possible, for reasons of practicality and affordability, for an asset owner to maintain lighting infrastructure in an as-new, defect-free condition all of the time, nor does the law require them to do so. In setting standards for maintenance management, an asset owner should take account of the financial and other resources that it can reasonably make available for the purposes of lighting maintenance within the total resources for all its functions.

Financial considerations mean that the ideal maintenance treatment may not always be possible. The asset owner must use a structured and prioritised approach to manage the risks associated with the selection of maintenance options.

The concept of affordability acknowledges the financial constraints asset owners often have and it is recognised that the legal position will need to evolve to give effect to this.

- Lighting is a valuable and highly visible asset, essential to the social and economic well-being of a community.
- Highway authorities have a non-delegable statutory duty to maintain their lighting, and failure to do so can result in serious injury or death to road users and/or damage to property, and can lead to civil claims.
- Local policy and priorities, finance and affordability are influences that will define the risk appetite/risk tolerance of the asset owner and so determine how risks are managed.

The law

The law relating to highway and other asset-owner liabilities continues to evolve. Therefore this section does not provide a definitive guide but sets out the main statutory provisions and case law which provide the core principles on which highways claims are considered.

Competency

Competency requirements

Competence is the ability to work to an agreed standard on a regular basis. It involves practical and thinking skills, experience and knowledge, and may include a willingness to follow agreed standards, rules and procedures. The combination required depends on what needs to be done, in what circumstances and how well. The right people/resources must be employed for the relevant tasks.

Construction (Design and Management) Regulations (CDM)

Any asset manager, asset owner, asset supervisor commissioning any works must comply with the requirements of the Construction (Design and Management) Regulations 2015 (CDM). No duty-holder must appoint a principal designer, designer, principal contractor or contractor unless they have taken reasonable steps to ensure that the organisation or individual they propose to appoint has the skills, knowledge, capacity and experience and – if they are an organisation – the organisational capability necessary to fulfil the role in a manner that secures the health and safety of any person affected by the project.

A range of competency registration schemes exist, of which the following are but a few and are relevant to luminaire supports.

The ILP has published Guidance Note 4 which provides an overview of CDM.

Client

It is essential that client asset managers have sufficient training and development to support their competence in their decision-making processes about the development of service levels, priorities and programmes or the issuing of works instructions. It should also be remembered that anybody issuing works instruction will be considered to be the client, and should a client alter or amend a design then they could also be considered to have taken over the role of designer. In most cases the asset manager will be considered the client and therefore a duty holder under the requirements of CDM. It will be the decisions made and records kept by the client that will be the first port of call should there be an investigation following an accident, claim or audit of service delivery.

Designers

Designers are in a unique position to reduce the risks that arise during construction work and have a key role to play in CDM. Designs develop from initial concepts through to a detailed specification, often involving different teams and people at various stages. At each stage, designers from all disciplines can make a significant contribution by identifying and eliminating hazards and by reducing risks.

Designers' responsibilities extend beyond the construction phase of a project. They also need to consider the health and safety of those who will maintain, repair, clean and eventually demolish a structure. Failure to address these issues adequately at the design stage may make it difficult to devise a safe system of work. It could also cause additional costs later because either scaffolding or access equipment is required.

Designers under *The Management of Health and Safety at Work Regulations* (MHSWR) are required to carry out suitable and sufficient risk assessments. The CDM regulations, in particular Regulations 11 and 13, set out the duties of the designer with respect to construction work, and this applies even if the work is not notifiable under the scope of the regulations.

Designers need to consider many factors as they prepare their specifications and designs. These must be weighed alongside other considerations, including cost (capital and revenue), fitness for purpose, aesthetics, buildability, maintainability, and environmental impact.

Designers must reduce foreseeable risk to health and safety, based on the information available when the design is prepared or modified. The greater the risk, the greater the weight that must be given to eliminating or reducing it. Designers must not produce designs that cannot be safely constructed and maintained.

Where risks remain, designers must provide the information needed to ensure that planning supervisors, other designers, and contractors are aware of them and can take account of them; the information should be present within the maintenance file.

Who are the designers?

Designers are those who have a trade or a business which involves them in:

- preparing designs for construction work including variations. This includes preparing drawings, designs, details, specification, bills of quantities, and the specification of articles and substances, as well as all the related analysis, calculations, and preparatory work;

or

- arranging for their employees or other people under their control to prepare designs relating to a structure or part of a structure.

This means that designers include:

- design practices contributing to, or having overall responsibility for, any part of the design;
- anyone who specifies, modifies or alters a design, or who specifies a particular method of work or material. This can include clients;
- those procuring materials where the choice has been left open;
- contractors carrying out design work;
- temporary works design;
- heritage organisations who specify how work is to be done in detail.

It is recognised that all designers will have their own processes and procedures for undertaking the review and mitigation of risks as part of their design practice.

Registration with sector schemes

In order for the client to ensure they are engaging the right people, it is recommended that the competency of all staff – whether carrying out site surveys or accessing electrical equipment, and including contractors undertaking installation, maintenance and inspection work – should be assured through a registered organisation such as the Highway Electrical Registration Scheme (HERS) (www.highwayelectrical.org.uk/HERS) or similar sector scheme relevant to the works to be undertaken.

NHSS 8 (The National Highway Sector Scheme for the *Overseeing* and/or *Installation* and/or *Maintenance* of Highway Electrical equipment and Supporting works), for example, requires that employees working in the highway electrical industry shall be assessed as being competent by the organisation that employs them. The organisation must issue the employee with an authorisation certificate that identifies them as an authorised person allowed to carry out work appropriate to the level of competence against which they have been assessed. All evidence supporting the assessment of competence carried out by the organisation shall be contained in a portfolio and retained by the organisation.

In order to ensure rigour in this process, NHSS 8 requires that each authorised person shall be placed on a register by their employer under this scheme and a registration card issued. The ECS HERS card is issued by the JIB/SJIB under their Electrotechnical Certification Scheme (ECS), which also provides affiliation to the Construction Skills Certification Scheme (CSCS).

Competency of NDT organisations/operatives

Non-destructive testing and inspection are vital functions in achieving the goals of efficiency and quality at an acceptable cost. In many cases, these functions are highly critical; painstaking procedures are adopted to provide the necessary degree of quality assurance. The consequences of failure of engineering materials, components and structures are well known and can be disastrous.

It is an increasing requirement of quality assurance systems that a company's engineers, technicians and craftsmen need to demonstrate that they have the required level of knowledge and skill. This is particularly so since NDT and inspection activities are very operator dependent, and those in authority place great reliance on the skill, experience, judgement and integrity of the personnel involved. Indeed, during fabrication NDT and inspection provides the last line of defence before the product enters service, and once a product or structure is in use in-service NDT is often the only line of defence against failure.

All operatives and organisations carrying out non-destructive testing are able to demonstrate a suitable level of competency to conduct their appropriate duties. An example is the Personnel Certification in Non-Destructive Testing (PCN) scheme, which is an international programme for the certification of conformance of non-destructive testing personnel, and satisfies the requirements of a number of European and international standards. The PCN scheme is designed to set and maintain the highest standards for the proficiency of NDT personnel through independent examination and assessment. The management and administration for the PCN scheme within the UK is provided by The British Institute of Non-Destructive

Testing (BINDT) from its Northampton headquarters (http://www.bindt.org/).

BINDT operates an accreditation scheme for NDT and condition monitoring training establishments. The scheme is centred around the published criteria:

- Minimum requirements for the structured training of NDT practitioners; and
- Minimum requirements for the structured training of condition monitoring personnel.

A certificate of competence is a document issued under the rules of the certification system, indicating that the certificated person is competent to perform the inspection tasks defined in the relevant documents, demonstrates the ability to detect flaws and defects, and satisfies specifications that call for certificated competent inspectors.

A company in corporate membership which is also a member of BINDT's Service Inspection Group is required to provide evidence of a quality system, preferably to the requirements of BS EN ISO 9001, and also to undertake to operate in accordance with a code of ethics approved by the council of BINDT.

Where inspection and testing operations are undertaken within the highway boundaries, but do not require access to electrical equipment, then staff should hold Construction Skills Certification Scheme (CSCS) cards (see www.cscs.uk.com/), and understand the full requirements with respect to temporary traffic management.

Those accepting commissions are reminded that no duty holders – designers, principal designers, contractor, principal contractors and workers – must accept an appointment to undertake a role unless they meet the requirements detailed above.

Rail sector

Those working on the UK rail network are required to hold a Personal Track Safety (PTS) card, which is the basic requirement to allow a person to work on or near the railway line. In 2014, new Sentinel smartcards completely replaced the previous cards which are no longer valid. The Sentinel card is the demonstration that an individual has achieved the required level of competency and is working for an approved employer. Without a Sentinel card, Network Rail does not know if an individual has attended approved training or is working for an unauthorised employer.

Managing risk

The principal part of managing health and safety at work is the identification of hazards and the management of risk in order to remove or reduce to a minimum the possibility of injury; this is covered under the MHSWR.

The asset manager, in setting programmes, prioritising works or delivering services to achieve agreed service standards, will need to manage risk. The asset manager's decisions will define the service risk so decisions must be recorded and reported for future scrutiny so the users of the service are aware of the decision-making process. In order to deliver the service the asset manager must identify the risks and assess the consequences of those risks should they be realised. The asset manager will then need to put in place sufficient control measures to deliver the service within the risk parameters agreed with senior managers or elected members representing the service users.

It is not the intent of this section to detail all requirements with respect to the management of risk, as anyone undertaking design, installation, maintenance and de-commissioning needs to meet the required competencies as described earlier in this document. However, there are key elements that are worthy of mention, as follows:

- The *Health and Safety at Work etc. Act* (HASAWA) Section 2 sets out the duties of employers to employees, requiring employers to ensure, as far as is reasonably practicable, the health, safety and welfare of their employees at work;
- Similarly, Section 3 requires employers to conduct their undertaking in such a way that persons other than employees, including the general public, are not exposed to risks to their health and safety;
- *The Management of Health and Safety at Work Regulations* (MHSWR) takes this further and places an absolute requirement to carry out suitable and sufficient risk assessments.

The main stages in undertaking any risk assessment are:

- Identifying the hazards, anything with the potential to cause harm;
- Identifying who may be harmed;
- Assessing the risks and extent of the risk;
- Identifying any existing controls that are applicable;
- Identifying the required standard;
- Identifying what preventative and protective measures/actions need to be carried out, by whom and when in order to reach the standard required. this may require the production of method statements;
- Recording the process;
- Reviewing and revising as necessary.

Appendix 10
History

Minor structures are relatively low-cost, low-technology items of infrastructure, but are essential structures to support apparatus such as traffic signs, CCTV or smart-city equipment, or luminaires for street, area and specialist lighting. All too frequently they receive insufficient priority for inspection and maintenance as they are generally not regarded as high-risk items. Accepting a local authority column population in the UK currently of the order of 7.5 million, plus many more thousands in private/public ownership, the actual number of failures form a very small percentage of this total. In the past when failures have occurred, they have usually been in a period of extremely high winds where either members of the public are absent from public areas or, in extreme cases, there is so much other debris that they are very aware of the potential problems and avoid all possible falling or flying objects. As a result, personal injuries from falling columns and brackets are relatively few; excepting deliberate acts of vandalism or misbehaviour.

In recent years, commercial pressures and greater sophistication of design have resulted in minor structures which are more closely matched to their design loading requirements, without the additional strength that may have been present in earlier designs. Lighting columns and the like are also seen as ideal for the erection of post-installation attachments such as hanging baskets, flags, banners and electronic apparatus that were not taken account of in their specification. Such attachments to minor structures with little additional strength will, at best, shorten the life of the asset but could easily render the asset overloaded. The danger of this trend is that undetected corrosion may cause premature failure. The same financial pressure on authorities has also resulted in limiting the amount of maintenance that can be carried out, resulting in some minor structures being left in a very poor condition for longer than desirable.

This situation would still have remained a matter for little concern had it not been for a further trend in today's society where members of the public readily pursue litigation, and the values of claims when awarded are increasing to penal levels. There is also increasing awareness of responsibilities under health and safety legislation.

Some high-profile accidents involving failure of lighting columns have made it essential for all authorities to consider the condition of their minor structures, and to identify and record the need for maintenance or replacement on a sound and logical basis. The financial constraints make it essential for such examination, repair and replacement to be targeted as closely as possible to those minor structures most needing attention, and records kept of the decision-making process.

Lighting columns of various types have been in use in the UK since the mid-18th century. Various developments have taken place, using different materials to meet the differing requirements of providing lighting and an aesthetically pleasing structure in the street scene.

Early timber columns were replaced with cast iron columns of ornate design for prestigious sites, with cast tubular columns being replaced by structural steel tubular columns in later years. In the war years, the cost and scarcity of steel boosted the concrete column industry and allowed a variety of designs of concrete columns due to the flexibility of casting.

More recently, fabricated folded sheet steel columns have become popular, with the sheet thicknesses much less than those used previously for tubular steel columns. Aluminium columns also feature within the market and are available in the full range of mounting heights. Limited developments have also taken place with other materials, such as stainless steels, fibreglass and reinforced plastic, all of which, in theory, are not subject to corrosion but may have their own particular weaknesses requiring inspection.

Back in 2000, the Institution of Lighting Professionals in its leaflet *Protecting a Vital Asset* reported the number of lighting columns owned by local authorities in the UK as 6.2 million. In addition, there are substantial numbers of lighting columns in private ownership used to illuminate car parks, the rail network, delivery yards and other industrial and commercial property. With the increased use in such locations it can be assumed that there will have been significant growth in the overall lighting column population since 2000. The design of these lighting columns will generally have been based on a structural loading for wind at a speed which occurs statistically once in 25 years, but the expectations of the life of the column are generally much greater than this period.

Since about 1990, there has been a general downward pressure on all public costs, resulting in a continual decline in the amount of money available to carry out inspections and maintenance, as well as a downward pressure on the original product cost on the manufacturers.

The requirement to address their failing public lighting infrastructure has, in part, been considered by a number of local authority public lighting private finance initiative (PFI) schemes, which has seen those authorities replacing some 80% of their lighting stock over a core investment period normally of five years. While this addresses an immediate problem, it does mean that 80% of the authorities' lighting stock will become life expired over a five-year period in years to come.

There has also been a loss of experienced staff in the industry, resulting in a lack of knowledge to identify and deal with problems in the lighting column stock. It was therefore inevitable that column failures due to corrosion were bound to increase. The potential cost of repairing or replacing the existing lighting column infrastructure is rarely estimated, and even where an attempt is made, it probably grossly underestimates the true cost. In its *Protecting a Vital Asset* leaflet, the ILP reported that out of 333,000 steel lighting columns structurally tested in the previous year 9,000 were deemed to be in such a dangerous condition that they required immediate removal from service; a further 21,600 were in a state that required their replacement within two years. The non-destructive test houses in 2017 advised that on average 3.5% of the columns inspected and tested required immediate action/removal.

The need to replace columns is well known, and there have been many photographs of different types of columns with failures, illustrating the need for immediate replacement. Fortunately, most of these failures have not resulted in personal injury accidents, and they have therefore been covered within existing budgets. However there have been a number of high-profile accidents where members of the public have been involved. In one significant case a settlement of £3 million was reportedly agreed. The fear of ever-increasing litigation with the potential of very high costs will justify the relatively small costs of routine inspection and planned replacement.

With pressure to reduce initial product cost and improve visual appearance, inevitably designs have become slenderer with less allowance for possible corrosion.

However, the simple introduction of additional thickness to allow for corrosion does not of itself prevent early failure, as corrosion can be extremely localised and it still needs to be identified even if failure is delayed. Many of the very old ornate cast iron columns will become a matter for concern from corrosion for aesthetic reasons long before they become structurally unsound, and, with the rise in heritage lighting and conservation areas, the need for the protection of such ornate columns becomes apparent to the public long before the risks of failure. The main concern is columns designed and manufactured since about 1970 which have minimum material thickness and are the outputs of low-cost production. The sheer volume of columns creates the greatest problem, particularly as many of the corrosion effects will occur unnoticed and potentially undetected.

The current trend is for the majority of lighting columns to be protected by a hot-dip galvanised coating, which can generally be expected to ensure adequate internal protection to the base column material for the life of the external surface of the column.

Other solutions using non-corrosive materials such as stainless steel or aluminium are also available; they have a higher initial cost but may provide a whole-life saving. Aluminium columns are inherently passively safe and mainly low- or non-energy absorbing and therefore need to be considered in this context, based upon the guidance in the ILP Technical Report 30 *Passive safety: Guidance on the Implementation of Passively Safe Lighting Columns and Signposts*.

The requirement on local authorities to ensure best value may see an increasing use of initially higher-cost products such as stainless steel or aluminium when considering a whole-life cost of ownership approach.

Appendix 11
Development of lighting columns with the UK

Introduction

This appendix describes the development of lighting columns/luminaire supports over time, considering the materials used and methods of construction as well as protective finishes. The time scales indicated in this appendix are approximate.

Design requirements:

The specification and design of lighting columns is covered by:

BS EN 40 Lighting columns, and consists of the following parts:

- Part 1: Definitions and terms;
- Part 2: General requirements and dimensions;
- Part 3-1: Design and verification — Specification for characteristic loads;
- Part 3-2: Design and verification — Verification by testing;
- Part 3-3: Design and verification — Verification by calculation;
- Part 4: Requirements for reinforced and pre-stressed concrete lighting columns;
- Part 5: Requirements for steel lighting columns;
- Part 6: Requirements for aluminium lighting columns;
- Part 7: Requirements for fibre reinforced polymer composite lighting columns.

Within the UK, the application of BS EN 40 can be found in PD 6547:2004 +A1:2009 *Guidance on the use of BS EN 40-3-1 and BS EN 40-3-3*. This is discussed in more detail within Appendix 2 of this document.

Materials

Cast iron columns were generally replaced with structural steel tubular columns until the war years, when the cost and shortage of steel boosted the concrete column industry; concrete columns of various designs and mounting heights were produced. Concrete columns remained in production for many years after the war but their use generally ended in the early 1970s though production continued into the 1990s. In the 1980s, in order to extend the life of such columns, the concrete bracket arms, which had a tendency to crack and spall, were removed and suitable folded steel galvanised sleeve brackets were fitted. The concrete columns within the UK are now well past their original operational expected life, but if in good condition are still serviceable.

Until the 1970s the steel used for columns was fairly thick at 6mm plus. It was only during the energy crisis in the 1970s and with the introduction of computers for design that the tube thickness came down quite considerably to what was calculated as needed for the structural load. Wall thickness of 3.5mm was not uncommon. When the concern of column failures due to corrosion was raised, there was a change in the industry with many specifiers considering that thicker was better, so changing their specifications to the thickest tube the production lines could take, circa 6mm for >8m columns and 5mm for <6m columns. In addition whole column protective treatments such as galvanising became the norm.

There has also been the development of fabricated folded sheet galvanised steel columns with sheet steel thicknesses much less that those for tubular steel columns.

Aluminium columns featured for a time and were mainly used for low mounting heights (circa 5m) but in the past fifteen years there has been an uptake of these with new production techniques for a greater mounting height, generally being tubular and tapered in their design. Aluminium columns are inherently passively safe structures.

There have also been stainless steel and wooden columns although the take up due to cost has not been great.

Protection

Column

As a rough guide the following protective systems have been applied to steel columns over the years.

>40 years ago: Essentially the only protection to the materials was a paint finish.

40 years ago: Steel columns were shot blasted and metal sprayed, a treatment for the exterior of the column only.

25 years ago: Hot-dip galvanising started, so protection could now be provided both inside and outside a column.

Root

The roots of metal columns tended to be treated with a bitumen finish until about 10 years ago, since when glass-flake paint and other more-resilient protective coatings have been applied. The application should be for both the inside and outside of the root to 150mm above ground level.

The easiest way to consider the protective finish, and hence perhaps get some idea of age, is to remove the column door and look at the inside of the column as that is likely to show the original protective system.

The current trend is for the majority of steel lighting columns to be protected by a hot-dip galvanised coating, which can generally be expected to ensure adequate internal protection for the life of the external surface of the column. It would appear from the number of installations that other solutions using non-corrosive materials such as stainless steel or aluminium are still felt to be initially more expensive than adequately maintained hot-dip galvanised lighting columns, but based upon a cost-of-ownership whole-life assessment may prove more economical.

A number of asset owners also look to provide a painted finish to improve the aesthetic appearance of the columns as well as extend its operational life. Although aluminium is relatively resistant to corrosion and does not require paint protection, there may be times when for aesthetic reasons the columns are painted.

Construction

Concrete columns

The manufacture and use of concrete columns has ceased in the UK. Concrete columns are of two basic types, reinforced and pre-stressed, and are discussed in detail in Section 8 of this document.

Tubular steel

Shoulders

Welded

For smaller columns (circa 5m) the shoulder may be a fabricated section with welds at the top and bottom of the tapered section, the welds being dressed to improve the appearance.

In some cases, these will be supported by an ‘internal washer’ to provide a stronger connection. This requires openings to be left around the washer to permit the galvanised costing to penetrate all areas and drain after emersion. The washers normally have at least three points of connection. It should be noted that this form of connection was also used before

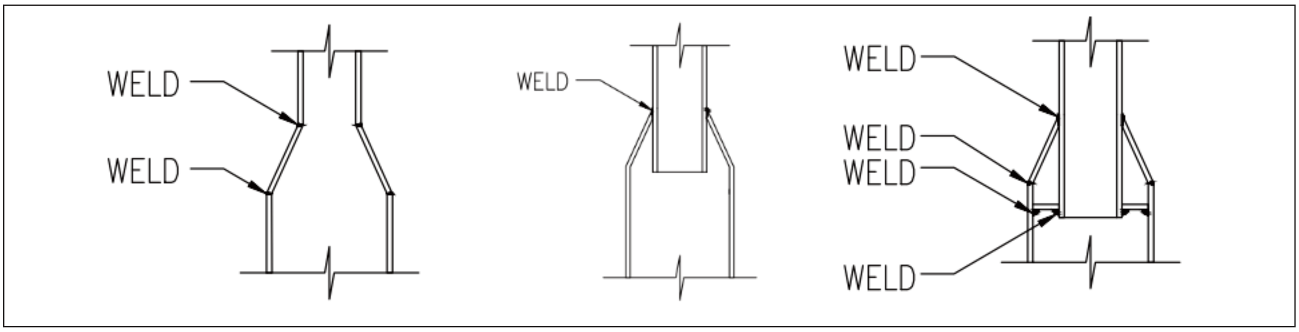


Figure A11.1: Tubular steel shoulder construction details

galvanising was considered as a protective finish and the inside of the column received little if any treatment. Figure A11.1 demonstrates the three main methods of constructing a welded shoulder interface between a column base and shaft.

Swaged and welded

For a swaged and welded base-to-column-shaft shoulder, demonstrated in Figure A11.2, the base tube is normally swaged down to a diameter slightly greater than the shaft tube, a top weld is then applied and may be supported by an internal washer, and these may not be welded but should provide a tight fit.

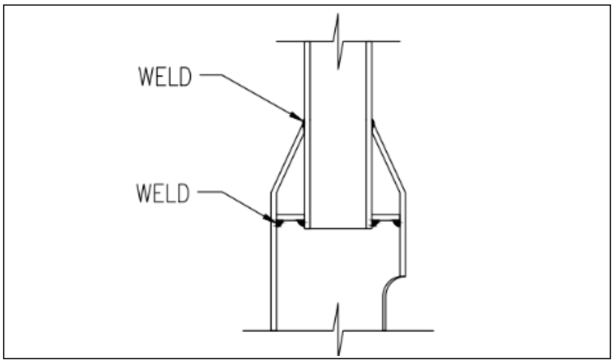


Figure A11.2: Tubular steel column welded joint detail

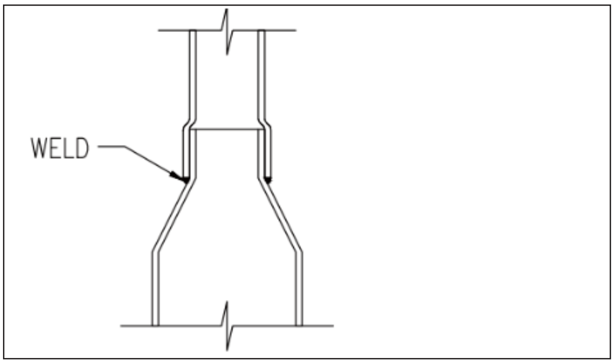


Figure A11.3: Tubular steel column swage joint detail

Swaged

A swaged base-to-column-shaft shoulder is shown in Figure A11.3. This is where the base tube is swaged down to form a short parallel section of a smaller diameter than the shaft. The shaft is then heated such that it expands and then slid over the base section to form a tight fit as it cools. A decorative weld may also be applied.

Flanges

Flanges are generally made out of thick flat plate with the column connected to them and the holes drilled or profiled for the foundation fixing bolts, shown in Figure A11.4. Washers are essential to spread the load from the nut into the flange and should be of sufficient size and thickness to resist distortion.

Another type of flange that is occasionally used, particularly for smaller columns, is a pressed or preformed flange which relies on its three-dimensional shape for the transfer of load to the foundation bolts.

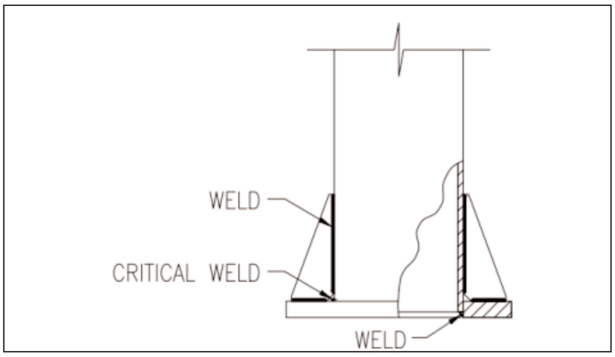


Figure A11.4: Typical flange plate detail

This form of flange plate is likely to be thinner than a flat plate and problems of distortion can therefore occur around the anchorage fixings.

Flange surface fixing

Some designs allow for the column shaft to be welded on to the top surface of the flange plate, requiring a higher quality of weld and flange plate material.

Gussets

Gussets are not generally used on lighting columns; they may be used on lighting (masts) columns of 15m and above.

Anchorage fixings

Depending upon the foundation design the flange plate anchorage may be either studs, where washers and nuts are used to secure the column to the foundation, or bolts that engage into cast-in threaded sockets within the foundations. In all cases, suitably sized washers should be used to distribute the load from the nut/bolt head to the flange plate and foundation. In the case of studded anchorages, it is likely that a second lock nut will be used.

Door openings

Door openings will generally be one of two basic types, either un-reinforced, being merely a cut out in the section of the column, or reinforced where strength is replaced by the addition of steel members in a number of different configurations.

The main problem that can occur with un-reinforced door openings will depend upon the corner radius. The British Standard recommends a minimum radius of 20mm for corners. Door openings with smaller radii or a complete right angle should be examined carefully, as small corner radii or right angle openings are known to be places at which fatigue cracking occurs.

Historically there was an issue with Stewart & Lloyd columns which have a square door corner profile. The majority of these were quickly identified across the UK and removed.

Roots

Roots will tend to have a cable entry hole cut within the main tube section.

Additional Shaft Steps

Some columns have multiple steps and will normally be of the swaged designs discussed earlier but without the use of an internal washer. Those with an internal step that are not galvanised will carry a higher risk of corrosion. Examples of over-sleeve arrangements are demonstrated in Figure A11.5 and A11.6.

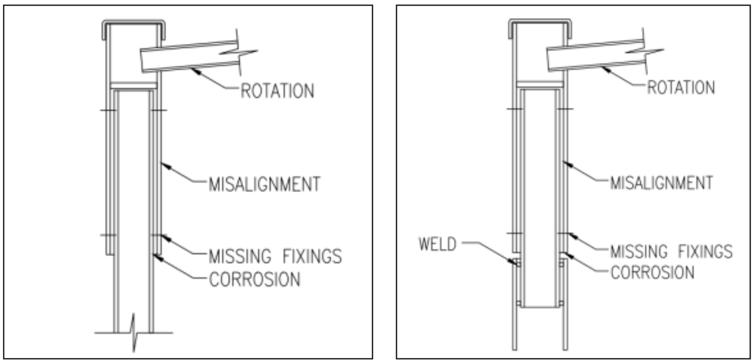
Bracket joint

Over sleeve

Many brackets over-sleeve the column shaft and incorporate the use of pinch screws or other fixing means to maintain location. Structural failure at this point is rare, except in the Millennium Map's cases

Picture A11.1: Stud anchorage

Picture A11.2: Bolted anchorage



Figures A11.5 & A11.6: Over-sleeve and parallel bracket details

of very high corrosion, poor maintenance or overloading due to, for example, a luminaire being replaced with a larger and/or heavier one. Water ingress is highly unlikely with this detail.

Parallel (female bracket/male shaft)

This design allows the continuation of the same cross section of the vertical part of the shaft on to the vertical bracket tube by having an internal fixing using a spigot swaged or welded in to the top of the column shaft. With this detail, the fixings will be on the bracket, either internally by means of washers and anti-rotation bars, or externally by means of pinching screws. The main problems with this detail will be corrosion and stripping of any pinching screws, allowing the bracket to rotate or tilt. Water ingress must be prevented by specific mechanical details or sealing.

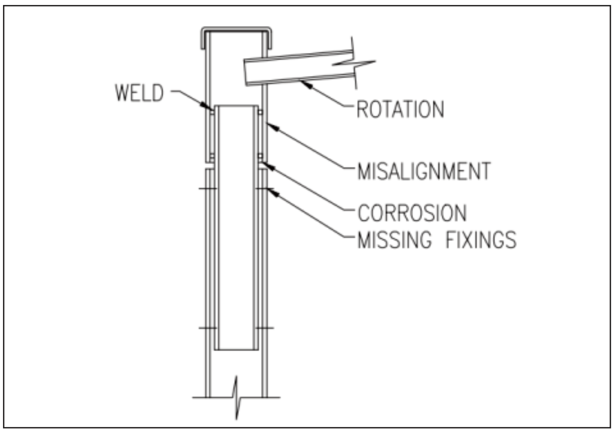


Figure A11.7: Bracket connection

Parallel (male bracket/female shaft)

This is the reverse of above design with an internal fixing using a spigot swaged or welded in to the bottom of the vertical part of the bracket, shown in Figure A11.7. Any pinching screws present would be in the shaft of the column connecting on to a smaller diameter of the bracket. This detail is more likely to give rise to water ingress and internal corrosion or joint corrosion, and should therefore be inspected.

Bolted bracket fixing

One bracket fixing used extensively in the past consists of a small flange to which the bracket is welded, shown in Picture A11.3. The flange is then bolted on to a similar threaded plate on the top of the column. Missing or corroded bolts can allow the ingress of water and the increased potential for internal corrosion. There is evidence that bolts can vibrate loose with time and should therefore be subject to routine inspection and tightening.

Picture A11.3: Bolted bracket connection

Historically there was an issue with one design of British Steel columns which had these brackets. They were of an age when galvanising was not used and had a type 6 swaged joint, and corroded from the inside at that point. The majority of these were quickly identified across the UK and removed.

Folded/sheet steel columns

These columns are normally formed in steel plate with eight sides, though six, ten or twelve sides and irregular octagons are also frequently used. See Picture

Pictures A11.4 and A11.5: Standard folded steel column and square profile steel column

A11.4. Many of the problems are similar to those of tubular steel columns above.

In some authorities, special square (four-sided) profile columns have been designed and used as a local authority style or as an enhancement feature combined with a wooden-clad main shaft, shown in Picture A11.5.

Sheet steel columns are more prone to minor impact deformations than tubular steel columns resulting in dents or distortions in the section of a column, which can substantially reduce its strength.

A buckle or crease across one face of an octagonal column, including the two adjacent corners, which results in a deformation of only three or four times the material thickness (typically 6–10mm), can result in the section not being able to sustain the wind loading for which it was originally designed.

Shaft joints

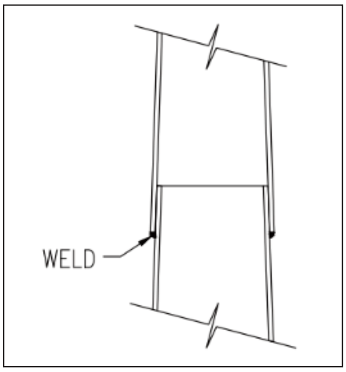


Figure A11.8: Sleeved joint detail

For longer columns, transverse joints may be necessary so as to limit the length of sheet used in fabrication. The usual method of connection is an overlapping

Pictures A11.6 and A11.7: Base- and shoulder-hinged fold-down columns

Pictures A11.8 and A11.9: Mid-hinged fold-down column and corrosion at the pivot point

sleeve with an external fillet weld at the lower edge, shown in Figure A11.8.

The potential for internal corrosion is the same as with the swaged shoulder joint in tubular steel columns, and the importance of galvanising is therefore clear.

Dry sleeve joints

For segmental columns or for columns of considerable height requiring site joints, the usual form is an overlapping taper sleeve without welding. As above, the presence of galvanising is important for these joints to prevent internal corrosion at the step.

Aluminium columns

Tubular fabricated aluminium columns are generally a one-piece single extrusion. Older aluminium columns, generally providing a 5m mounting height, may consist of a cast base section with an aluminium main shaft.

In addition, there may be specialist fabricated sheet aluminium columns where the assembled sections use cut extruded profile sections which clip together. Such sections will sleeve in part with the other members but will have to be fixed by means of adhesives or with welding at intervals and ends; see Figure A11.9.

Fold down

Fold-down or hinged columns can be made from steel, stainless steel or aluminium. The hinge can be located at the base, shoulder or along the shaft (mid-hinged). The majority require the use of a fold-down unit to be brought to site and used to safely lower the column. The columns can range in height from 4m upwards as demonstrated in Pictures A11.6 to A11.9.

Protective coating

Concrete, aluminium and stainless-steel columns do not tend to have any corrosion protection treatment above ground, although protection of any root section may well be essential. Some aluminium lighting columns have a felt 'bandage' around the root section that acts as an interface between the concrete surround and the root. This acts as a damper that absorbs some of the forces when the columns moves due to wind or other forces.

A common treatment has been a simple coating of bitumen, but other paint systems and plastic coatings have been used.

For carbon-steel columns, a corrosion protection treatment is essential; a paint system is often used to give the column a more aesthetically pleasing appearance even when the column has been protected from corrosion by other means.

The life of any coating will be dependent upon the amount of acidity in the atmosphere at the particular location. A number of documents give advice on the life of coatings and in particular maps produced by the Galvanizers Association are useful in determining the life expectancy of galvanized coating.

Steel columns

Some authorities have been installing galvanised columns for twenty years or more, while others have been installing them for less than ten years. A major advantage is that both the internal and external surfaces are well protected. Furthermore, the coating corrodes preferentially (sacrificially) to provide cathodic protection to small areas of steel that are exposed by damage.

Prior to the application of galvanising, the practice used to be metal – zinc or aluminium sprayed. The columns were shot blasted and cleaned, and the coating applied by special pistols that melt wire fed into the pistol and blow the molten metal onto the steel. The molten droplets flatten on impact, and overlapping layers are built up to the required thickness. This was a process started some 40 years ago.

Aluminium spray can be applied to almost any thickness, but is typically 100µm or 150µm thick. Since sprayed metal layers are slightly porous, they are sealed with resin to form an impermeable coating. An additional protective treatment is applied to the root.

Galvanising is one of the few system which ensures that corrosion protection internally is at least as good as the external protection. Identification of the presence of a galvanised coating is therefore a first step when inspecting columns to determine a maintenance regime.

The worst corrosion has been found on old columns that have only a paint system on their external surfaces. Older paint systems have generally provided protection for only about ten years above ground.

Polymer coating

High-build fusion-bonded polymer coatings offer both aesthetic and corrosion protection. This process usually involves heating the column to a temperature of approximately 280°C, and either

- Immersing the structure in a bed of fluidised powder granules;

or

- Electrostatically spraying the column.

The granules then melt onto the surface of the structure, creating the fusion bonded polymer coating.

This process can also be applied locally to the root section of the column both internally and externally and is an alternative to bitumen or painted root-protection systems.

However, two main problems are common in practice. Firstly, the coating can become damaged by accidental contact or vandalism, or even incorrect erection procedures. Once damaged, corrosion will commence locally and can creep under the coating. Secondly, the coating can become debonded from the metal surface potentially allowing corrosion to take place unseen.

Such coatings are not generally applied to the internal surface.

External non-galvanised

Some old columns may have no internal protection; most that have a paint system or metal spray on their external surfaces have thin bituminous paint on their internal surfaces. This appears to have offered little protection when water has accumulated at hot-swaged joints and when base compartments have flooded. Some authorities reported that thin bituminous paint has tended to embrittle with age and debond from the steel.

Internal surfaces: galvanising

The internal surfaces of many galvanised columns with no additional protective treatment may suffer corrosion in the long term if aggressive groundwater and de-icing salts in solution frequently flood the root. Some authorities are now sufficiently concerned about the performance of galvanised columns below ground that they are specifying additional protective treatments on the internal and external surfaces of the roots of galvanised columns in all ground conditions.

Root coatings

As corrosion is more likely to occur in the root and just above ground level, many columns have alternative or additional protective treatments on the external and internal surfaces in these areas.

The most common treatments are thick bituminous coatings, although thin bituminous paints have also been used. Some authorities now specify pitch epoxy or glass-flake paint treatments.

Root: concrete foundations

Some columns have a concrete foundation (surround) to provide the required ground resistance moment to overturning. Concrete is alkaline and it tends to passivate steel by forming a protective oxide film. Therefore, concrete surrounds usually limit the amount and rate of corrosion on the external surfaces of columns below ground, provided they are bonded to the column and have been profiled to prevent water ponding adjacent to the column shaft.

Many surrounds stop about 150mm to 200mm below ground.

Pictures A11.10 and A11.11: Luminaires mounted to wooden poles

Most concrete foundations are cast using a relatively dry mixture that is compacted so that the column can be left unsupported soon after the concrete is placed. The placement and compaction of the concrete may damaged protective treatments, and contribute to the corrosion of columns with only a paint system.

Wooden/timber poles

In the past it was and still remains common practice within rural communities to use existing telegraph or power company distribution poles to support the road lighting luminaires. There is normally agreement between the local authority and the operator for their support to be used. In these cases, the structural integrity of the pole has been the responsibility of the communications or power company.

The method of attachment of the bracket to the wooden pole should be part of the local authority's routine inspection and testing.

However, in some areas the communications or power company has removed its equipment from these poles and their sole purpose has become that of a luminaire support; therefore the responsibility for the structure is transferred to the local authority (Pictures A11.10 and A11.11).

Wooden or timber poles

For many years wooden poles have been used to support exterior lighting luminaires, usually sharing the pole with the local electricity company, also known as distribution network operator, or in some cases with telephone communication cable supports. When the pole owner removes their equipment the exterior lighting asset manager has been offered the redundant pole to continue to use as a lighting support. Examples of a typical wooden pole installation is demonstrated in Picture A11.11 and a typical shared

Pictures A11.12 and A11.13: Modern wooden clad columns

wooden pole installation is shown in Picture A11.12.

More recently, a number of companies have started the manufacturer of timber poles, and in most cases the timber element is made from laminated wood components and forms a cladding over a central steel support. They are marketed under a range of names including wood candelabras.

In the past such structures have had problems due to cracking or debonding of the laminates; this aspect is addressed by the use of a reduction of the thickness of the wood associated with a tubular reconstitution and makes it possible to minimize the surface cracking inherent in the wood material.

The manufacture must be made of laminated wood class GL24H or GL28H according to standard NF EN 14080 and CUAP 01.06/07 with an additional requirement limiting the size of the laminate strips/nodes to 20mm. The adhesives used in the reconstitution of columns are type 1 according to standard NF EN 301.

The columns are normally fixed on the foundations or implanted in the ground by means of a metal base. In both cases, a minimum crawl space of 10cm must be maintained between the finished floor and the wood.

Manufacturers offer various forms of candelabra for heights ranging from 3m up to 16m. Masts of great heights (maximum current height of 27m) are also made of glue-laminated timber for specific projects.

The glue-laminated candelabras are claimed as offering the following advantages:

- Use of renewable natural resources;
- High decorative function allowing a good integration in the urban space;
- Good durability of the timber ensured by the choice of species and by the constructive provisions retained or conferred by treatment;
- Good corrosion resistance of the steel components conferred by treatment;
- Vast possibilities of forms by machining wood;
- Energy absorption capacity in case of violent shocks with vehicles for masts with inspection doors in the metal base;
- Reduced weight resulting from the low density of the wood (about 500kg/m^3).

The final coating of the wood drum is made using stain, paint or varnish. The coating is the most common component that requires periodic maintenance every ten years or so.

Protection and finishing

The final coating of the timber is made with stain and varnish on the basis of three layers:

- An impregnation layer;
- Two layers of protection and colour

The coating is a surface treatment that reduces moisture absorption, protects against UV and allows it to withstand dimensional variations of wood without flaking.

Periodic maintenance every seven to ten years (depending the location of the installation) allows renewing the surface condition of the wood for the same period.

The metal base is protected against corrosion by hot-dip galvanizing, by immersion of the finished product, according to standard NF EN ISO 1461 and then a polyester thermal coating is applied.

Appendix 12
Indications from past inspection and testing

Appendix 11 discusses the development of lighting column design within the UK, and within this section each column type is reviewed to understand the problems/issues that can affect a column's structural integrity and thus its structural stability.

Cast iron columns

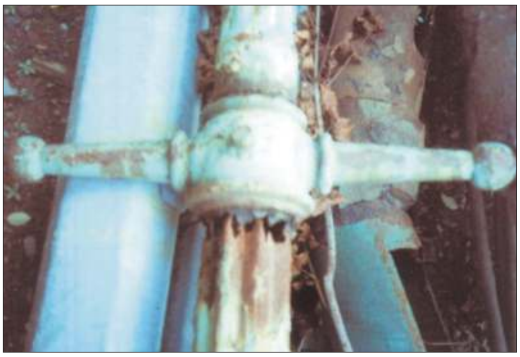
General

Cast iron columns may be of very old design, using poor-quality cast iron and, as a result, have a very heavy section and therefore low stress levels. More recently, cast iron or cast steel may have been used, particularly for ornamental columns, and again the section will have been selected more for aesthetics than specific structural design, and will tend to have low stress levels. However, the principle for inspection applies equally to both types.

Door openings

The door opening creates the potential for higher stresses due to the reduced section and also a stress concentration, subject to the detailing. The door itself is a non-structural item and therefore need only be considered for aesthetics.

The main problem that can occur with door openings will emanate from the corners where a stress concentration can result. Additionally, in the case of castings, the presence of small defects such as voids or blowholes cause porosity. Unlike structural steel, poor detailing can result in a satisfactory design due to low stress levels being used. A radiused corner will provide a more satisfactory detail than a sharp right-angled corner, but in the case of castings with low stress levels even a right-angled corner can prove satisfactory in service. Another source of deterioration can be at the interface of the shaft with separate sections such as the base or other features such as a ladder bar, as demonstrated in Picture A12.1.



Picture A12.1: Severe corrosion at a ladder arm detail on an ornate column

Ornamentation

Cast iron columns are generally used as heritage columns, with ornamentation to match older designs in selected areas as demonstrated in Pictures A12.2 and A12.3. Current production frequently utilises additional ornamentation on simplified designs for economies in production. These details may be an embellishment kit sleeved over an existing shaft or welded to it, and can result in moisture traps or stress raisers, with potential cracking in the longer term. Careful inspection of these details is therefore essential and is discussed within this section. If the item is removable, it may well be good practice to remove or reposition it periodically. Where practical, moisture penetration should be prevented using appropriate caulking/ sealing techniques.

Embellishment kits tend to be used to turn a standard tubular column into a more decorative structure and may provide a heritage or modern style to the final column, examples of which are shown in

Picture A12.2: Cast iron decorative column

Picture A12.3: Embelished column base section

Figure A12.1. They can either be in component parts – consisting of a base ring at ground level, a ring that sits at shoulder level, then a mid-ring along the column shaft and perhaps a ladder bar – or may be full base embellishments which cover the standard column from ground level to the top of the base/shaft shoulder.

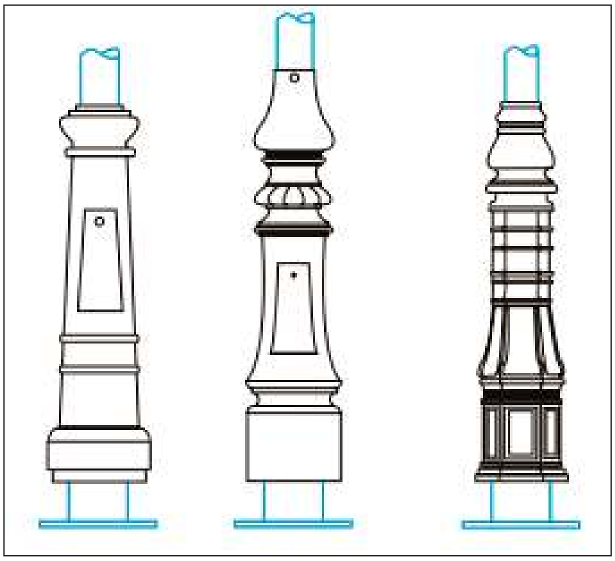


Figure A12.1: Full base embellishment kit

They can be fitted and clamped to columns; however, the fitting and removal of these needs to be undertaken with care to ensure that the grub screws and fixings do not cause damage to the column's galvanised/painted protective coatings when the parts are slid or clamped into position.

When considering attachments/ embellishment kits it should be noted that certain metals are subject to corrosion when placed in contact with other metals. This is commonly referred to as electrolytic or dissimilar-metals corrosion. Contact of different bare metals creates an electrolytic action when moisture is present. If this moisture is salt water, the electrolytic action is accelerated. The result of dissimilar-metal contact is oxidation (decomposition) of one or both


Pictures A12.4 and A12.5: No sealing between column and embellishment kit/water ponding at poor joint on embellishment

metals. The metals should be separated by, for example, metal primer, aluminium tape, washers, grease, or sealant, depending on the metals involved and the environment in which they will be operated.

Embellishment kits may be cast iron or cast aluminium and in general fitted to galvanised steel columns although they can be fitted to aluminium columns. The kits and columns, especially where dissimilar metals exist, should be coated with an approved duplex coating before assembly.

When inspecting an embellishment kit it should be noted that it is normal that a gap exists between the embellishment kit and the column (demonstrated in Pictures A12.4 and A 12.5) and this should be appropriately filled. Where possible, this should be undertaken using tape instead of caulk. Caulk, when used in a joint between dissimilar metals, can squeeze out – as the two metals contract and expand at different rates – allowing the two metals to contact. Closed-cell neoprene tape is generally recommended, as this tends to do a better job of isolating and, where required, the joint can then be cleanly sealed using an appropriate caulk suitable for the exterior environment. The finish to the caulk should be beaded such that water does not pond around the joint.

Where shrinkage occurs during the curing process additional caulking should be applied to achieve the required profile. Where it is evident that the gap between the embellishment kit components and the column is not sealed then further investigation is advised to determine the condition of the column under the kit. This is not an easy process.

The problem with embellishment kits is that once they are installed correctly they can be very difficult if not impossible to remove to inspect and test the column structure underneath.

In the extreme case shown in Pictures A12.6 and A 12.7 the structural column under the full base embellishment kit has corroded almost totally away. It was only the fact that the kit extended below the surrounding paving that kept the column upright. Looking at the installed column, to all intents and purposes it appeared satisfactory.

The only test that will indicate any issues relating to embellished columns where the embellishment kit can't be removed may be a static load test, although technologies may improve and new test methods developed that may enable further testing of columns fitted with embellishment kits. However, it should be noted that the surrounding ground conditions can be such that the base embellishment kit is

Pictures A12.6 and A12.7: Failed and installed column with embellishment kit

contained within sufficiently compacted or surrounding pavement, so that whilst the internal column may be corroded there is sufficient resistance to support the column via the embellishment kit and it will pass the tests. The defects only becoming apparent when the surrounding ground is disturbed.

It should be noted that this relates more to base embellishment kits that are designed to extend below ground level and not so much to those that are designed such that the bottom of the embellishment sits on the surface of the surrounding ground.

Ornamental details

Due to the scope and versatility of casting techniques, the range of designs available is extremely wide, and considerable depth of relief in the casting may be included at areas of high stress on the column, such as the brackets or change of section near the shoulder or ground level. As with ornamental additions above, these may not be a problem if the stress levels are low, but could give rise to stress concentrations in certain cases. Visual inspection for cracking at such details is therefore essential. Corrosion will be quite apparent if the ornamentation has resulted in corrosion traps, allowing deterioration of the protective coating as shown in Picture A12.8.

Picture A12.8: Failure of a casting fitment on an old ornate cast iron column which could be the start of severe corrosion damage to the structural tube, which would go undetected

Flanges

Cast columns are generally not flanged, as the planted method of installation is far more common. However, where flanges do occur, they are likely to be of considerable size due to an ornamental base and the flange and flange-bolt loadings are likely to be small. The main risks in this area, subject to the detailing of the column, will be corrosion and, possibly, cracking around the bolt area.

Roots

Very heavy corrosion can take place on the column root particularly in areas of aggressive ground conditions. Old columns are less likely to have good root protection so corrosion can take place unseen, but the very heavy sections generally used for this type of column make such corrosion less likely to be structurally significant.

Significant defects

Picture A12.9: Typical cast iron root

The defects that are considered to be the most significant in cast iron columns can be summarised as follows:

- *Corrosion of the root of planted columns*: This can occur in aggressive ground conditions, but is most prevalent in poorly-drained ground, ie clay soils, when the root is prone to frequent flooding. The corrosion is normally worse on the internal surfaces and can be at any height (dependent on the water table)
- *Corrosion from just below ground level to the bottom of the door*: This can occur on all types of planted column. It will be more severe when the protective treatment is damaged, and when dog urine and de-icing salts are present. The corrosion is normally worse on the external surfaces. Similar corrosion occurs on flange-plated columns.
- *Corrosion at ornamental detail*: Trapped moisture can cause local corrosion at ornamental details giving potential problems particularly at bracket joints

Concrete

General

Although the manufacture and use of concrete columns has ceased in this country, there are still many in service with a considerable life expectancy. Concrete columns are of two basic types, reinforced and pre-stressed, although this is not always apparent from an external examination.

A reinforced column has concrete cast around a substantial reinforcement cage extending over the full length of the column. The main reinforcement diameter is likely to be not less than 10mm, and corrosion of the reinforcement, if it occurs, will take a considerable period of time. However, in flexure (bending), reinforced concrete will crack, though cracking only becomes significant where visible, and greater than 0.1mm in width. Cracking of reinforced columns can be apparent from the retention of surface moisture and mould growth, even where the crack is less than 0.1mm in width. Measurements can be taken by using a crack gauge after removal of surface deposits.

A pre-stressed column is formed around high-tensile pre-stress wires under tension with a much-reduced diameter, typically 2–3mm. These pre-stress wires hold the concrete in compression throughout its life, preventing cracking under normal loading. If cracks do appear from overloading or mishandling, there is a danger that water will be able to reach the pre-stress wires, giving rise to corrosion which can make the column unsound in a much shorter period of time than for reinforced concrete. It is therefore essential that impact or handling damage is identified and cracking sealed at the earliest possible stage. Pre-stressed columns greater than 8m height usually include an unstressed reinforcing cage, usually of carbon steel, over the base portion in addition to the pre-stress wires, to resist the effects of impact.

As a generalisation, smooth line concrete columns, such as those with a constant taper and octagonal in cross section, will be pre-stressed, whereas shaped columns with a bulbous base compartment for example are more likely to be reinforced.

Corrosion of concrete itself is not a problem, but inspection should identify where the concrete has become damaged, potentially allowing corrosion of the internal reinforcement. Examples of cracking are shown in Pictures A12.10 and


Pictures A12.10 and A12.11: Hairline cracking of concrete columns

A12.11. The column sections most prone to damage of this type are around the door opening and the bracket joint. Damage often occurs at door openings due to attempts at unauthorised access or heavy-handed access by authorised users. However, unlike steel columns, hairline cracks emanating from the corners of door openings are unlikely to be of concern.

The bracket-to-column joint, if not correctly sealed, can result in corrosion of internal connecting tubes and can cause bursting of concrete from the formation of corrosion products, which expand to a greater volume.

There are two details of bracket joint in use:

- Where the connecting tube is part of the column shaft and
- Where the connecting tube is part of the bracket.

The securing grub screws are in the bracket and the column shaft respectively.

Over-sleeved galvanised steel brackets have been used for at least the past 30 years to extend the life of a structurally sound column shaft where the bracket column shaft joint was beginning to deteriorate, leading to spalling as the corroded reinforcing bars expanded due to corrosion. If is unlikely that any structural loading information regarding these and their suitability for the task has been retained by the asset owner. It is also most unlikely that they can be removed and the condition of the concrete and any associated internal connecting tubes be established.

Where the column shaft is in good condition, showing no signs of damage or failure, the structure may be considered as sound. However if any deterioration is evident, such as that shown in Pictures A12.12 to A12.16, perhaps it should be considered for replacement and a full structural inspection undertaken by a competent structural engineer.

There have been reports of catastrophic failures, during routine maintenance when using ladders, of pre-stressed concrete columns from a particular period of construction. These columns were 5m nominal height and used mortar spacing plugs to protect the steel pre-stressing wires at the point of deflection around the base compartment. Failure was due to the spacing plugs shrinking away from the


Pictures A12.12 and A12.13: Cracking of column shaft below the sleeve bracket

Picture A12.14: Cracking at bracket/shaft joint

Picture A12.15: Poor fitting sleeve bracket 5m column

Picture A12.16: Sleeved 5m concrete column

spun concrete body of the column, allowing corrosion of the pre-stressing wires, as shown in Picture A 12.17 and Figure A 12.2. Information on inspection and corrective action was sent to all local authorities by the manufacturers at the time the potential problem was identified. It was initially believed that inspection of the inside of the base compartment would readily identify those columns at risk. However, subsequent failures have shown that some columns of this type have spacing plugs located behind the back board making them difficult to locate and check.

Picture A12.17: Potential failure at spacing plugs inside the door opening of a concrete lighting column

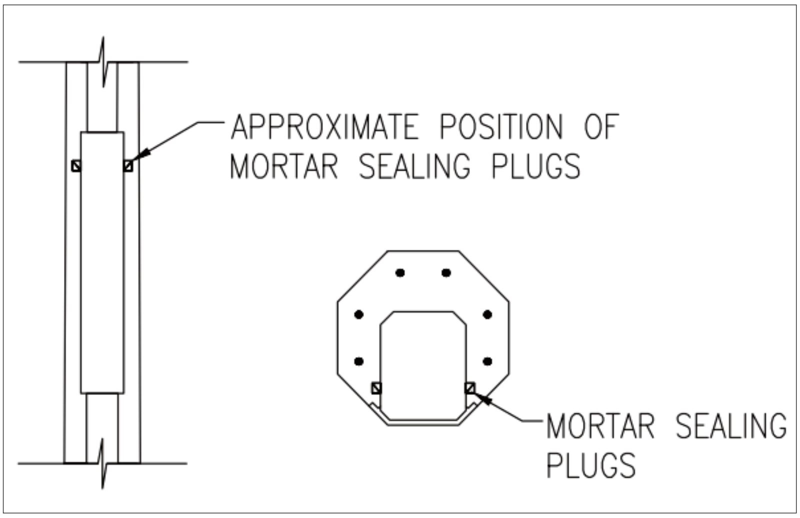


Figure A12.2: Mortar sealing plug locations

Most concrete columns will have been spun during manufacture, providing a dense concrete surface resisting the ingress of moisture. However, due to their shape, concrete brackets cannot be spun in manufacture and are therefore of a lower density. Frequently, concrete deterioration will occur in the bracket before it occurs in the column shaft and, provided the ingress of moisture into the shaft is prevented quickly, the column itself may be unaffected.

Possible solutions of replacement brackets and over-sleeved steel brackets are well tried but are not the subject of this guidance. However, bracket deterioration will generally commence either from the bracket joint or the luminaire spigot, and both areas should be examined during inspection.

Flanged concrete columns are extremely rare. Where they do occur, they are very difficult to inspect as the likely mode of failure will be corrosion of the connection of the concrete to the flange plate, and this is not generally open to inspection without the column being taken down.

Significant defects

The defects that are considered to be the most significant in concrete columns, some of which are shown in Pictures A12.18 and A12.19, can be summarised as follows:

- *Corrosion of pre-stressing wires of pre-stressed columns*: This has been significant when water has obtained access to the wires because the spacing plugs are loose or missing.
- *Corrosion of the reinforcement and cracking of the concrete at the base of columns, especially around the door openings*: The strength of the column may be reduced significantly if there are large cracks.
- *Corrosion of reinforcement and cracking of concrete brackets*: This may cause pieces of concrete to break away and fall to the ground, and allow ingress of water to the column shaft.

Picture A12.18: Deterioration at a concrete column bracket joint aggravated by lack of sealing or maintenance of sealing during column life.

Picture A12.19: Damage around the door opening on a concrete column which could eventually result in exposure and corrosion of the embedded reinforcement.

Tubular steel

Corrosion is an electrochemical reaction that can take place anywhere on a column. The amount of corrosion is dependent on:

- The effectiveness of the protective treatment provided to both the internal and external surfaces of the column;
- The presence of sodium chloride in de-icing salt;
- The presence of dog urine;
- The use of grass strimmers;
- Atmospheric pollution and acid rain;
- The presence of chemicals, for example weed killers;
- Aggressive ground conditions; for example sulphates, sulphides, chlorides.

Pictures A12.20 and A12.21: Column distorted due to impact, and column door opening installed below ground level

Water and oxygen are both required to sustain the corrosion reaction, and are likely to be freely available near columns. Hence corrosion is almost certain to occur once the protective treatment has broken down. The following sections summarise the parts of columns where severe corrosion has occurred, the effect of the site conditions and the protective treatment on corrosion, and the effect of corrosion on the structural integrity.

Above ground to just below ground level

Many authorities have reported severe corrosion on lighting columns from the door opening to just below ground level. Some reported that corrosion was worst just below ground and others that it was worst just above ground. One authority estimated that the loss of wall thickness is typically 0.5mm more below ground than above, and considers that the condition just below ground can be estimated from the condition above.

Significant corrosion has occurred at the door openings of some un-reinforced and reinforced columns, especially at the corners. Experience shows that relatively little corrosion has to occur at some un-reinforced door openings before a column becomes under strength.

There has been a serious loss of strength when the reinforcing strips have become detached at the top and bottom of reinforced door openings.

Sodium chloride is very corrosive to unprotected steel. It has increased the rate of corrosion of columns adjacent to roads and seafronts.

Dog urine has been a problem in residential areas, parks and urban footpaths. Urea in dog urine should not corrode steel, but urine usually contains sodium chloride at a concentration of about one per cent and this could be significant.

Corrosion influenced by dog urine in particular, but also other factors, is often not uniform around the circumference of columns as shown in Picture A 12.22.

Rain in industrial environments can be quite acidic due to the sulphur and nitrogen oxides in the air. The rate of corrosion is higher in industrial areas.

Although herbicides used to control grass and weeds around the base of columns have caused corrosion, this appears to vary according to the chemical composition of the herbicide.

Both dog urine and herbicides can possibly damage paint coatings due to the organic chemicals in their composition. Therefore, the increase in corrosion due to these factors may be as much to do with corrosion starting earlier, due to the breakdown of the protective treatment, as with the increase in the rate of corrosion.

Picture A12.22: Early onset of corrosion caused by dog urine

Below ground

Generally, authorities have found less corrosion on the external surfaces of columns more than 200mm below ground than near ground level. However, several authorities have reported severe corrosion at all depths in clay soils on the internal surfaces of base compartments that flood regularly.

The ease with which corrosive substances can reach a column root, hence the potential for corrosion of the root, depends on:

- What medium the column is planted in;
- How well the planting material is bonded to the column;
- The drainage falls of the planting material at ground level.

Some soils, especially clays, contain corrosive substances such as sulphates, sulphides and acidic groundwater. Corrosive substances such as sodium chloride, dog urine and weed killers are more likely to accumulate in clay soils if they drain poorly. Furthermore, the lower oxygen content of clay soils is likely to stimulate the formation of differential oxygen corrosion cells, and pitting corrosion on the external surfaces of columns above ground is demonstrated in Pictures A12.23 and A12.24.

Columns planted in granular fill appear to have suffered less corrosion than those planted in the clay soils, as the free draining nature of the fill minimises the extent to which corrosive substances can accumulate around the root.

The fall of the ground around the base of a column, particularly one planted in concrete or asphalt, affects the amount of corrosive substances gaining access to the root. Where the ground falls away from

Pictures A12.23 and A12.24: Above ground corrosion

the column, the concentration around the root is reduced and, conversely, it is increased if the ground falls towards the column. The concentration can increase further if there are crevices around the column in which corrosive substances can become trapped.

Internal corrosion at type 6 swaged joints

Significant corrosion has caused the shaft of several tubular steel columns with a type 6 swaged joint to collapse. Some columns have been less than 20 years old, and the extent of the problem has prompted some authorities to replace all columns of this type.

The corrosion has occurred hidden on the internal surfaces of the shaft, a few centimetres above the lower end of the shaft. Water has collected on the ledge formed by the top of the swaged base section and in the crevice between the base and the shaft. Generally, the corrosion has been highly localised along the shaft, and has not always extended around the full circumference, examples of which are shown in Pictures A12.25 to A12.27.

Authorities have identified the columns most at risk as those with two types of bracket that enable water to enter the shaft. One type has a male spigot that fits into the shaft. The gasket between the bracket and shaft has sometimes not been

Pictures A12.25 and A12.26: Corrosion from internal moisture collection

Picture A12.27: Internal corrosion from swage joint

fitted at installation or it has failed in service. The other type of bracket extends upwards from a flat flange that is fitted to the top of the shaft by fixing screws. Sometimes none of the screws has been fitted.

Authorities have found minimal corrosion when these two types of bracket have been installed correctly. Very few cases of water ingress and subsequent corrosion have been reported when other types of bracket have been fitted. This suggests that condensation on the internal surfaces of columns has not been a major cause of corrosion. However, any changes in section or joints that provide an internal ledge or water trap may enable corrosion to occur hidden on the internal surfaces of a column, so they should not be ignored.

Door openings

Where the door opening has been strengthened by reinforcing, dependent upon the detail there is the potential for corrosion traps, particularly if the column has not been hot-dip galvanised. Examples are shown in Figure A12.3 and Picture A12.28.

Bracket tube connection (Elbow)

The most likely area for failure of brackets is at the connection of the horizontal portion of the detail to the vertical portion of the bracket, as demonstrated in Figure A12.4. These details vary between

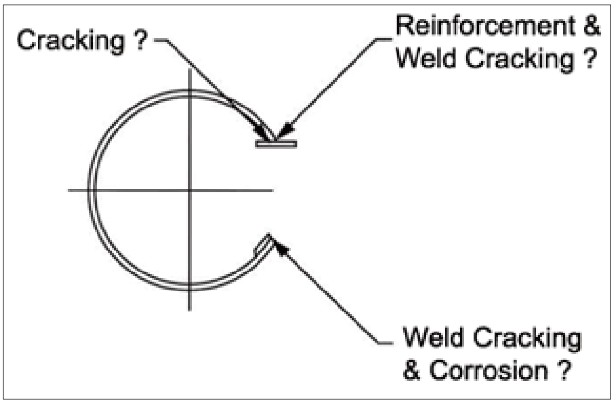


Figure A12.3: Door opening corrosion points

Picture A12.28: Crack from edge of door opening

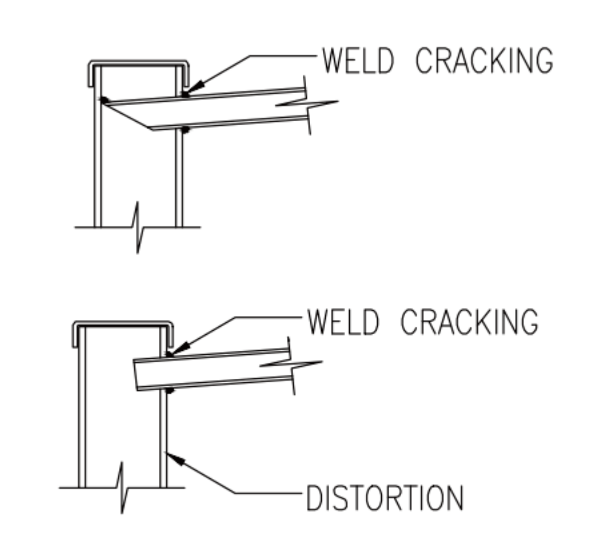


Figure A12.4: Cracking of bracket are connections

manufacturers. Loadings are generally light, except in the case of very heavy luminaires with long projection brackets. In such cases, the provision of a bracket gusset will strengthen this joint.

Corrosion of flanged columns

Very little information is available on the condition of flanged columns. Some flange plates are totally above ground and are susceptible to the same type of corrosion described previously in this appendix. Flange plates that are buried, including the holding-down bolts, can be susceptible to the effects of the ground conditions as described previously.

Folded/sheet steel

General

These columns are normally formed in steel plate with eight sides. However, six, ten or twelve sides and irregular octagons are also frequently used. Many of the problems are similar to those of tubular steel columns, discussed above, and are therefore only listed briefly below. However, sheet steel columns are more prone to minor impact deformations than tubular steel columns resulting in dents or distortions in the section of the column that can substantially reduce its strength.

A buckle or crease across one face of an octagonal column, including the two adjacent corners, which results in a deformation of only three or four times the material thickness (typically 6–10mm) can result in the section not being able to sustain the wind loading for which it was originally designed, as demonstrated in Pictures A12.29 and A12.30. All too frequently, such damaged sections are carefully painted to prevent corrosion without it being realised that the column

Pictures A12.29 and A12.30: Damaged sheet steel column due to impact, and onset of failure at a sharp cornered door, from which cracks can propagate with time should have been replaced for structural reasons.

Door openings

These may be either reinforced or un-reinforced, although with sheet steel columns, reinforced door openings are more common. Some designs use a projecting hoop around the door opening, allowing the use of a flat door. Even with this form of reinforcement, an adequate corner radius is essential, with welds in the hoop positioned away from the corners. Any welds in the immediate vicinity of tight radiused corners should be inspected for cracking.

The welded connection of the reinforcing hoop to the shaft wall is essential for structural reasons, particularly at both ends of the door opening. Failures in this area are virtually unknown for properly designed columns, but the importance is mentioned here so that inspectors can appreciate the need for these welds to be intact; the welds down the sides of the door opening, though still necessary, are of less importance.

Picture A12.31; Severe column distortion caused by impact damage

Flanges

These are subject to the same considerations as tubular steel columns.

Roots

As for tubular steel, corrosion of the root can become a significant factor in the life of the column, particularly as material thicknesses for sheet steel columns are likely to be less than for tubular steel columns.

Shaft joints

Welded joints

For longer columns, transverse joints may be necessary so as to limit the length of sheet used in fabrication. The usual method of connection is an overlapping sleeve with an external fillet weld at the lower edge. The potential for internal corrosion is the same as with the swaged shoulder joint in tubular steel columns, and the importance of galvanising is therefore clear. The length of the overlapping section should be determined from drawings or by thickness measurements so that the area just above the overlap of the joint can be checked.

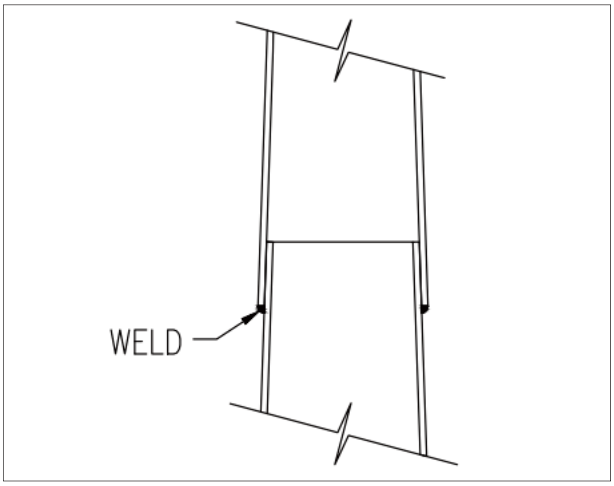


Figure A12.5: Shaft overlapping joint; potential corrosion points

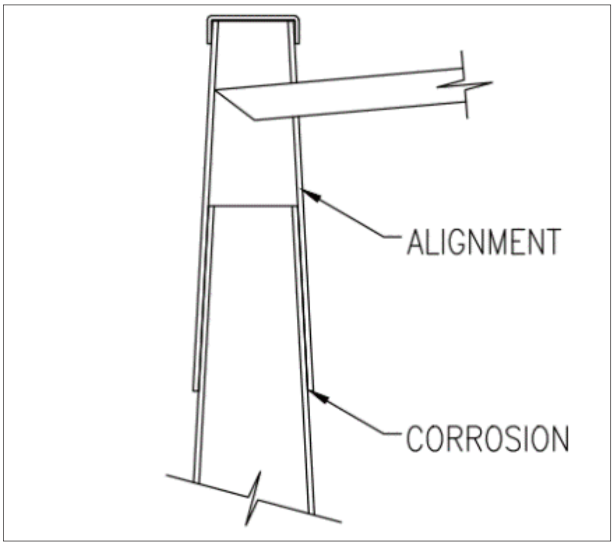


Figure A12.6: Shaft bracket locations of potential concern

Dry sleeve joints

For segmental columns or for columns of considerable height requiring site joints, the usual form is an overlapping taper sleeve without welding. As above, the presence of galvanising is important for these joints to prevent internal corrosion at the step, but the lack of the weld does allow a further inspection possibility as internal corrosion will generally result in leeching of corrosion products below the joint on the external surface.

Bracket joint

- *Over-sleeve*: As for tubular steel columns.
- *Bracket fixing on column*: As for tubular steel columns.
- *Dry sleeved*: Frequently, brackets are fixed at the top of tapered sheet steel columns in the same manner as a dry sleeved shaft joint above, and inspection and testing principles are the same.

Luminaire fixing

As for tubular steel columns.

Significant defects

As for tubular steel columns.

Fold-down columns

Past problems with fold-down columns have been found at the hinge, with cracking of the main hinge support (Pictures A12.32 and A 2.33), as well as corrosion and material loss around this area resulting in the performance of the hinge joint failing (Pictures A12.34 and A12.35).

Picture A12.32: Cracking at hinge support

Picture A12.33: Corrosion and cracking at hinge support

Picture A12.34: Corrosion at hinge support

Picture A12.35: Failed hinge joint

Aluminium

Corrosion in aluminium is unlikely to occur above ground level except in the first 300mm where chemical attack can occur from the unwelcome attention of dogs or the use of certain weed killers. This area should normally be covered by the root protection system. Inspections should be carried out to identify signs of corrosion at this location, as well as impact damage and possible failure from fatigue or vandalism. Cracking of welds or castings at the shoulder, flanges or changes in section should be checked using the same principles given for other columns.

Aluminium columns are also subject to electrolytic attack as aluminium is generally sacrificial and will therefore suffer loss of section where electrolytic action takes place. This can occur in the root section if the root protection is not intact and the ground conditions are such that an electrolytic cell can be set up. In general, the production of aluminium columns in the last 20 years has resulted in satisfactory root protection but, prior to this, where root protection was very poor or non-existent, corrosion and structural failure could occur in ground saturated with road salts.

Another potential corrosion problem is associated with the use of dissimilar metals. Although rare, cases have been seen where backboard fixings have incorporated unprotected steel which, in the presence of moisture, can result in aluminium being lost around the connection. Great care needs to be taken if earth rods or mats are to be installed near aluminium columns; galvanic action will cause the root section to corrode very quickly.

On flange-plated aluminium columns, it is often considered that separation is required between the aluminium flange plate and the connecting bolts. While, in theory, electrolytic action can be set up, in practice it is very small if the bolts are satisfactorily galvanised or are of stainless steel. Such electrolytic action is evident prior to serious loss of section by pitting around the area.

Tubular aluminium columns

Tubular fabricated aluminium columns are generally of two types, older 5 metre columns (> 30 year) consist of a cast base section with an extruded aluminium tube for the shaft, or a single-piece extruded tubular section. Columns can also be formed by a spinning and drawing process to give both tapered and stepped profiles without joints. The required inspection will be dependent on the design of fabricated bases, as stress concentrations may not arise at what appears to be a right-angled door opening due to the use of extruded sections continuing past the opening.


Pictures A12.36 and A12.37: Cracking from door opening and corrosion of the base compartment

However, inspection of the corners of door openings in castings and of the connecting welds of extruded fabrications should form part of the inspection

Tapered aluminium columns

In addition, there may be specialist fabricated sheet aluminium columns where the assembled sections using cut extruded profile sections which clip together. Such sections will sleeve in part with the other members, but will have to be fixed by means of adhesives or welding at intervals and ends.

Significant defects

The defects that are considered to be the most significant in aluminium columns can be summarised as follows:

- *Corrosion of the root of planted columns*: This can occur in aggressive ground conditions, but is most prevalent in poorly-drained ground, ie clay soils, where de-icing salt may be retained for long periods.
- *Corrosion due to electrolytic action*: Corrosion can occur in the presence of dissimilar metals such as earth rods. Particular attention should be given to backboard fixings and foundation bolts.
- *Cracking at welded joints*: Cracking can occur at welds connecting extruded sections.

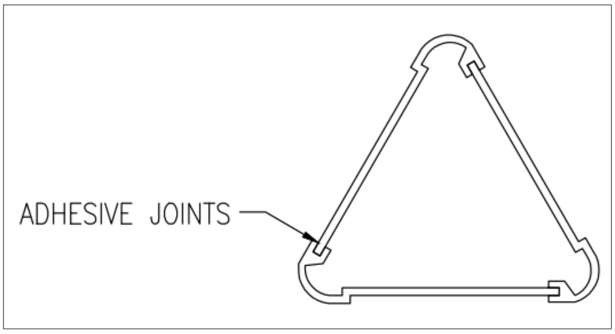


Figure A12.7: Assembled joint section

- *Failure of adhesives*: Failure of adhesives holding extruded and fabricated sections together.

Fibre-reinforced polymer compound (FRPC)

General

There has been only a small use of fibre-reinforced polymer compound (FRPC) columns in the UK, and these have generally been restricted to the lower mounting heights. The two main disadvantages with FRPC columns are high cost of manufacture and poor performance due to high deflections. Such columns rarely suffer from conventional corrosion aspects and are far more prone to physical damage from impacts or deliberate vandalism. However, two effects that have occurred with age can give rise to problems and deterioration of the structural integrity of such columns, dependent on the constituent materials.

The first affects FRPC columns and consists of delamination of the material. This will be seen as local swellings and can be located by tapping the surface to hear the change of sound where a void has been located. Eventually, such delamination can become completely detached and peel off in layers, leaving a section of considerably reduced strength. Delamination generally affects older columns that have been manufactured using a manual layering process. Modern spun or woven FRPC columns have a denser and more uniform composition of the reinforcement and the resin and are less prone to delamination as they are manufactured in one process.

The second effect that can occur with reinforced plastic columns is the effects of ageing due to UV light. Plastics subjected to ultraviolet light can become brittle allowing crack propagation from stress concentrations, such as the door opening or bracket joint or, indeed, the ground line. Both effects can be determined fairly reliably by visual inspection.

Significant defects

The defects that are considered to be the most significant in FRPC columns can be summarised as follows:

- *Impact damage and vandalism*: FRPC columns are prone to accident damage and vandal damage. Particular areas of concern are at ground level and around the door opening.
- *Delamination*: This can occur at any point on the lighting column and is first seen as a local swelling which eventually may completely delaminate and detach from the column leaving it weakened.
- *Effects of UV*: Reinforced plastic columns can suffer from the base material becoming brittle due to the effects of ultra violet light unless the correct stabilisers are used in the material during manufacture. This is seen in the form of cracking and crazing of the column at ground level, around the door opening and at bracket joints.

Plant growth

Where plant growth such as ivy has used a column for support it should be reported as soon as it is identified. Operatives attending such structures to undertake routine or reactive work should be aware of the effect such growth has on the structure, report it, and consider if they should proceed with their task.

Such growth not only brings a considerable wind loading to the structure but can also weaken its integrity especially on concrete columns that are not structurally sound. It also makes visual and other testing methods at best difficult if not impossible to undertake. This is particularly the case with common or English ivy which supports itself by aerial roots which can penetrate any cracks or joints and have the potential to cause structural damage through the widening of cracks and allowing moisture to enter the structure.

Care should be taken when removing ivy as ivy roots may damage the paint when taken away. Where a column or support has been cleared of ivy the structure should be inspected and tested.

Picture A12.38: Significant ivy growth on concrete column

Picture A12.39: Ivy cut back to give door access

Pictures A12.38 and A12.39 indicate a concrete column, fitted with a galvanised sleeve bracket many years ago which has been retro-fitted with an LED luminaire. The operatives appear to have trimmed the ivy to access the door and undertake their work. Months later the position remains the same. Ideally, the ivy should have been removed prior to the replacement lantern being fitted to ensure the structural integrity of the column had not been compromised.

Appendix 13
Corrosion of steel and aluminium lighting supports (dissimilar metals)

Introduction

This section addresses the various issues relating to the corrosion which affects steel and aluminium supports in the UK.

When corrosion does finally become apparent on a support, it is the end result of a series of events from the conception of the support, through the fabrication and installation of the support, to the various conditions that the support has experienced during its lifetime.

The aspiration is to produce supports that will be fabricated, installed and maintained in an economical and efficient manner for their proposed lifetime whilst respecting other conditions as dictated by the client, such as the aesthetic aspect.

If the choice is directed towards a metallic support, the base material and the protection afforded to it are important considerations when estimating the resistance of the support in the environment in which it is located.

The metallic materials available can be summarised as follows:

- Cast iron – available in tubular sections and decorative castings;
- Tubular steel – mainly circular sections but square and elliptical are available;
- Folded/sheet steel tubes – generally octagonal pressings with overall taper to the shaft length;
- Cast aluminium – aluminium with various alloying elements;
- Extruded aluminium tubes.

The main components with respect to the environment can be summarised as follows:

- Impacts from small objects – for example gravel projections from vehicle wheels;
- Impacts from maintenance tools – for example grass trimmers;
- De-icing salts – as salted on to the roadway;
- Urine – for example in parks or well-known walkways for dogs;
- Vandalism.

Metal corrosion

The corrosion of metals occurs when oxides and hydroxides form from the parent metal and thereby often help to create even better conditions for the corrosion to continue.

The basic mechanism of metal corrosion in atmospheric conditions requires the presence of both oxygen and water; oxygen is the principal partner material, which associates itself with the metal formed, whilst the water is the medium through which the ions of the two materials travel. Crucially, the metal oxide will associate itself with molecules of water to form a much more voluminous product.

Atmospheric water (rain) is initially a weak acid due to the presence of dissolved carbon dioxide but it can become more active as it absorbs additional ions such as de-icing salts, and even more so as the pool of water dries out and attains a concentration even more favourable to corrosion. The duration and frequency of the wet-dry cycle will define the period of adverse corrosion conditions. The general environment (as just discussed) and the local geometry of the affected areas (particularly details which allow the persistence of wet-dry areas) can combine to generate adverse corrosion conditions.

Steel corrosion

Steel products come in various forms which corrode in different fashions. The two types of steel product which used for steel supports are cast steel and laminated steel, noting that the tubular steel supports are fabricated from laminated steel plate.

Cast iron

The crystalline molecular structure of the cast steel – where the carbon is precipitated into localised areas – does not create the ideal conditions for deep rust development. Effectively, the rust forms a powder on the surface of the metal which is washed off by the rain and is then reformed. The depth of corrosion is generally minimal even over a long period of time, although severe conditions can accelerate the corrosion rate. The rust on a support will form after the removal of the protective coating, which is unsightly; however it is rarely structurally important due to both the limited depth of corrosion and the thickness of the support material which means that the percentage loss is quite small relative to the geometrical properties.

Steel tubes

The process of fabricating laminated plate creates layers of steel which become apparent when the rust layers develop. The layers within the steel structure create a slight weakness in the transversal direction in comparison to the longitudinal direction; this is more apparent in the thicker sections (>40mm) but the effects are still apparent when rust develops in the thinner sections.

At the instigation of the rusting process, two adjacent layers allow for a crevice where the humidity can linger and so prolong the wet-dry period and so corrosion increases. The plates are further pushed apart by the volume of the iron oxide molecule with its associated water. This in turn creates conditions which accelerate the corrosion as the crevice increases in length and so the humidity lingers longer.

The two main products for supports that are formed from laminated plate are the structural tubes and the folded-plate tubes. The structural tubes generally vary in thickness from 2.5mm to 5mm whilst the folded-plate supports tend to be of thinner plate. The layering of the rust is less visible in the thinner plate as the corrosion works its way through quite quickly, although flakes are still visible.

Aluminium corrosion

Aluminium is well known as having a passive outer oxide layer that will prevent further corrosion in a dry atmosphere. Even when in contact with pH-neutral water, the oxide layer is well adhered to the base metal. Problems arise when salt solutions or acid solutions are present (in our situation equivalent to road salting and rainwater) meaning dissolution of the oxide layer occurs and further oxidation can take place.

Many of the aluminium alloys contain alloying elements which give rise to pitting corrosion, for example copper in high-strength alloys. This will be most corrosive in the presence of chloride electrolytes.

Although aluminium is relatively resistant to corrosion and does not require paint protection there may be times when for aesthetic reasons the columns are painted. This presents certain problems as the paint needs to first react with the oxide layer so as to adhere to the body of the material. However, it is inevitable that this paint layer will be slightly damaged due either to fastening in openings or to physical damage from its environment.

This leads to a particular type of corrosion called poultice corrosion, where the initial damage provides a crevice between paint and metal, and the aluminium oxides/hydroxides which form push the paint away and so the crevice increases in size. Blisters under the paint system then appear till they are broken and another point of entry for the electrolyte with adverse corrosion conditions is created.

Galvanic corrosion

Galvanic corrosion – also referred to as bimetallic corrosion, dissimilar metal corrosion or contact corrosion – occurs when two dissimilar elements are in contact or in very close proximity and an electrolyte is present. In the case of external supports there are a number of causes that will create galvanic corrosion:

Metal being considered (a) / Metal in Contact	Magnesium	Zinc	Aluminium alloys	Carbon Steel	Cast iron	Lead	Tin	Austenitic Cast Iron	Brasses	Gunmetals / Tin Bronzes	Copper Nickel Alloys	Nickel Aluminium Bronze	Alloy 400/K-500	Low alloy stainless steel	Nickel-chrome moly alloys	Titanium	High Alloy Stainless Steel	Graphite
Magnesium	Unlikely	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur
Zinc	Unlikely	Unlikely	Possible	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur
Aluminium alloys	May occur	Unlikely	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur
Carbon Steel	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur
Cast iron	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur
Lead	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur
Tin	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur	May occur
Austenitic Cast Iron	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Possible	Possible	Possible	Possible	Possible	Possible	May occur	May occur	May occur	May occur
Brasses	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Possible	Possible	Possible	Possible	May occur	May occur	May occur	May occur	May occur
Gunmetals / Tin Bronzes	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Possible	Possible	May occur	May occur	May occur	May occur
Copper Nickel Alloys	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Possible	Possible	May occur	May occur	May occur	May occur
Nickel Aluminium Bronze	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Possible	Possible	Possible	Possible	Possible	May occur
Alloy 400/K-500	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Possible	Possible	Possible	Possible	May occur
Low alloy stainless steel	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	May occur	May occur	May occur	May occur
Nickel-chrome moly alloys	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Possible	Possible	May occur
Titanium	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Possible
High Alloy Stainless Steel	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	May occur
Graphite	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely

Additional corrosion unlikely | Additional corrosion may occur | Additional corrosion possible

Table A13.1: Galvanic table showing the risk of additional corrosion from bimetallic contact in neutral aqueous electrolytes (each metal should be read against the other, the degree of corrosion always refers to the metal on axis A)

- An electrolyte bridging the two metals, such as rainwater with dissolved salts and dissolved carbon dioxide allowing the passage of the metal ions so that the oxides and hydroxides can readily form;
- Electrical connection between metals, such as a brass earthing system fitted to an aluminium support;
- A sufficient difference in potential between the two metals to provide a significant galvanic current.

The reactions which occur are similar to those disused above, but the rate of attack is increased, sometimes dramatically.

The different elements commonly encountered in galvanic corrosion of supports are:

- Steel
- Aluminium
- Zinc (galvanised steel)
- Chrome (stainless steel fasteners)

Looking at the galvanic table (Table 13.1), where elements are noted according to their propensity to exchange their electrons, some of the elements would seem to be reactive with each other as they have very different galvanic indexes. However, in a practical situation, other factors are of importance:

- Environment: wet-dry cycles, electrolyte concentration, etc;
- Relative size of anode and cathode;
- Protective oxides/hydroxides.

As demonstrated above, care should be taken when designing with or incorporating more than one metal on the same support and, in most cases, it should be avoided or mitigation/insulation measures put in place.

A secondary danger of galvanic action occurs when additional items are fixed to the body of the support – for example signs with stainless steel fixing bands – which damage the paint protection and then expose the internal base material to the chrome in the stainless steel and brass earth bolts on aluminium columns. Another cause of galvanic action can also occur when earth electrodes are installed near aluminium planted columns, where the aluminium rooted section can erode in a very short period if, for example, a copper or brass earth electrode or mat is installed close to the rooted section.

Prevention

The extent to which bimetallic corrosion is likely to occur and the selection of countermeasures should be considered at the design stage. Ideally, metals should be selected that are close to each other in the galvanic series, but engineering requirements for different material properties in various parts of equipment or an installation often necessitate the use of several different metals.

A major consideration, however, is to identify the composition and conductivity of the environment, since these will play a significant role in defining the extent and severity of any corrosion. Other objectives are to maximise the area of the anodic metal and minimise that of the cathodic metal. Experience in similar situations should be sought where possible because it is not uncommon for dissimilar metals to be coupled without adverse effect, especially when the electrical conductivity or oxygen content of the electrolyte is low.

Areas where moisture can collect and hence corrosion begin need to be designed out or suitable protective finishes applied.

Here it is important that the design effects good drainage and thus minimises accumulation of condensation and rain-water at joints and in hollow sections with, say, the internal construction joints of tubular steel columns.

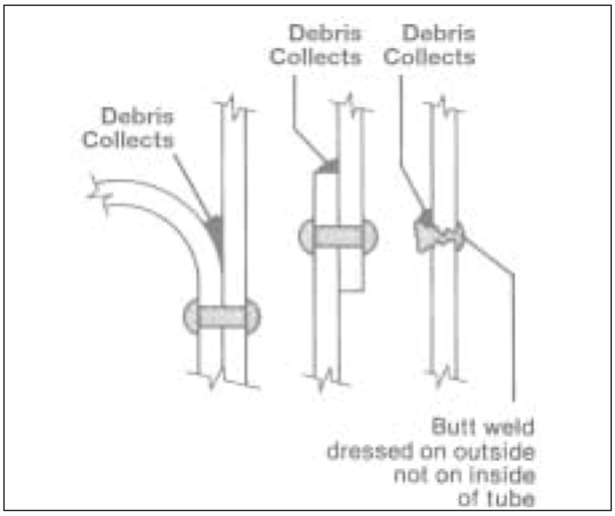


Figure A13.1: Effect of design considerations

Protection systems

Several protection systems are available, most of which consist of creating an external coating to the support which prevents the oxygen and water (with the dissolved salts) from coming into contact with the base metal.

All of the coatings have a finite life since they are under continuous attack from the rain, the sunlight and other environmental factors. If no damage to the surface occurs, the coatings have a predictive life which goes beyond the normal life of a support.

Given the environment in which the supports find themselves, it is inevitable that some damage will occur where the protection system is removed. Depending on the nature of the coating and the severity of the damage, the protection system can either repair itself or can be repaired during maintenance.

Galvanisation

The two types of galvanising most common are hot-dip galvanising and electro galvanising. Generally, hot-dip galvanising is applied to steel used for structural purposes. The process is applied to steel supports which are cleaned, pickled (to remove grease and salts from the surface of the metal), dried and then dipped into a vat of molten zinc. The resulting reaction between the steel surface and the molten zinc creates multiple layers of zinc-iron compounds which adhere very well to the surface of the steel. The thick external layer is zinc which, on contact with air, produces a very thin stable layer of zinc oxide on the surface. This protects the zinc layer from further oxidation.

If the galvanising is left bare to the environment, the rain will wash the zinc oxide off over a long period of time (50 years or more). If that environment is salted, the zinc is very quickly eliminated as the zinc oxide reacts with the salts producing other zinc compounds which are easily diluted by water and quickly washed off, allowing more zinc to enter the cycle. This shorter time period can be as little as two years, therefore care needs to be taken regarding protective coatings in aggressive environments such as coastal locations where there will be a high atmospheric salt content.

When small damage occurs, such as scratching within the depth of the zinc coating, the zinc oxide will reform. However, if deeper damage is made which goes through the zinc layer, the steel will become exposed and a galvanic reaction will begin between the zinc and the steel which can spread quite quickly forming large rusty patches. On supports, this type of corrosion tends to occur in the lower regions of the support where physical damage is more likely.

Painting or thermo-lacquering on galvanised steel presents similar problems to painting aluminium insomuch as the zinc oxide layer is very difficult to adhere to and so requires specific preparations and/or painting primers for the paint protection to be effective. In a similar way to the aluminium, the poultice corrosion phenomenon can also occur where the zinc oxides/hydroxides grow between the paint layer and the base material and so create a corrosion acceleration locally which allows blisters to form under the paint.

Repair of painted galvanised supports also requires careful preparation and the correct paint repair systems. This needs to take account of the zinc oxide layers in place and to be able to create a good adherence between paint system and base material otherwise the paint system will flake off causing an unsightly appearance.

Paint

The paint system prevents oxygen and water attaining the inner base material. The systems are generally 2-pack or 3-pack systems with a primer coat/intermediate coat/finishing coat. Paint systems have developed substantially in the past 25 years. Today, we have a choice between several types of paint system including:
- VOC paint systems (Volatile Organic Compounds);
- Non-VOC or low-VOC paint systems;
- Brush or roller applied;
- Sprayed-on;

Pictures A13.1 and A13.2: Debonding of paint due to poor surface preparation

- Thermoplastic;
- Powder coating.

The basic lifespan of a paint systems varies from 10 to 25 years in normal environments, not counting the heavy-duty 50-year life systems for marine environments. This is the lifespan with respect to its effectiveness as a protection against corrosion of the base material.

Two other criteria are useful for defining a paint system's life: aesthetic appearance period and life to first maintenance. Aesthetic appearance is mainly defined by comparison to a predicted visual degradation over a period of three, six or ten years. Samples are provided which can be compared to the time period considered. Life to first maintenance is the period where no particular maintenance is required in terms of treatments to the paint system. After this initial period – depending on the weathering of the paint system – a treatment may be applied such as a re-coating of the finishing coat.

Normal corrosion of the paint system under normal environmental conditions takes the form of the finishing coat being affected by the rainwater, temperature and direct sunlight which degrade the paint structure. The final product is a fine dust on the surface which will eventually expose the primer coat and then the base metal (or galvanisation if the two protections are combined).

Paint systems are particularly prone to physical damage and the poultice corrosion phenomenon is the principal mechanism by which corrosion is rapidly propagated.

Powder coating

Powder coating has the particularity of not using VOCs in its production. The powder is applied electrostatically and is then baked. These coatings tend to have a better adherence to the base material and are also harder and more difficult to chip when exposed to physical damage. When it becomes chipped or cut it can allow moisture access below the coating and hence to the structural material. This is particularly an issue with older powder coating systems compared to modern systems where, if the coating is applied correctly, debonding should not be an issue. See Pictures A13.3 and A13.4.

Resin casing

A sheath of resin with glass fibre reinforcement is placed around the base of the support and glued to it. This protects the base of the support from physical impacts and also reduces the risk of high concentration wet areas at the base of the support.

Repair systems

The repair of the protection systems is limited to in situ application. The area needs to be cleaned and prepared for the repair system. This is not always easy to do as it relies on good access to the areas needing repair and also suitable atmospheric conditions: not raining and above 15°C are the two main conditions for any repair.

Pictures A13.3 and A13.4: Damaged powder coating where moisture has entered and caused corrosion behind the coating

The point of corrosion is often at or below ground level where the wet-dry cycle creates the worst corrosion conditions. This is the most difficult area to repair as it requires excavation.

Environments

Local conditions around various parts of the individual lighting support will influence the life of the protection system. Detailing of the lighting support can alleviate or aggravate the effects of local conditions and, particularly when detailing permits, the retention of wet and humid areas which would create optimum concentrations of electrolytes over longer periods of time.

Wet/dry cycles

Although rainwater will wash off the outer protective coatings, this is part of the normal lifespan of the protective coating and so would not significantly alter its anticipated life.

The main areas of corrosion occur where water pools can dry to smaller pools and so become more concentrated in acidity and saline content. These areas tend to be located either at the base of the support due to poor pavement installation, at areas where design details such as embellishments have created a flat or dished area, or at flat spots created by attachments.

The base of the support is the place where most corrosion can be expected as there is systematically an interface between the support material and the ground. Hence a crevice is always present which will allow for an electrolytic pool of salts from road de-icing material and urine to accumulate in increasingly higher concentrations.

Since these pools will form in the same area of the lighting support and not elsewhere, one part of the lighting support tends to be affected. A lighting support can seem to be in a good state of repair but have one area which is badly affected. In the case of base corrosion at ground level, the corrosion is occurring at the point of highest stress and can be hidden just beneath ground level.

Salt/grit

The standard winter maintenance of main roads and also secondary roads includes the use of salt/grit to enable vehicles to circulate safely in low temperatures. Although most of the salt/grit goes on to the roads, circulating vehicles can project the salt/grit onto the lighting support or the vicinity of the lighting support base. As noted in the wet/dry section above, the concentration of the salts increases as the wetness decreases over time thereby creating conditions even more conducive to corrosion.

Urine

Although this can be seen as a anodyne cause, when dogs see a support during their walk they do have a tendency to leave their mark. When one particular support is in pole position – such as the first support in the park, or the park most favoured by a series of dogs – that support will tend to accumulate throughout the entire year a series of salted urines that will concentrate at the base of the support and probably on the same side of the support.

Grass

The presence of grass implies there will be a maintenance regime of cutting. This is done mechanically with strimmers, and so multiple impacts of the nylon or steel wire tines are in the same area at the base of the support. This will ultimately affect the adherence of the paint system to the base material and will also puncture any blisters formed from other corrosion.

Street cleansing

Modern street cleansing vehicles and processes can be particularly harsh and

Pictures A13.5 and A13.6: Assorted street cleansing systems aggressive and can cause damage to the protective systems at the base of the installation near the ground. Such aspects should be discussed between the asset owner and those responsible for the cleansing of the surrounding area to avoid the potential for damaged to protective coatings.

History of columns and their corrosion in UK

As discussed earlier, lighting columns have existed in the UK since the middle of the 18th century. The multitude of types of column installed over the years – and particularly over the last 70 years – now leaves the current stock with a large variation in the type of column, the age of the column and the particular environment in which it is located. All of these factors will have affected the state of corrosion on the column.

Future evolution of column corrosion issues

New technologies are made available and so new issues with respect to corrosion will arise. When a new type of lighting column is proposed or when an original way of combining existing column technology with some new development is proposed, the various mechanisms of corrosion should be considered.

Likewise, new maintenance techniques may become available such as protection to the base by patches or new coatings to be applied on site. Each type of proposal should take into consideration the different possible corrosion mechanisms.

Cost effectiveness of protection systems

Although a type of column in a given environment may give the longest life for that column, the type of the chosen protection system may not be the most cost effective.

The resistance of the column to corrosion will also depend on the maintenance regime that is put in place which has itself a cost.

Maintenance to reduce corrosion may include:

- Reduction of physical damage using protection around the base;
- Repainting of columns at the optimal moment for repainting;
- Removal of unhelpful additional materials.

Appendix 14
Corrosion and cracking of concrete columns

Corrosion of steel in concrete

General

Normally steel (for example reinforcement) embedded in sound, uncontaminated concrete is protected against corrosion. This is because good-quality concrete results in an environment of high alkalinity (typically pH >12). This environment forms as a result of the hydration reactions that take place during construction when hydroxides of sodium, potassium and calcium are produced.

The high pH conditions of concrete result in the formation of a stable, protective, iron oxide film on the reinforcement surface. This passive film is protective and ensures that, if the concrete conditions don't change, the corrosion rate of reinforcement is essentially negligible (at less than 0.1μm/year).

Unfortunately, over time the concrete conditions can change and if contamination occurs then the passive film can be disrupted. In practice, the two most common causes of corrosion of reinforcement in concrete are the carbonation of the surrounding concrete, which reduces the pH, and the contamination of concrete by chloride ions.

Carbonation is the term given to the neutralisation of the naturally high alkalinity in the concrete, resulting in a reduction in pH over time. This neutralisation arises as a result from exposure to atmospheric carbon dioxide. A reduction in concrete pH may also arise over time from exposure to other acidic conditions.

Chlorides act as de-passivation anions. If they reach the reinforcement surface they can result in damage to the passive film and in such cases corrosion may commence. The following sections describe the mechanisms by which these deterioration mechanisms work.

The cover concrete to the reinforcement acts as a barrier to the contamination that results in carbonation and chloride ion attack. In general terms the greater the cover and the better quality the concrete the longer the time period is before the reinforcement will be at risk of corrosion.

Whilst typically concrete columns were factory manufactured (and so not subject to the type of construction defects that can be found with cast-in-situ concrete structures), the necessity for slender sections, coupled with the fact that the deterioration mechanisms of reinforcement in concrete were not fully understood at the time of manufacture, has resulted in a large stock of concrete columns with less than ideal concrete cover.

By way of example, the typical concrete cover found in as-constructed columns will be significantly less than today's standards[1] for reinforced concrete which would require a minimum cover of 20mm for columns exposed to atmospheric carbonation and 40mm for columns exposed to chlorides in order to provide a 50-year life.

Often in columns the link reinforcement has the least cover as it was tied around the perimeter of the main reinforcing bars. When corrosion occurs, it is often at the corners of the link reinforcement where the least cover is provided.

1 BS 8500-1: 2006 Concrete – Complementary British Standard to BS EN 206-1. Table A.4. Concrete classes XC3 and XD4

Carbonation of concrete resulting in reinforcement corrosion

Concrete is a microporous material with a continuous pore structure throughout, which allows for gaseous diffusion to occur.

The general method by which carbonation can occur is as a result of carbon dioxide in the air. This reacts with moisture within the concrete which results in the formation of a weak carbonic acid. This allows acid–base reactions to occur within the micropores, leading to the neutralisation of the alkaline phases in the concrete.

Exposure to other weak acids (one example is dog urine, at a pH of 6 to 6.5) will also increase the risk of pH reduction of the concrete, with similar consequences to carbonation.

The rate at which the carbonation front proceeds through the cover concrete (from the concrete surface to the depth of the reinforcement) is dependent on the rate of gaseous diffusion (within the concrete pores) and the quantity of alkali in the cement paste matrix. The more permeable a concrete, the more at risk of carbonation it is.

When the carbonation front has reached the depth of the reinforcement and the pH at the concrete surrounding the reinforcement is reduced to below circa pH10, the protective passive film is no longer stable. Once these conditions occur, corrosion can initiate, but the presence of moisture in the carbonated zone is essential for corrosion of the reinforcement to occur.

Once corrosion has occurred, the extent and rate of subsequent corrosion will depend on the ease with which the anodic and cathodic reactions that result in corrosion at anodic sites can occur. Factors that may affect the rate of corrosion include the availability of oxygen and the availability of water at the reinforcement surface, which in turn are determined by concrete quality.

Chloride contamination of concrete resulting in reinforcement corrosion

Chloride contamination may occur from various sources. Two main sources that can affect columns are de-icing salts and wind-blown chlorides.

De-icing salts applied during winter months may result in contamination as a result of direct application, for example on pedestrian footways. A more significant cause is the application of de-icing salts to highways. In addition to the direct exposure to columns that may arise during the distribution of salt, contamination from vehicle spray can be widespread. Standards[2] identify that concrete structures are classed as subject to exposure from de-icing salts if they are within 10m of a carriageway subject to de-icing salt application, or if they buried to a depth of <1m below carriageway level where de-icing salts are applied.

Windblown chlorides are present in coastal locations where air carrying salt originating from sea water can result in contamination.

Other sources of chloride contamination can include exposure to saline groundwater (which may be naturally occurring or as the result of the run-off of de-icing salts) or, in older concrete structures, chlorides were sometimes cast into the mix through constituents such as calcium chloride-based set accelerators. This practice was stopped in the UK by around the mid-1970s.

Where concrete is subject to an external source of chlorides then the transport of

2 BS 8500-1: 2006 Concrete – Complementary British Standard to BS EN 206-1 Table A.1 Exposure Classes

ions through the cover concrete can be by various methods:

- Diffusion through the water-filled pores of the concrete (the most common type for items such as columns);
- Absorption;
- Wick action, whereby the lower section of the columns may be immersed or saturated and the section above drier;
- Capillary action through capillary pore networks.

The time taken for chloride ions to get to the reinforcement is therefore subject to several factors, but most important is the depth of cover concrete and the concrete quality. In practice, the time taken for chlorides to reach the reinforcement can vary from months to decades.

Once the chlorides have reached the depth of the reinforcement they disrupt the passive iron oxide film, and corrosion may initiate. The amount of chloride required to initiate corrosion is not a fixed value and remains subject to technical research and debate.

Current best-practice guidance on the extent of chlorides required to initiate corrosion in concrete is presented in Building Research Establishment (BRE) guidance.[3] It should be noted that the extent of chloride required for corrosion to occur can depend on several factors and cannot be represented by a single value. The criteria given in the guidance to assess corrosion risk are based on the chloride content by weight of cement.

Corrosion damage

When corrosion occurs, iron is converted to an iron oxide. This results in a loss of section to the reinforcement. In the case of chloride contamination this results in pitting corrosion which can result in localised but severe section loss.

Whilst the reinforcement suffers from section loss, the resultant iron oxide product has a considerably greater volume, up to nine times the original bar volume. As a result, bursting stresses can result in the cover concrete, which results in delamination of the cover concrete from the reinforcement, cracking of the concrete, and ultimately spalling of concrete sections.

Once cracking and delamination occur the reinforcement has no remaining protection from the concrete and the rate of corrosion can rapidly accelerate.

Testing reinforced concrete for corrosion

It is recommended that concrete survey and investigation work, and in particular the interpretation of data to determine corrosion risk, is undertaken by suitably experienced and qualified personnel.

NDT inspections for corrosion damage

Visual inspection

As noted above, once corrosion of steel in concrete is advanced visual defects will become manifest. Visual inspections should identify any locations of spalling or cracking. Where corrosion of main bars has taken place then vertical cracks in the corners of column shafts may be present. Corrosion of link bars may exhibit more localised cracks or spalls at the corners of the bars.

Delamination survey

Prior to the concrete spalling it will become delaminated. Delamination can be identified using a hammer tap survey. Delamination surveying should be undertaken using a ball-pein hammer. Hammer sounding should be undertaken

3 BRE Digest 444: Part 2 'Corrosion of steel in concrete – Investigation and assessment', 2000.

at intervals of nominally 100mm along the elements being surveyed, at nominally 50mm along their edges/corners from the edge/corner, and at nominally 100mm repeats across the element. Where concrete has delaminated, the sound will 'ring hollow' compared with undamaged concrete. In some cases dragging the hammer over the concrete surface, rather than tapping can better identify the delaminated areas.

On hollow concrete sections the change in tone will be less pronounced but will be evident to the trained ear.

Further testing to identify the risk of corrosion

The tests mentioned previously identify areas where corrosion has already taken place, to the extent that the cover concrete has deteriorated. Once deterioration to this extent has taken place then section loss to the reinforcement will also have occurred. The following tests can be used to determine if there is a risk of corrosion occurring in the future as a result of carbonation or chlorides, or the likelihood that corrosion is presently active.

It should be noted that each of the following tests requires some physical intervention to the column. In the determining the requirements for a condition survey, it should be established if samples and tests can be undertaken on in-situ columns or if a representative sample of columns should be removed from a locale to enable more rigorous laboratory investigation. In the latter case the exposure condition of the removed columns should be suitably representative, in particular if there is a concern regarding exposure to chlorides.

It may also be necessary to determine the depth of cover to the reinforcement in order to interpret the findings. This can be either determined by break-out of the cover concrete, or non-destructively by the use of a cover-meter.

Determining the depth of carbonation

The depth of carbonation is determined by spraying a universal indicator, comprising a solution of phenolphthalein, on to a freshly fractured piece of concrete. A pink/purple colour shows the concrete is still alkaline (pH >9.5), with the carbonated area colourless. A standard exists for the undertaking this test.[4]

If the age of the column under test is known, ie the depth of carbonation is measured after a known period of exposure to the atmosphere, then the time for the carbonation to reach the depth of the reinforcing steel can be estimated.

Determining the extent of chloride contamination

In order to determine the chloride content of concrete it is necessary to remove samples of the concrete and have them laboratory analysed.

The samples may take the form of:
- Dust samples – these are usually collected by the use of percussive drilling into the concrete to provide the dust. At each sample location two holes should be drilled and the drilled concrete dust retained in sample bags. (Samples from two adjacent drill holes are combined to remove errors from, for example, larger aggregate pieces)
- Lump samples (for example, concrete that has spalled or delaminated from a structure;
- Core samples.

4 BS EN 14630:2006 'Products and systems for the protection and repair of concrete structures. Test methods. Determination of carbonation depth in hardened concrete by the phenolphthalein method'

A standard exists for the sampling and determination of chloride content.[5]

In order to determine if chloride is from an external source, and to determine the depth of contamination, it is usual to sample at several incremental depth ranges. For example for columns 5–15mm, 15–25mm, 25–35mm and 35–45mm may be suitable, but this may vary according to the column type and depth of cover. The outer 5mm is usually discarded as non-representative.

The samples should then be submitted to a suitably accredited laboratory for chemical analysis. The laboratory will present the chloride data by weight of sample. In order to assess the data against the criteria set by Building Research Establishment (BRE) it is necessary to determine the chloride content within concrete as a percentage by weight of cement. Often the cement content of concrete is estimated to be 14% by mass of concrete, but for a widespread assessment of columns of similar age, type and manufacturer, laboratory testing to determine the cement content is also recommended.

The test data can be assessed to determine the depth of chloride contamination, and, if contamination has reached the depth of reinforcement, to assess the risk that corrosion may occur.

Measurement of corrosion potential (half-cell testing)

Steel/concrete/portable reference electrode surveys, commonly referred to as half-cell surveys, measure the corrosion potential of reinforcement in concrete. This information provides some indication as to whether the reinforcement is in a passive state or corroding. Best-practice guidance on the field implementation of this, and on other investigations is provided by The Concrete Society.[6]

In order for the test to be effective it is necessary for all reinforcement within a column to be electrically continuous, so this should be checked for each column type under investigation. A connection is made to the reinforcement and a calibrated reference electrode placed on the concrete surface. For columns, a measurement interval of 250mm on a minimum of two faces would provide suitable data for later assessment.

The interpretation of the results, in terms of determining the risk of corrosion, should be undertaken by a corrosion specialist, and the data assessed together with other data sets on carbonation and chlorides. The magnitude of potential gradients may be probably as important in data analysis as the level of measured potential.

5 BS EN 14629:2007 'Products and systems for the protection and repair of concrete structures. Test methods. Determination of chloride content in hardened concrete'

6 Concrete Society Technical Report 60, 'Electrochemical tests for reinforcement corrosion'; 2004.

Appendix 15
Attachments

Lighting columns are all too readily seen as a convenient supports for other attachments such as CCTV, banners, hanging baskets, smart-city technologies, and signage for which they may not have been designed. This can potentially lead to the structure and associated foundation becoming overloaded, leading to failure.

In order to ensure the safety of the public, the integrity of the lighting column to which the attachments are to be fixed must be understood. This will then dictate the best choice of assembly with regard to acceptable loads.

It is important that safe fixings are achieved, and that will require an understanding of the structure to which the attachments are to be fixed, the choice of brackets, details of the assembly and the loadings which will be applied, to name but three considerations. It is therefore important that a structural assessment is undertaken within the process.

The majority of lighting columns within the UK are of tubular steel construction conforming to British Standard BS EN 40-3-1 *Lighting columns; Design and verification* or earlier standards, and unless specifically designed to do so are unlikely to have been designed to support anything more than the bracket, luminaire(s) and perhaps a sign of up to $0.3m^2$.

When a column is designed the attachments and luminaires that may be attached to it at some time in its life must be specified. The specified attachments and luminaires are then considered to be approved for use on the column. If, in the future, there is a need to use additional or larger attachments, larger or heavier

Pictures A15.1 and A15.2: Potentially overloaded columns, one of which has started to lean luminaires, or fix a height extension, it is essential that structural considerations are made to ensure that the column can withstand any additional induced loading. Appendix 2 of ILP PLG06[1] contains an 'additional load flow chart' that should be considered if any additional attachments are to be considered. It is possible that the additional loading may cause the column to collapse because the wall thickness is too low, whether or not there is any corrosion or fatigue cracking. Therefore, any column with an unapproved attachment, luminaire or height extension for which the column was not designed must be given a 'First' priority for action.

Columns with unapproved attachments, luminaires and height extensions do not require urgent action if it can be shown that the design of the column is sufficient to satisfy the service criteria. For this, the original thickness of the column sections (base and shaft), away from areas where there may be corrosion or cracking, can be measured using, say, ultrasonic testing. The bending and torsional strength of the critical sections, and the wind and dead loads can be calculated in order to determine whether the ultimate limit state requirements are satisfied. If the requirements are satisfied, the attachments, luminaires and height extensions assumed in the analysis become approved and the column may be prioritised on the basis of the risk assessment model. However, other details of the design of the column such as joints and connections may also need to be considered before approval.

It is recommended that at every visit to the column, either scheduled maintenance, fault repair or survey, an inspection is undertaken in line with the processes previously discussed. In addition, it is recommended that details of all attachments are captured, photographed and recorded in the asset management system.

Type of fixing

The fixing for an attachment will need to be of a construction that does not impair the integrity of the protective system/painting system of the lighting column. The brackets should have a suitable neoprene rubber gasket or insert fitted between the lighting column and the bracket to ensure this integrity is maintained and any galvanic reaction minimised. All support brackets, clips and associated attachments need to be manufactured from a non-corrosive material such as galvanised steel or stainless steel being of sufficient size and strength to support the attachment(s) when subject to wind pressure and exposure class.

If anti-rotational fixings are used then suitable protection should be used to ensure that these do not adversely affect the column's protective finish.

No undue stress, bending or bowing should be imparted to any lighting column, which must remain vertical at all times. The loading should be kept equally balanced at all times to mitigate this possibility.

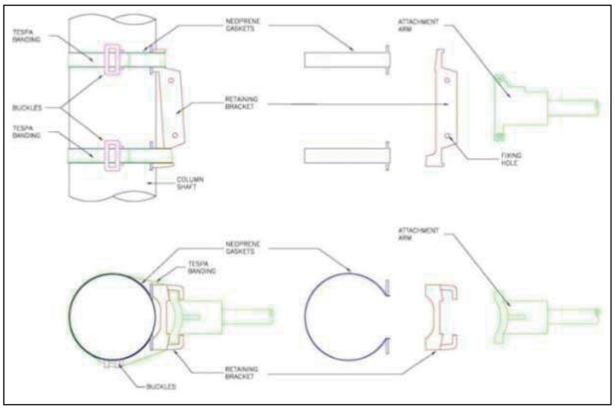


Figure A15.1: Suitable strapping system for use with attachments

1 ILP PLG06 *Guidance on installation and maintenance of seasonal decorations and lighting column attachments*

Appendix 16
Guidance for the assessment of the condition of lighting columns

General

This appendix gives guidance on the visual inspection and assessment of the structural condition of lighting columns. Visual inspection is intended to give the asset owner an indication of the structural condition of the luminaire supports, the results of which may then be used to determine an overall asset condition leading to decisions on intervention strategies.

This document gives guidance on development of a condition index, reporting on the condition of minor structures at a point in time. Whilst steel columns form the vast majority of the column stock within the UK, inspectors may use the public lighting inspection pro forma in Appendix 3 for all columns material types as reporting using 'severity' and 'extent' as the consistent approach.

At the time of any maintenance or operational visit, equipment evaluated as being in poor condition should be highlighted and reported for urgent review. This will require further investigation and should prevent the planned maintenance work being undertaken until suitable investigation has taken place.

This section should be read in conjunction with Section 7.

The following examples identify and recommend the main areas of a minor structure that require inspection, so that an overall condition assessment can be made by the asset owner. It is important that the operative undertaking the inspections is competent and that inspections and reports provide consistency in assessment and recording.

To ensure this, it is recommended that training and an inspection manual be produced and given to inspecting operatives.

Each asset owner should determine the level of inspection based upon the age, condition, available funding and historical or factual evidence of their stock. Where possible, the results of such inspections should be stored in some form of computer database.

Inspections should be carried out on each occasion that the luminaire support is visited for routine or non-routine maintenance, but at least every three years.

The condition of each element of the lighting support is assessed visually and recorded for processing in accordance with the methodology described in Section 10 of this document. This methodology and calculation will result in one of the following condition status being determined:

- Excellent;
- Good;
- Fair;
- Poor.

Attention should be paid to areas where damage to the protective coating may accelerate corrosion and reparation work. Interventions should be considered for these areas to reduce the risk of increased corrosion.

The following diagrams and examples identify by equipment type the levels of inspection recommended. The results of the inspections should be reported.

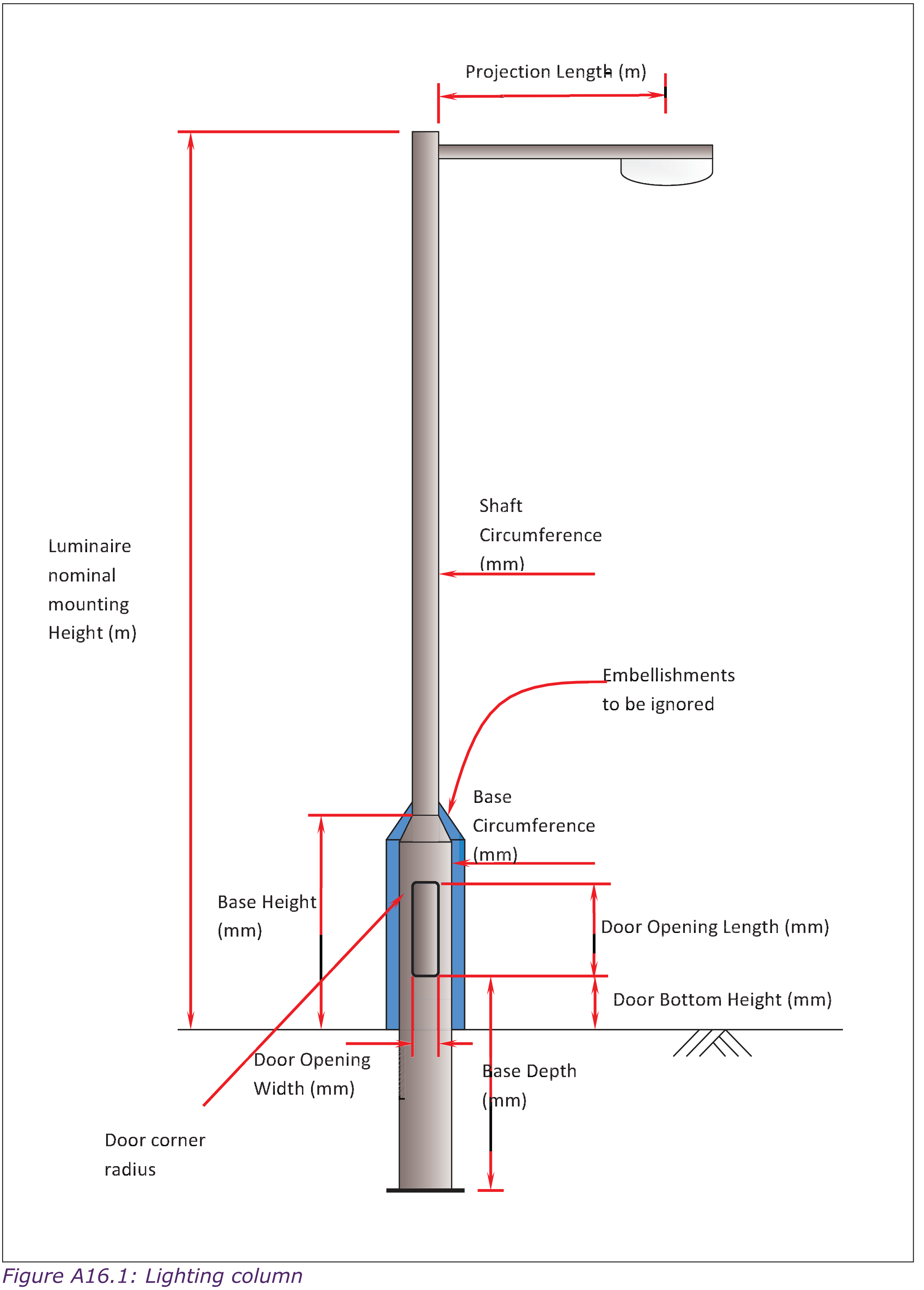


Figure A16.1: Lighting column

Metal columns

Flange plate

The photographs below show examples of the flange plates of metal lighting columns at various stages of deterioration and relate to the extent. The condition of flange plates should be assessed and recorded.

Extent				
A	B	C	D	E
				
No indication of corrosion on root or bolts	Minor internal corrosion and signs of bolt corrosion	Moderate corrosion around flange plate and bolts	Layers of rust within parental metal or bolts corroded	Support leaning or structurally damaged

Base compartment

The photographs below show examples of the base compartment of metal lighting columns at various stages of deterioration and relate to the extent. The condition of base compartments should be assessed and recorded.

Extent				
A	B	C	D	E
				
No indication of corrosion on base and protective root coating intact	Minor internal corrosion on base and protective root coating fading	Moderate internal corrosion on base and protective root coating fading	Layers of rust within parental metal on base; no root protective coating or poor foundation	Layers of rust within parental metal, support leaning or structurally damaged.

Door opening

The photographs below show examples of door openings of metal lighting columns at various stages of deterioration and relate to the extent. The condition of door openings should be assessed and recorded.

Extent				
A	B	C	D	E
				
Sound aperture	Signs of minor corrosion	Moderate corrosion/ indentation	Aperture showing signs of stress; splits	Aperture split

Base compartment shoulder

The photographs below show examples of the base compartment shoulder of metal lighting columns at various stages of deterioration and relate to the extent. The condition of base compartment shoulders should be assessed and recorded.

Extent				
A	B	C	D	E
				
Sound weld; no signs of corrosion or stress	Paint loss but sound metal work	Signs of moderate corrosion	Signs of major corrosion	Welds showing signs of cracks and corrosion/ distortion.

Base compartment internal

The photographs below show examples of the base compartment internal area of metal lighting columns at various stages of deterioration and relate to the extent. The condition of base compartment internals should be assessed and recorded.

Extent				
A	B	C	D	E
No indication of corrosion	Minor internal corrosion		Layers of rust within parental metal/below ground level	Layers of rust within parental metal thin walls

Shaft

The photographs below show examples of the shaft of metal lighting columns at various stages of deterioration and relate to the extent. The condition of shafts should be assessed and recorded.

Extent				
A	B	C	D	E
				
Sound external; no rust indication or paint in good condition	External rust spots or paint beginning to flake	Moderate external rusting (25%)	50% external rust cover or paint flaking	60% external rust cover or structurally damaged.

Bracket/shaft interface

The photographs below show examples of the bracket and shaft interface of metal lighting columns at various stages of deterioration and relate to the extent. The condition of bracket/shaft interfaces should be assessed and recorded.

Extent				
A	B	C	D	E
				
Sound and sealed spigot attachment	Poor seating or loose shaft joint	Missing grub screws or key way; bracket not aligned	Extensive cracking around grub screws or signs of corrosion from missing seal	Bracket failure

Fold-down column base hinge

The photographs below show examples of the base hinge interface of metal lighting columns at various stages of deterioration and relate to the extent. The condition of fold-down column base hinges should be assessed and recorded.

Extent				
A	B	C	D	E
				
Sound hinge point	Signs of minor corrosion around the hinge point	Layers of rust at key fold down unit anchorage point	Layers of rust within parental metal; corroded attachment point	Extensive cracking/ corrosion around hinge point

Concrete columns

Base compartment

The photographs below show examples of the base compartment of concrete lighting columns at various stages of deterioration and relate to the extent. The condition of base compartments should be assessed and recorded.

Extent				
A	B	C	D	E
				
No indication of damage to root base*	Minor damage or plugs missing	Moderate damage, cracking	Signs of stress splits	Support leaning, overgrown, concrete cracking

* If plugs are found to be secure and in position, wire brush around them and apply a layer of suitable sealant.

Door opening

The photographs below show examples of the door opening of concrete lighting columns at various stages of deterioration and relate to the extent. The condition of door openings should be assessed and recorded.

Extent				
A	B	C	D	E
				
Sound aperture	Signs of minor damage	Moderate damage, cracking	Aperture showing signs of stress splits	Aperture split

Shaft

The photographs below show examples of the shaft of concrete lighting columns at various stages of deterioration and relate to the extent. The condition of shafts should be assessed and recorded.

Extent				
A	B	C	D	E
				
Sound external, no damage	External damage, slight cracking	Signs of stress split	Signs of cracking and concrete spalling	Cracks, splits, reinforcing bars showing, areas of concrete missing

Bracket/shaft interface

The photographs below show examples of the bracket/shaft interface and bracket arm of concrete lighting columns at various stages of deterioration and relate to the extent. The condition of bracket/shaft interfaces should be assessed and recorded.

Extent				
A	B	C	D	E
				
Sound external, no damage	External damage, poor fitting bracket sleeve	Signs of stress splits, poor connection	Signs of cracking and concrete spalling	Cracks, splits, spalling, reinforcing bars showing, areas of concrete missing

Wall mounting

The photographs below show examples of the base compartment of concrete lighting columns at various stages of deterioration and relate to the extent. The condition of wall mountings should be assessed and recorded.

Extent				
A	B	C	D	E
				
Sound anchorages in separate bricks clear of edges	Uncertain anchorages requiring investigation	Lack of anchorages; insufficient attachment	Anchorage in mortar joints; close to edges	Anchorage close to edges

Other indications

The photographs below show examples of the other indicators that should be noted and reported at the time of inspection, maintenance or operation visit. These require further investigation and are perhaps aspects that should prevent any planned work being undertaken until suitable investigation has taken place.

			
Column base installed too high, embellishment kit proud of surface	Void around base of column; ground movement	Leaning column due to ground movement	Damaged paint surface, dog urine

Detailed structural visual inspection forms

The forms shown in Appendix 3 are for example only and the criteria should be amended to suit each individual asset owner/operator's lighting stock.

These forms are examples of structural visual inspection forms capturing data for assets of material types other than steel. Use of these forms will allow the competent practitioner to operate the strategies previously detailed within this report.

It is recommended that the forms are integrated into the inspection manual to provide the inspecting operative with information to achieve consistent assessment.

It is recommended that the condition assessment information is collected electronically as this will help the experienced practitioner to process and evaluate the outcomes, whilst retaining a historical electronic record.

Appendix 17
Wall brackets

Introduction

To understand the visual assessment and testing of wall brackets which support luminaires, CCTV or other equipment, it is important to understand how their anchorages should be designed and installed. This section draws on the key aspects to be considered when looking at the installation, and when checking the positioning and condition, of the anchorages for building-mounted brackets and equipment, and hence what an inspector/tester should look for and consider on site. It draws on national guidance such as that produced the Construction Fixings Association (CFA) in its guidance note *Anchorage Systems for Seasonal Decorations*.

The CFA document gives valuable information about the types of anchorage/bolt fixings available, how they should be installed, and the method of testing them. The guidance note applies specifically to catenary-based installations but is also relevant to building-mounted brackets.

It is important that safe fixings/anchorages for the brackets are maintained, and that will require an understanding of the structure to which the brackets are fixed, the choice of anchorage, bracket details, and what the bracket is to support.

Suitability of the structure

The suitability of the building structure to sustain the loads transferred from the bracket should have been established and

Pictures A17.1 and A17.2: Typical heritage wall bracket; Floodlight mounted on cladded building recorded within the project documentation prior to any design and installation work. Where this information is not available, an assessment of the structure must be undertaken by a competent person, for example a structural engineer.

As a rule:

- Concrete elements which are part of a load-bearing structure will be considered suitable, but care should have been taken not to drill into any reinforcement elements;
- Masonry structures may be suitable if they are load bearing and of solid, rather than cavity, construction, and composed of strong masonry units with sound mortar joints;
- Cavity brickwork constructions, and especially cladding panels, may not be capable of transferring the tensile loads involved and require careful consideration.

Brackets are normally attached by anchorages such that the load is directed into the anchorage at differing angles depending upon the anchorage location and bracket details. Where brackets have top and bottom anchorages then the lower ones will be subjected to more of a downwards force whereas the high ones will also have a slight forwards force applied. This effectively means that the correct choice of anchor and installation is critical. Brackets that just have two horizontal anchorages are subject to forces both forward and downwards.

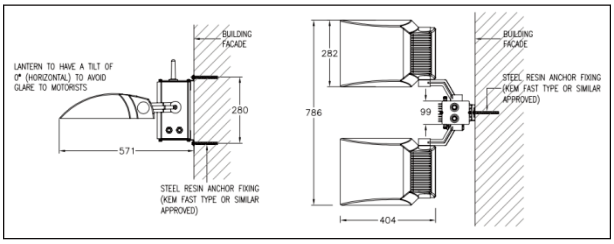


Figure A17.1: Typical luminaire and bracket fixing arrangement

Type of anchor

There are various forms of anchorage that may have been used and these are briefly summarised as follows. The inspector should consider if the right system for the structure has been used.

Resin studs/anchors

Resin anchors are suitable for use in concrete and hard masonry including brickwork, stonework and concrete blockwork as they do not stress the base material as would be the case with expansion anchors.

A nut and washer is attached to a threaded rod set into resin in concrete – in the case of solid brickwork resin sleeves may be needed – as shown in Figure A17.2. Care should have been taken to ensure adequate thread engagement is achieved within the nut and adequate stud embedment within the base material.

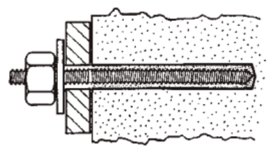


Figure A17.2: Installed resin stud

Brackets

It is unlikely that a single anchor will be used for a bracket-and-luminaire arrangement as it will not support the loads. Brackets will come in various shapes and sizes to suit the style of equipment being used, be it heritage or modern, and for these M12 resin stud anchors should have been used, the number depending upon the bracket style, which in part will be dictated by the building structure and the loads being attached.

For masonry walls, the dimensions of the plate should have been chosen such that the spacing between anchors matches the

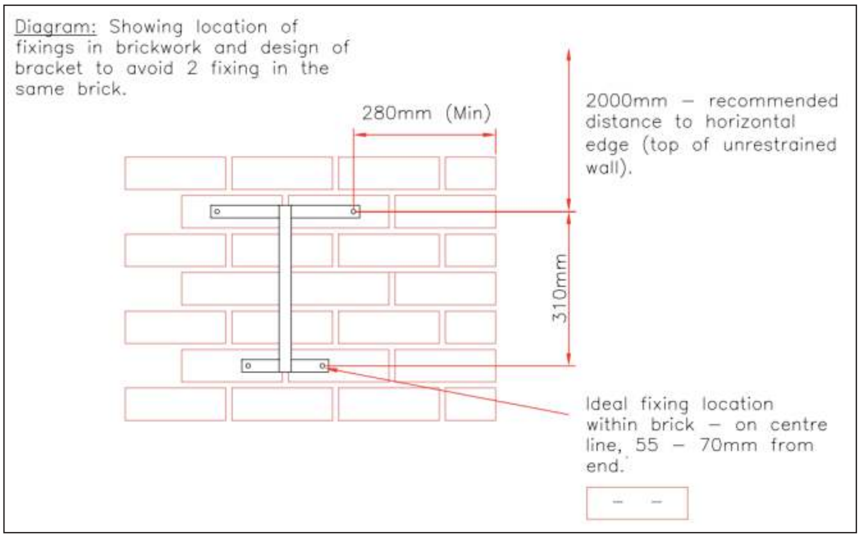


Figure A17.3: Masonry bracket and fixing considerations

brick course, see Figure A17.3. The bracket width should have been considered to allow standard test equipment to be positioned over the bracket or the anchor tested before the bracket is attached.

Embedment depths

Embedment depths in concrete are straightforward and should follow the manufacturer's recommendations.

Embedment depths for anchors in solid brickwork, as shown in Figure A17.4, depend on the structure. To gain maximum strength from anchors set into nine-inch solid brickwork it is important to achieve optimum embedment into the rearmost leaf which then benefits from load transfer via the front leaf, see (A) below. Maximum hole depth for anchoring into the rear brick of nine-inch structures should be 170mm. Any deeper risks breaking the back of the brick out under the drilling action.

Anchor positioning

The correct location of anchors is important to ensure that the structure can support the loading. The recommendations of the anchor manufacturer should be followed regarding close edge distances and spacing between anchors used in pairs or groups.

Anchor positioning in brickwork

Anchors which are used to fit brackets to the structure should be located as indicated in Figure A17.4, at least one full masonry unit from a vertical edge; in brickwork this means at least 280mm. This distance may need to be increased substantially for lateral or shear loads; the distance may depend on the magnitude of the load and condition of the masonry.

A minimum edge distance of at least 2m should be allowed from a horizontal edge in brickwork for loads in any direction.

It is noted that this requirement may cause positioning difficulties on structures in order to achieve the desired lighting performance. Where these requirements cannot be met, a structural engineer

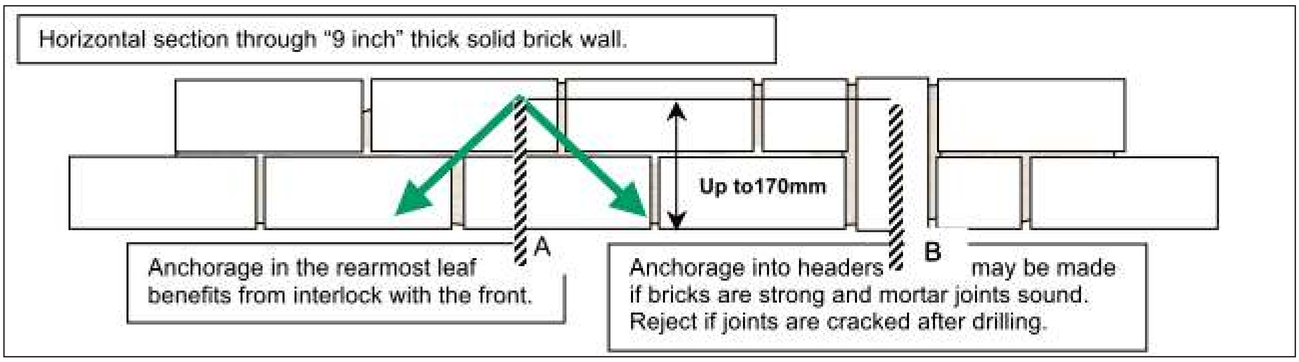


Figure A17.4: Embedment depth

should have considered the design and, if agreed that the recommended clearances could be encroached upon, the full details should have been recorded in the technical file. If this evidence does not exist then the fixing shall be referred to a structural engineer for a suitability review.

Centre spacing between anchors should have been chosen to avoid setting two anchors in the same brick.

When drilling into brickwork the anchor should ideally have been located in the solid portion of the brick rather than in to the mortar joint.

If anchors may not have been fixed into the bricks themselves, for example as a result of a conservation order, then the following approach – as shown in Figure A17.5 – may have been sanctioned by the responsible engineer if approved by the manufacturer.

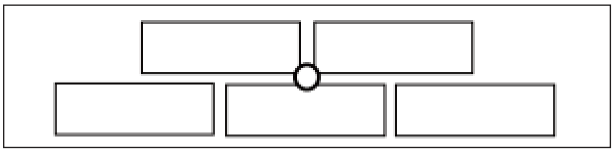


Figure A17.5: Anchorage location within a mortar joint

- Choose an anchor with a diameter significantly larger than the width of the mortar joints, for example >14mm in a 10mm joint;
- Fix into the base of the junction between bed and perpendicular joints;
- Proof tests must be carried out, on each individual anchor.

Proof tests

Test procedures

Test procedures for either preliminary or proof tests should be carried out in accordance with the CFA guidance note. Prior to carrying out any tests on anchors, the tester should examine carefully the structure surrounding the anchor position and note any conditions giving rise to concern that the anchorage may not sustain the required load.

Such conditions will include deterioration of masonry units or mortar joints and damage such as cracks across masonry units or in mortar joints. Where a tester is concerned that the structure may not be sufficiently strong, anchorages have been located incorrectly etc, then that concern should be included in the test report and it made clear that any test results (even positive ones) do not imply that the structure can take the loads.

Regular inspection

The regular inspection of all brackets and their anchor points is required to ensure that the anchor points remain in serviceable condition, can carry the required loads, and have not suffered any damage or deterioration likely to affect this capability.

When the bracket is inspected the anchorage should also be checked.

The regular inspection should be carried out in accordance with any recommendations made by the original manufacturer and/or installer and as stated in any technical file.

Regular inspection should be carried out by competent persons and should include the following aspects:

- Every visit:
  - Nuts on stud anchors to be checked for nominal tightness (ie they are not loose and cannot be undone by fingers);
  - The bracket/luminaire is firmly attached and not loose in anyway;
  - Visual inspection of componentry for rust;
  - Visual inspection of surrounding substrate to check for damage, including cracks in masonry or mortar joints.

- Every three years.
    - Test loads to be applied, usually the same as the proof test load applied at the time of installation. If the load test equipment cannot be positioned due to bracket design then a weight test should be used and the bracket checked to ensure no movement of the bracket or anchorages. The load attached to the bracket end should be between 1.25 and 1.5 times the applied load.

Technical file

Installers of anchors for supporting brackets under the Construction (Design and Management) Regulations must pass to the client a technical file that will enable future examiners of the anchorages to fully inspect and load test them

Details which should be covered include[1]:

- All loading calculations and the derivation of loads applied to the fixings for each building;
- Details of the anchors that have been installed (type and make). In the case of resin anchors the stud or socket diameter used, make and type of resin, hole diameter and embedment depth, installation torque applied;
- The test loads applied at the time of installation;
- Any restrictions regarding life expectancy.

1 See Construction Fixings Association (CFA) Guidance Note, *Anchorage Systems for Seasonal Decorations*

Appendix 18
Specialist supports

Catenary systems

To understand the visual assessment and testing of anchorages used within catenary systems supporting luminaires, festive decorations or other equipment it is important to understand how such anchorages should be designed and installed.

The details of this are covered within ILP Professional Lighting Guide PLG06 *Guidance on Installation and Maintenance of Seasonal Decorations and Lighting Column Attachments*. Additional supporting information is provided in national guidance such as that produced the Construction Fixings Association (CFA) in its *Anchorage Systems for Seasonal Decorations*.

The CFA document gives valuable information about the types of anchorage/bolt fixings available, how they should be installed, and the method of testing them. The guidance note applies specifically to catenary-based installations.

It is important that safe fixings/anchorages are maintained, and that will require an understanding of the structure to which the brackets are fixed, the choice of anchorage, bracket details and what the bracket is to support.

Suitability of the structure

The suitability of the building structure to sustain the loads transferred from the catenary wires must be established and recorded within the project documentation prior to any design and installation work. It has been the case that failure of the structure is normally due to poor anchorage location rather than the anchorages or catenary system.

This assessment must be undertaken by a competent person, for example a structural engineer.

Anchor positioning

The recommendations of the anchor manufacturer should be followed regarding close edge distances and spacing between anchors used in pairs or groups.

For anchor positioning in brickwork see Figure A18.1

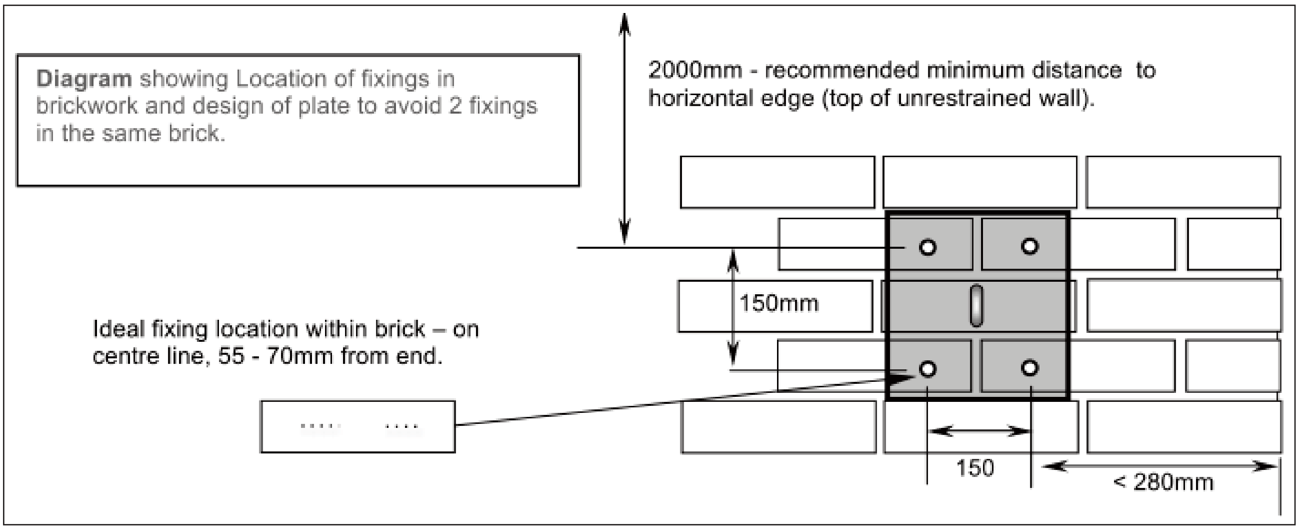
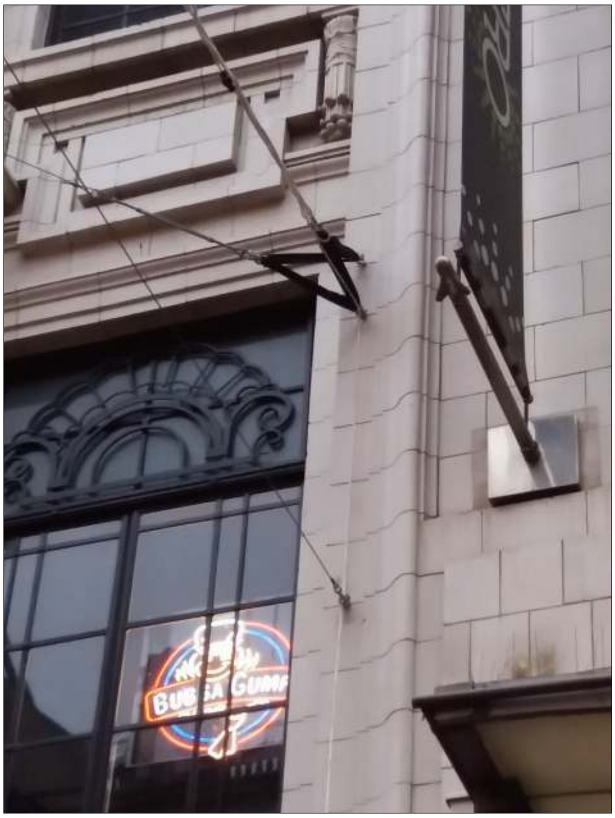


Figure A18.1: Anchor positioning in brickwork



Pictures A18.1 and A18.2: Poor anchorage choice in mortar joint; Good anchorage plate

Anchors which are used to support tensile loads should be located at least one full masonry unit from a vertical edge; in brickwork this means at least 280mm. This distance may need to be increased substantially for lateral or shear loads; the distance may depend on the magnitude of the load and condition of the masonry.

A minimum edge distance of at least 2m should be allowed from a horizontal edge in brickwork for loads in any direction.

Centre spacing between anchors should be chosen to avoid setting two anchors in the same brick.

Where these requirements cannot be met, a structural engineer should have considered the design and, if agreed that the recommended clearances could be encroached upon, the full details should have been recorded in the technical file. If this evidence does not exist then the fixing should be referred to a structural engineer for a suitability review.

When located on brickwork the anchor should ideally be located in the solid portion of the brick rather than into the mortar joint. If the brickwork has been rendered the location of the centres of the courses of bricks should be identified by removing the render or by test drillings.

If anchors may not have been fixed into the bricks themselves, for example as a result of a conservation order, then the following approach – as shown in Figure A18.2 – may have been sanctioned by the responsible engineer if approved by the manufacturer.

- Choose an anchor with a diameter significantly larger than the width of the mortar joints, for example >14mm in a 10mm joint;

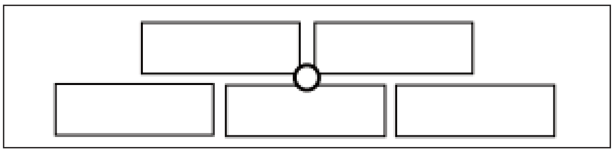


Figure A18.2: Anchorage location within a mortar joint

- Fix into the base of the junction between bed and perpendicular joints;
- Proof tests must be carried out, on each individual anchor.

Regular inspection

The regular inspection of all anchor points is required to ensure that the anchor points remain in serviceable condition, are capable of sustaining the required loads, and have not suffered any damage or deterioration likely to affect this capability.

The frequency of inspections will depend on factors such as the use to which anchor points have been put.

The regular inspection should be carried out to a specification determined by the original Installer and stated in the technical file.