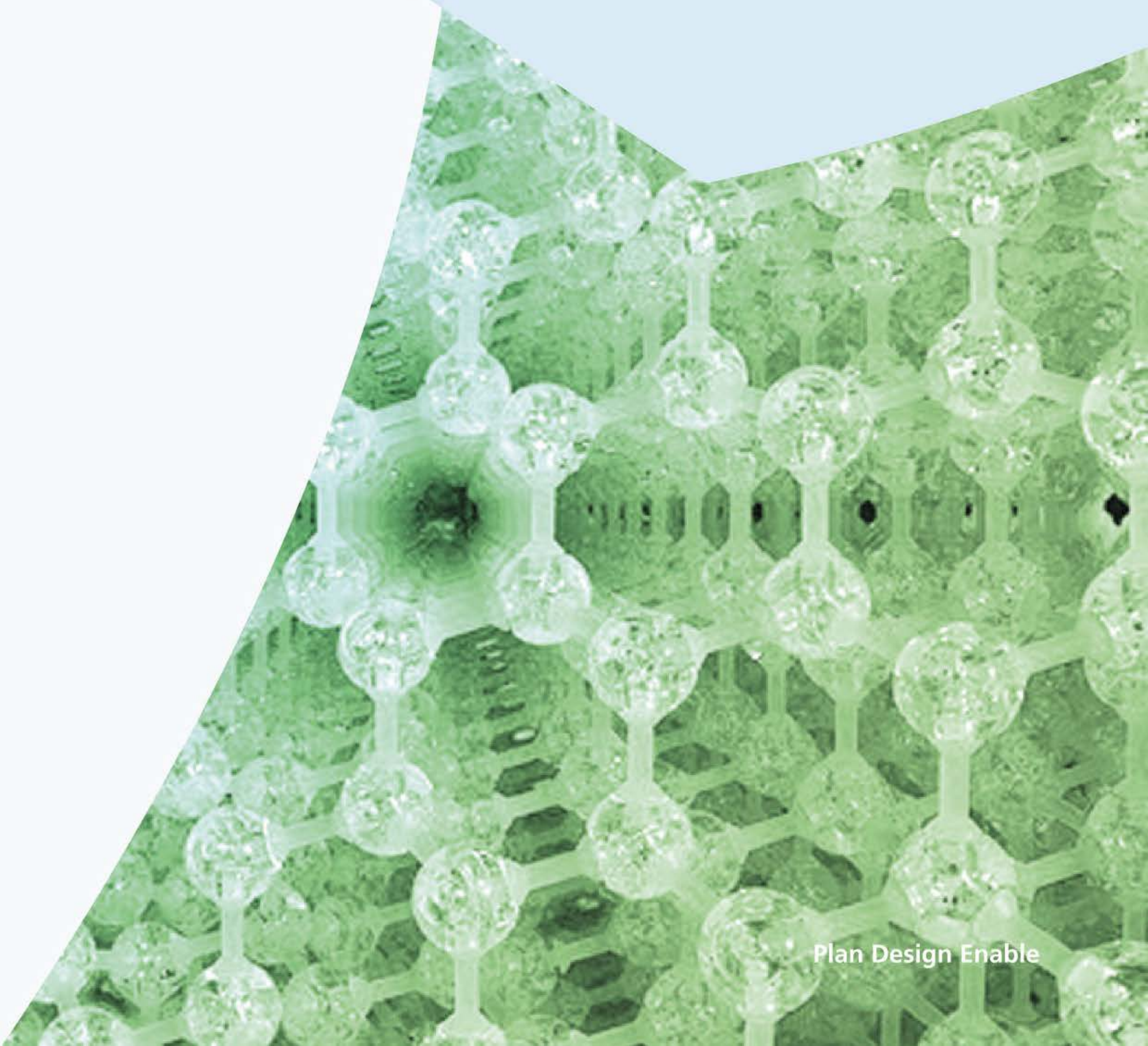
ATKINS

06

Technical Journal

Papers 081 - 098



Plan Design Enable

FOREWORD

Welcome to the sixth edition of the Atkins Technical Journal which again showcases the impressive breadth and depth of the technical solutions we offer our clients. This edition is strengthened further by the inclusion of papers for the first time from our colleagues in Atkins North America.

Sustainability features heavily throughout this edition, confirming that it is as the heart of all we do whether it be planning, designing or enabling infrastructure solutions for our clients. Some salient examples include:

Planning - our work to produce a BIM-style tool to optimise urban masterplanning for sustainability and decarbonisation; the production of holistic energy plans for the UAE; our advice to the Scottish Government to optimise carbon reductions in its transport policy; and our advice on water resource management plans to protect and restore the hydrological system in Winter Haven, Florida to meet increasing population demand.

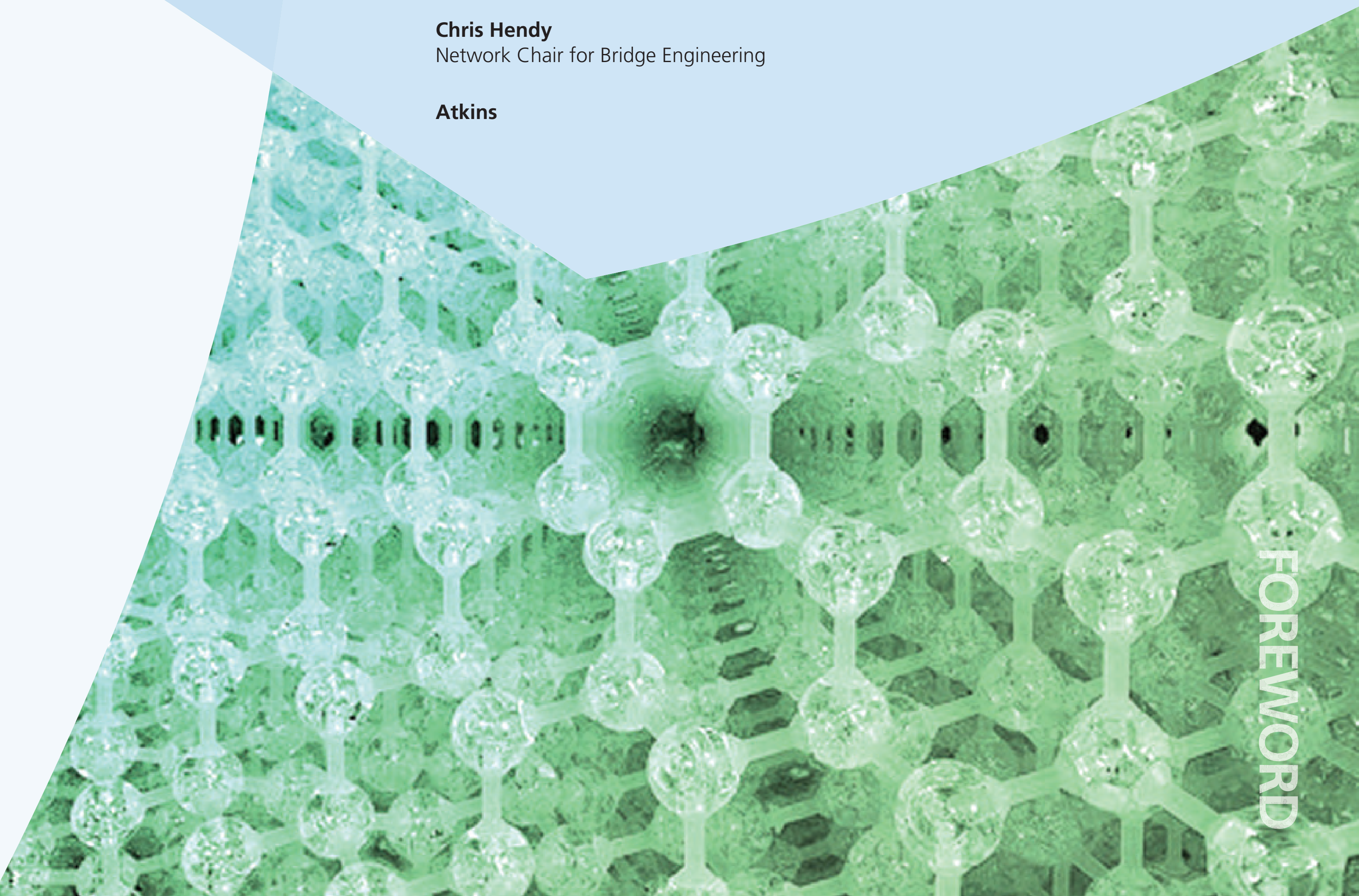
Designing - our production of steel bridge design charts to optimise design for our clients and industry as a whole; our design of the architecturally impressive Almas Tower, Dubai, with equally impressive design efficiency; and our publications on stay cable vibration and fatigue to prevent the need for future expensive remedials on large bridges around the world, as continues to occur at present.

Enabling - our use of GIS and information management to drive efficiencies and get it "right first time" on projects like Crossrail; our expert advice on translocating wildlife habitats to mitigate the environmental effects of construction; our creation of new wetland habitats in the London 2012 Olympic Park; and our evaluation and deployment of Open Road Tolling to reduce driver accident statistics and provide safer highways.

The above summary merely scratches the surface of what we do and what is contained in Technical Journal 6. I hope you enjoy the selection of technical papers included in this edition.

Chris Hendy
Network Chair for Bridge Engineering

Atkins



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Corrosion mitigation of chloride-contaminated reinforced concrete structures: A state-of-the-art review

Sunil C Das

Principal Engineer

Amey

Homayoon Sadeghi Pouya

Materials Engineer

Highways & Transportation
Atkins

Eshmaiel Ganjian

Reader in Civil Engineering Materials

Coventry University

Although corrosion is not a new problem, awareness of it associated with civil engineering structures, particularly reinforced/pre-stressed concrete highway bridges, multi-storey car-parks and buildings etc. it is relatively new. Corrosion is insidious in nature. Steel corrosion in concrete is only apparent when it is quite advanced. It manifests itself progressively in the form of 'rust' stains, cracking, delamination and finally spalling with exposed and corroding steel reinforcement. Proper application of available science and technology can save a large amount of waste due to corrosion. Over the last two decades a number of corrosion mitigation techniques have been developed. Some are more successful than others. Cathodic protection is the only proven technique to stop corrosion of steel in chloride contaminated concrete.

Introduction

The problems of concrete deterioration due to corrosion of steel reinforcement and/or pre-stressed/post tensional systems in concrete structures are worldwide, costing nations billions of pounds, equivalent to around 4 to 6% of Gross Domestic Product (GDP). A recent cost of corrosion study[4] estimated the annual cost of corrosion on US highway bridges to be at $8.3 billion overall, with $4.0 billion of that on the capital cost and maintenance of reinforced concrete highway bridge decks and substructures. In the UK the Department of Transport's estimate of salt-induced corrosion damage is a total of £616.5 million on motorway and trunk road bridges in England and Wales. These bridges represent about 10% of the total bridge inventory in the country (Wallbank, 1989). There are no simplistic models to predict the rate of deterioration due to corrosion. However, we have sufficient understanding of the corrosion mechanisms and concrete deterioration processes.

With the development of various Non Destructive Testing (NDT) assessment techniques and the recent advances in protection and rehabilitation methods, a large percentage of these costs could be reduced.

This paper briefly describes the fundamental principles of corrosion of steel reinforcement in concrete. The paper then gives an overview of currently available methods of protecting corrosion damaged structures. Finally, the recent advances in cathodic protection technology for reinforced concrete structures are discussed.

Corrosion mechanism of steel in concrete

The corrosion of steel reinforcement in concrete is an electrochemical process involving two equal, but opposite, reactions. These are anodic, or oxidation reactions

(eg $Fe \rightarrow Fe^{++} + 2e^-$)

and cathodic or reduction reactions.

(eg $O_2 + 2H_2O + 4e^- \rightarrow 4OH^-$)

Concrete has the inherent ability to protect steel against corrosion. This is due to the high alkalinity of concrete, ranging between 12.5 and 13.7, imparted by the chemical constituents of the cement, in particular calcium hydroxide $Ca(OH)_2$. In this alkaline environment, a thin film of oxide or hydroxide such as ferric oxide, Fe_2O_3, is formed on the steel surface rendering the steel 'passive' i.e. the corrosion rate becomes insignificant.

However, this protection mechanism may break down as a result of one or more changes in the concrete's chemistry, the most common and important factors being:

(i) Loss of alkalinity in the concrete

(ii) Penetration of aggressive ions to reinforcement depth

(iii) A combination of both factors.

The main offending ion for the breakdown of passive film on steel reinforcement is chloride in concrete. The chloride acts as a catalyst for oxidation of iron by taking an active part in the reaction. According to Uhlig[15] it oxidises the iron to form the complex ion $FeCl_3^-$ and draws this unstable ion into solution, where it reacts with the available hydroxyl ions to form $Fe(OH)_2$. This releases the Cl^- ions back into solution and consumes hydroxyl ions, as seen in the following reactions:

$$2Fe + 6Cl^- = 2FeCl_3^- + 4e^- \qquad (1)$$

Followed by:

$$FeCl_3^- + 2OH- = Fe(OH)_2 + 3Cl- \qquad (2)$$

The electrons released in oxidation reaction flow through the steel to the cathode surface. This process would result in a concentration of chloride ion and a reduction of the pH at the points of corrosion initiation, probably accounting for the process of pitting corrosion. The lowered pH at these sites contributes to the continual breakdown of the passive oxide film[12].

Alternative reactions for complex formation are:

$$Fe^{2+} + 6Cl^- = FeCl_6 + 4e^- \qquad (3)$$

or

$$Fe^{3+} + 6Cl^- = FeCl_6 + 3e^- \qquad (4)$$

The above reaction removes ferrous (Fe^3+) ions from the cathode area, allowing them to be deposited away from the bar, through the reaction:

$$FeCl_6 + 3e^- + 2OH^- = Fe(OH)_2 + 6Cl^- \qquad (5)$$

This reaction produces rust and releases chloride ion for further reaction with ferrous ions. In engineering situations, the electrochemical reactions of the corrosion process are more complex than described above. Detailed commentary on the corrosion mechanisms is outside the scope of this paper.

Deterioration processes of reinforced concrete structures

There is now sufficiently detailed understanding of corrosion induced concrete deterioration and it is becoming important to predict (if possible) the residual life of the structure/structural elements in order to inform decision making on whether the structures/elements are likely to maintain their structural integrity or serviceability until the end of a specified 'design life'. Such predictions are also useful in analysing the cost-effectiveness of the different repair or rehabilitation strategies.

A brief literature survey has identified a number of models for estimating the service life or residual life of concrete structures/elements with regard to reinforcement corrosion. All these models are constructed around a specific structure, but all the models have a common starting point i.e. the prediction is based on the damaged model first proposed by Tuutti[18], where the service life of a corrosion-damaged concrete structure is described as a two-stage process:

(i) An initiation stage, in which corrosion initiates once sufficient quantities of chlorides reach the reinforcement

(ii) A propagation stage, in which the extent of corrosion damage build-up reaches a 'limit state' i.e. it is the time when the probability of failure becomes unacceptable.

The service life can be expressed, according to Tuutti[18], as:

$$T = t_i + t_{corr} \qquad (6)$$

where:

T = service life

t_i = initiation time

t_{corr} = propagation (corrosion) time.

Many mathematical models have been proposed to estimate the values of t_i and t_{corr}[1,8,7,3]. All models use an equation to determine the t_i, the initiation time period, by solving the equations based on Fick's second law of diffusion; but estimating the values of t_{corr} is not simple.

In reality, the concrete deterioration processes resulting from reinforcement corrosion in chloride contaminated concrete are stochastic in nature. As a result, the equations to predict life (service life or residual life) need to consider variables such as cover thickness, diameter of the reinforcement, chloride content, water-cement ratio, temperature, relative humidity, oxygen concentration etc. Further complications arise due to the difficulties in defining the 'service life' of the structure or structural elements (which depend on the type of structure, element's function within the structure, failure mode and consequence, and maximum acceptable damage). Recently, two research groups National Cooperative Highway Research Program (NCHRP)[16] and the project funded by the European Community under the BRITE/EURAM Programme[4] have produced 'User's Manuals' on predicting the service life/ residual service life of corrosion-damaged concrete structures/ elements. Detailed discussion of various models to predict the residual life of chloride induced corrosion-damaged reinforced concrete structures is outside the scope of this paper. However, understanding of corrosion and concrete deterioration processes has led to development of various techniques for mitigating reinforcement corrosion and concrete rehabilitation. These are briefly described in the following sections.

Corrosion mitigation and concrete repair strategy

Highway and building structures worldwide are deteriorating at an ever-increasing rate. The option for repair of damaged/deteriorating concrete structures depends on the nature of the problem(s). The choice of rehabilitation and repair technique and material would be determined from full understanding of the underlying cause(s) of the problem. With regard to causes; perhaps one major cause of concrete deterioration is corrosion of steel reinforcement in concrete. This could lead to structural weakness due to loss of cross-section of the steel reinforcement or pre-stressing wire[17]. These may be grouped under the following headings:

- Traditional concrete replacement
- Electrochemical repair methods
- Corrosion inhibitors.

All corrosion mitigation methods can be applied to both structures to be built exposed to environments with risk of chloride contamination and existing chloride contaminated structures. The choice of appropriate repair solution will depend on a number of factors. The short, medium, and long-term operational requirements will dictate to a significant degree the maintenance strategy.

Traditional concrete replacement

Traditional 'patch repair' consisting of removal of defective concrete in an attempt to eliminate the cause of the problem is one option. Current practice specifies that the defective concrete is 'broken back to a sound alkaline base', and any steel reinforcement should be fully exposed to approximately 25mm behind the bar over a length greater than its corroding length, and thoroughly cleaned. Two key phrases in the paragraph above should be noted. These are: 'broken back to a sound alkaline base' and 'thoroughly cleaned' in reference to corroding reinforcement. In practice, the above two conditions may not easily be met. Furthermore, a potential problem with patch repair using cementitious materials is to prevent subsequent ingress of pollutants, including chloride ions.

To overcome this problem the patch repaired structure can be coated using various proprietary surface coatings in order to prevent further ingress of aggressive ions from the environment (Zhang and Mailvaganam, 2006).

Further, in order to prevent or retard the ingress of atmospheric pollutants, including chloride ions or oxygen into concrete, the coating system needs to be completely 'pin-hole' free; at the same time the coating must be 'breathable' i.e. must have sufficient permeability to water vapour to avoid water vapour pressure build up behind the coating causing blistering and subsequent failure. In effect, the aim is to control, or at least retard, the rate of further corrosion. Along with the control of corrosion, other strengthening/ rehabilitation methods are used, if so required. These methods consist of providing additional reinforcement, extra external pre-stress, replacement of damaged structural members etc. The details of such methods are based on assessment of strength, and are attended by the bridge engineer[10].

In summary, traditional patch repair is a short-term remedy which can be carried out to delaminated and spalled areas. Conventional patch repair of corroded concrete structures inevitably introduces 'incipient anode' effect. This is due to the different electrochemical behaviour of steel reinforcement in the 'new' concrete repair material and the surrounding 'old' but sound concrete (which may still be contaminated with chloride). The newly patched area (chloride free) becomes the cathode (less negative potential) and the neighbouring areas become the anode (more negative potential) and start to corrode. Conventional patch repair treats only the symptoms not the cause and the incipient anode effect makes this repair a never-ending process.

Electrochemical methods of repair

Electrochemical repair methods such as cathodic protection (CP) or chloride extraction (CE) may also be used to arrest the corrosion process. Both CP and CE require an active electrical circuit to be established which forces the steel reinforcement cage to become cathodic (non-corroding) by providing an external anode (corroding). Cathodic protection can be applied in two ways: (a) by impressed current CP system (ICCP) where CP uses a permanent external anode connected to an electrical power supply e.g. transformer-rectifier and (b) the second approach is termed Sacrificial Anode CP system (SACP) which uses a metal anode (such as zinc) with a higher natural galvanic potential than that of steel to establish the necessary drive potential directly connected to the steel structure to be protected[13].

Chloride extraction is similar to impressed current CP in that an external electricity supply is required to drive the process; however, an anode is supplied as a series of surface mounted panels containing an electrolyte. The drive voltage for CE is very much higher than CP as the aim is to draw the negative chloride ions away from the reinforcement towards the anode and out of the concrete.

Both CP and CE may require some initial minor concrete replacement repairs to those areas which are delaminated, as the current path would be inhibited by the cracked concrete. An additional requirement of the concrete specification for concrete repairs where CP or CE is to be used is that the resistivity of the concrete must be kept low such that an electrical current can be passed through the concrete.

An advantage with electrochemical repair methods is that those areas of concrete contaminated with chloride do not need to be broken out. This can result in significant cost savings which more than offset the cost of the system installation.

The durability of repairs using cathodic protection has been well established, provided the systems are actively monitored[6]. Advances in remote monitoring technology have reduced monitoring costs significantly in recent years. The durability of repairs using CE is less well established. The efficiency of the CE to remove chlorides from around the reinforcement to establish passivity of the reinforcement can vary significantly from structure to structure. CE may be required at intervals during the life of the structure as the remaining chlorides migrate towards the steel reinforcement. The advantage of CE over CP is the short duration of the repair works[6].

Impressed current systems also generate hydroxyl ions around the reinforcement, which raise the alkalinity of the concrete surrounding the reinforcement helping to re-establish passivity of the steel. This can give rise to an increased risk of alkali silica reaction if reactive aggregates are present in the concrete. The higher the impressed current the greater the risk of alkali silica reaction, therefore, CE in particular must be chosen with care.

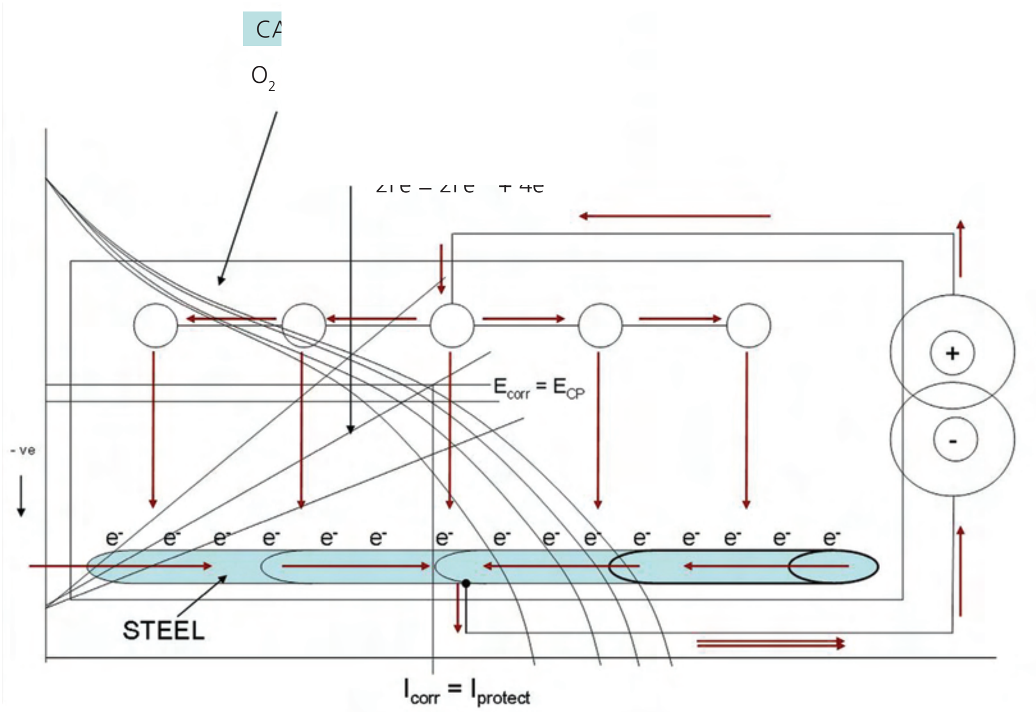

Figure 1 - Circuit diagram for an Impressed Current CP (ICCP) System (with super-imposed polarisation curves)

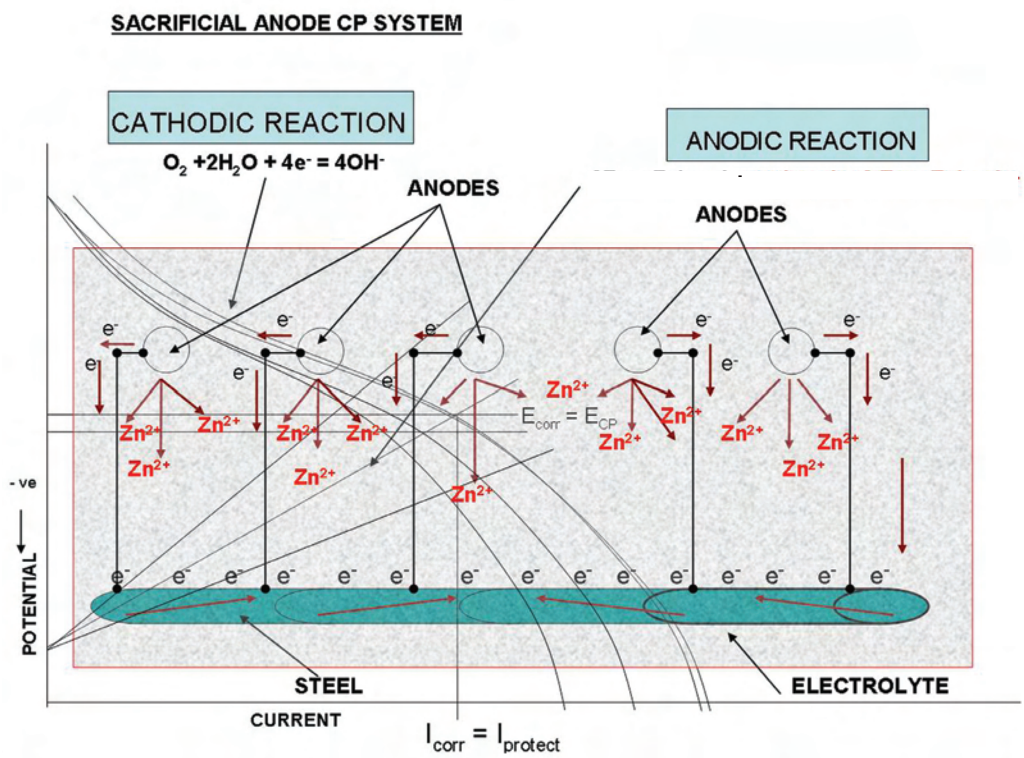

Figure 2 - Circuit diagram for a Sacrificial Anode CP (SACP) System (with super-imposed polarisation curves)

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Table 1 - Impressed current anode types and characteristics[13]

Anode Type	Long Term Current Density per m^2 of anode mA/m^2:	Long Term Current Density per m^2 concrete mA/m^2:	Typical Anode Life Estimate (years)	Suitable for Wet Structures	Suitable for Running Surfaces	Dimensional & Weight Impact/ Installation	Performance Queries	Typical Anode Cost £/m^2
Conductive organic coatings	20	20 Max	10-15	No	No	No Painted	Some unproven products	20-40
Sprayed zinc	20	20 Max	10-15	Possibly	No	No Thermal spray	Consumption rate Health & Safety	60-100
Mixed metal oxide (MMO) coated titanium mesh and grid in cementitious overlay	110-220	15-110 Varying grades	25-120	Yes	Yes	Yes In circa 25mm overlay	Overlay quality control	60-100 including overlay
Discrete Pt/ Ti or MMO Ti anodes, with carbonaceous surround	800 from carbonaceous surround	Circa 10-110 subject to distribution	10-20	Yes, not tidal	Yes	No Placed in pre-drilled holes		40-100
Discrete anodes in cementitious surround. MMO Ti or conductive ceramic	800	Circa 10-110 subject to distribution	20-50	Yes	Yes	No Placed into holes or slots		40-100
Cementitious overlay incorporating nickel plated carbon fibre strands	20	20 Max	10-20	Probably	No	Yes Sprayed, circa 8mm thick	Limited experience	30-60

Furthermore, the benefit of CE is considered to be limited to short to medium-term and would require a number of repeat applications over the design life of the structure.

In recent years an alternative impressed current repair technique termed 'electro-osmosis' (although there is some debate as to whether this is an accurate description of the actual process which takes place) has been trialled on chloride contaminated concrete (Chess, 1998). The process works by drawing moisture away from the rebar thereby increasing the local resistivity and significantly reducing the rate of corrosion. The effectiveness of this technique is very much an unknown at this time, although limited trials have been undertaken by the Highways Agency.

For a longer-term solution for controlling corrosion, application of cathodic protection is considered to be the 'only technique' proven to stop/mitigate on-going corrosion of steel reinforcement. This is particularly the case for chloride-contaminated concrete.

Cathodic protection for reinforced concrete has generally been limited to impressed current systems due to the relatively high resistivity of concrete. More recently sacrificial anode materials have been developed which may be applied to reinforced concrete where particular criteria can be met. A typical circuit diagrams (schematic) of an impressed current cathodic protection system (ICCP) and a sacrificial cathodic protection system are shown in Figures 1 and 2 respectively.

The design concept of an impressed-current cathodic protection system would consist of the following:

(a) System hardware, consisting of a power source and ground-bed systems plus associated cabling, junction boxes, conduit etc.

(b) Control and monitoring system, consisting of feedback control equipment and an array of embedded reference electrodes plus cabling, junction boxes conduit etc.

The most important element of any successful cathodic protection system is the design of an effective anode system to distribute the necessary protection current economically and efficiently to the reinforcement. Also, it must be easy to install and possess long term durability. Other components (e.g. power supply/monitoring equipment etc.) of the CP system can then be selected to suit the anode system, the prevailing corrosion conditions and the environment.

Over the last 30 years there have been considerable advances and developments in anode materials and anode system design with real possibility of 'pick n mix' cathodic protection system(s) for above ground reinforced concrete structures. Anode systems currently available are given in Table 1.

Conductive coating anodes include a variety of formulations of carbon pigmented solvent or water dispersed coatings, and thermal sprayed zinc. Recently, thermal sprayed titanium has been used experimentally, with a catalysing agent spray applied to the titanium coating[9].

Mixed metal oxide coated titanium mesh or grid anode systems are fixed to the surface of the concrete and overlaid with a cementitious overlay which can be poured or pumped into shutters or sprayed.

Discrete anodes are usually installed in purpose cut holes or slots in the concrete. They are either:

- Rods of coated titanium in a carbonaceous backfill
- Mixed metal oxide coated tubes
- Strips and ribbon
- Conductive ceramic tubes in cementitious grout.

More recently, there have been successful experiences with the anode design based on utilising zinc rich paint as a sacrificial/impressed current anode material (Das, 1999).

The main characteristic properties of the zinc rich coating are:

(a) Coating is easy and safe to use. It is a one-pack compound containing 99.995% purity electrolytic zinc dust mixed in synthetic resins, pigments and aromatic solvents.

(b) Purity of the zinc content is such that there is no lead or cadmium present. The product does not contain toluene, xylene or methyl ethyl ketones (MEKs). Thus the product is non-toxic.

(c) On application it cures to a minimum of 96% zinc content in the dry film; thus is capable of providing full cathodic protection. The coating can be brushed or sprayed on. There is no barrier or interface between coatings i.e. every coat merges perfectly with previous coats and therefore can be topped up time and time again to provide indefinite cathodic protection at very low cost.

(d) On the steel surface the coverage is approximately 4-5 square metres at 30 to 40 microns.

(e) Coating has indefinite shelf life.

(f) Coating can be applied in moist or wet conditions.

Another recent development is the 'Discrete' Zinc Sacrificial Anode System. This is a proprietary zinc sacrificial anode unit embedded within a specifically formulated cementitious mortar and is currently available commercially. The main application of this anode system is for localised protection of steel reinforcement within chloride contaminated concrete by maintaining galvanic protection in areas adjacent to 'conventional patch repaired' areas and thereby prevents the formation of incipient anodes in neighbouring areas following anti-corrosion treatment and concrete repair to damaged areas.

This anode system is discretely placed within the patch repairs at maximum 750mm centres. Electrical connections are achieved by attaching tight wire ties, integral to the anode system, to the steel reinforcement; and then the areas are instated using appropriate repair mortar. More recently, trials in Norway have demonstrated that the woven carbon mesh with cementitious grout could be used as an anode system for the cathodic protection of reinforced concrete (Vennesland et al, 2006).

Not all of the anode systems, mentioned above proved effective, successful or suitable for all types of structural elements. The selection of the most suitable anode system(s) would depend on corrosion morphologies and structural geometry.

Advantages of cathodic protection

The principal advantage of cathodic protection over traditional repair is that only damaged concrete areas (i.e. spalled, delaminated or severely cracked) need to be replaced. Concrete, which is contaminated with chloride but otherwise sound, can remain since the possibility of subsequent corrosion will be prevented by the appropriate electrochemical process. The costs involved in the installation and operation of the cathodic protection system are more than offset by the savings which result from the reduction in concrete repair quantities and shorter duration of site work. In many cases, the reduction in repair may obviate the need for temporary propping with consequent reduction in costs.

CP does not restore lost steel, but provided the steel has sufficient reserves of strength, CP can provide a cost effective solution. Even when the strength is inadequate it is possible, in many cases, to combine CP with strengthening. With a well designed and installed CP system, the costs of operation and maintenance would be extremely low. It is now well recognised that in most cases cathodic protection can provide a cost effective solution to stop corrosion and the importance is acknowledged with codifying by a number of national and international standards[5].

The performance of the installed cathodic protection systems would be monitored using embeddable reference electrodes and other monitoring probes. All reference electrodes could be integrated into a Monitoring Unit and could be interrogated either manually or via an automatic data logging device which could be operated locally or remotely. In addition, the monitoring reference electrodes for ICCP systems would function to control the system output to provide adequate levels of protection.

Corrosion inhibitors

Corrosion inhibitors work by chemically raising the threshold of chloride required to initiate de-passivating the reinforcement and initiate corrosion. Corrosion inhibitors have been used for many years in the automotive industry and have been demonstrated to be effective in new build concrete, in particular the use of calcium nitrite as an admixture to fresh concrete. However, the application of corrosion inhibitors to existing chloride contaminated concrete has been shown to be less effective[2].

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Conclusions

The national and local government policy of providing unrestricted access to highway structures for free movement of trade, commerce and other road users requires that they are well maintained. The importance of inspection and testing (using various NDT techniques for identification and quantification of defects together with investigation into the cause and the consequence of the defects) was highlighted as an integral part of a well organised programme for Highway Maintenance Plan. Without this information a management decision with regard to technically correct and economically cost effective repair/rehabilitation options may not be possible.

Various methods of protecting corrosion damaged structures were discussed and it was concluded that cathodic protection is the most appropriate and proven technique to stop corrosion of steel in reinforced concrete structures.

For long-term durability of repair of concrete structures damaged by reinforcement corrosion, particularly in chloride-contaminated concrete, cathodic protection is recognised by highways authorities and building owners as the most cost effective method of concrete rehabilitation. The latest survey suggests that over 1 million m^2 of cathodic protection systems have been applied to highway structures and buildings worldwide. In the UK the application of cathodic protection systems has been reported for over 200,000 m^2 of concrete structures. A large number of installed CP systems are in operation and have been performing successfully for more than 20 years.

References

1. ACI. Service–life prediction – state of art report, American Concrete Institute, Detroit, 2000, ACI committee 365.
2. Alexander, M.G. Concrete Repair, Rehabilitation and Retrofitting, Taylor and Francis, London, 2006.
3. Amey, S. L., Johnson, D. A., Miltenberger, M. A. & Farzam, H. Predicting the service life of concrete structures: an environmental methodology. ACI Structural Journal, 95, No. 2, 1998, 205-214.
4. BRITE/EURAM. The Residual Service Life of Reinforced Concrete Structures. Synthesis Report , Contract No. BREU-CT92-0501, 1995.
5. BSI British Standards. Cathodic Protection of Steel in Concrete, BSI, London, 2000, BS EN 12696.
6. Chess, P. M. Cathodic Protection of Steel in Concrete. E & N Spon, 1998.
7. Clifton, J. R. & Pommersheim, J. M. Predicting Remaining service life of concrete, Proceedings of International Conference on Corrosion and Corrosion Protection of Steel in Concrete, Sheffield, 1994.
8. Clifton, J. R. Predicting the service life of concrete. ACI Materials Journal, 90, No. 6, 1996, 611-617.
9. Cramer, S. D. Corrosion prevention and Rehabilitation Strategies for reinforced Concrete Coastal Bridges. Cement and Concrete Composites, 24, No, 1, 2002, 101–117.
10. Danish Standards Association. Repair of concrete structures to EN 1504: a guide for renovation of concrete structures, repair materials and systems to the EN 1504 series, Dansk Standard, Butterworth-Heinemann, 2004.
11. Das, S. C. Confidential Report; Cathodic Protection Design Document for Bridge in Essex, 1999.
12. Fraczek, J. A Review of Electrochemical Principles as Applied to Corrosion of Steel in a Concrete or Grout. Corrosion, Concrete, and Chlorides. ACI SP-102, 13-24, 2001.
13. Highways Agency. Cathodic Protection for use in Reinforced Concrete Highway Structures. Design Manual for Roads and Bridges, 2002, Volume 3, Section 3, Part 3, BA 83/02.
14. Koch, G. H., Brogers, P. H., Thompson, N., Virmani, Y. P. & Payer, J. H. Corrosion cost and preventive strategies in the United States. FHWA report, FHWA – RD – 01 -156, Federal Highway Administration, Washington DC, 2002.
15. NACE. Cathodic Protection of Steel in Atmospherically Exposed Concrete Structures, 2000, NACE RP0290.
16. National Cooperative Highway Research. Manual on Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements. Transportation Research Board of The National Academies, Washington, D.C., 2006
17. Neville, A. M. Properties of Concrete; Fourth and Final Edition, Pearson Education, 2002.
18. Tuutti, K. Corrosion of Steel in Concrete. Swedish Cement and Concrete Research Institute, Sweden, 1982.
19. Uhlig, H. H. Corrosion Handbook. John Wiley & Sons, 2nd Edition, 2000.
20. Vennesland, O., Hang, R. & Mork, J. H. Cathodic protection of reinforced concrete – a system with woven carbon mesh, Concrete Repair, Rehabilitation and Retrofitting, M. G. Alexander (eds), Taylor and Francis, London, 2006.
21. Wallbank, E. J. The performance of Concrete in Bridges: A survey of 200 Highway Bridges. HMSO, London, 1989.
22. Zhang, J. & Mailvaganam, P. Corrosion of concrete reinforcement and electrochemical factors in concrete patch repair. Canadian Journal of Civil Engineering, 33, No. 6, 2006, 785-793.

Stochastic based lifecycle planning tool for ancillary pavement assets

David C Wightman

Principal Consultant

Highway Asset Management Group

Atkins

Seosamh B Costello

Senior Lecturer

Department of Civil and Environmental Engineering

The University of Auckland

Angela J Owen

Senior Engineer

Asset Management Branch

Trunk Roads Network Management

Transport Scotland

Abstract

This paper describes the application of a stochastic based lifecycle planning tool, previously utilised for the strategic assessment of pavement maintenance funding and policy decisions, to ancillary pavement assets such as road markings and safety barriers. The stochastic approach recognises the fact that not all assets of a particular type will deteriorate at the same rate, or in a strictly deterministic manner.

Historically, relatively little effort has been expended on investment planning for ancillary assets, compared to the pavement and structures stock. While this is understandable, given the relative asset values, recent public spending cuts have highlighted the need to include all pavement assets in public spending reviews. Unfortunately, because of this historic precedence, an understanding of current asset condition and future asset performance is lacking in most authorities for their ancillary pavement assets.

Transport Scotland, the agency responsible for the management of all trunk roads in Scotland, have addressed the former through the inclusion of condition assessment, as opposed to exception reporting, in their detailed inspection programme. A condition assessment manual has also been developed to assist inspectors in carrying out their assessments. Knowledge of asset performance will come with time, however in the interim, expert engineering knowledge has been harnessed to populate the deterioration algorithms. The application of the analytical tool in Transport Scotland is demonstrated and lessons learnt reported.

Introduction

Transport Scotland is the agency responsible for the management of the trunk road network in Scotland. The recent Asset Management Improvement Programme (AMIP) undertaken by Transport Scotland included a review of its existing asset investment planning capability[1]. This review identified a number of areas of comparative weakness and, therefore, opportunities for the strengthening of its investment planning capability. Foremost amongst these was a perceived weakness in the lifecycle analysis capability for ancillary assets.

In the Transport Scotland context, ancillary assets encompass a wide range of asset types (including road markings, road signs, traffic signals, safety barriers, etc.) as defined in the Transport Scotland 3rd Generation (3G) Maintenance Contract[2]. Historically, relatively little effort has been expended on investment planning for ancillary assets, when compared with (for example) pavement and structural assets. Whilst this situation is understandable given the respective asset values and associated annual expenditure, recent public spending cuts in the UK have increased the need for all trunk road agencies to demonstrate that scrutiny is being applied in all areas of expenditure.

The AMIP review of investment planning capability described above, made a number of recommendations for improving Transport Scotland's investment planning capability. Foremost amongst these was the development of a lifecycle planning tool for ancillary assets. The development of any lifecycle planning tool typically requires a number of key elements to be in place, namely:

- Inventory data – data on types and quantities of assets
- Condition data – current asset condition
- Intervention types – the range of treatments employed and their effect on asset condition
- Unit rates – the costs of enacting the various modelled treatments
- Deterioration models – to model the deterioration asset condition over time
- Scenarios - agreement on the type of decisions that that tool will be required to support
- Modelling approach – a modelling approach which is consistent with all of the above
- Implementation – of the above in a software tool.

Because of the lack of historic precedence in the modelling of ancillary assets in Transport Scotland, none of these necessary elements were in place at the outset of the AMIP. A plan was therefore developed to put in place each of these elements and thereby implement the required investment planning capability[3]. Whilst the above list implies a particular order of work, a number of tasks were executed in parallel. This paper outlines the process employed, and the various decisions made in the development of the ancillary asset lifecycle analysis capability for Transport Scotland.

Data collection

As described previously, a primary barrier to the introduction of a lifecycle analysis capability for ancillary assets for many trunk road agencies is a lack of knowledge regarding the condition and performance of such assets. This is something of a catch-22 situation. Since a model cannot be developed without this key information, and conversely, there is little motivation to collect such data if it is not to be used as part of a formalised decision-making process. A key phase therefore in the development of the Transport Scotland capability was a targeted data collection exercise.

It should be noted at this point that whilst Transport Scotland has overall responsibility for the management of the Scottish trunk road network, it subcontracts many of its day-to-day, operational responsibilities to four Operating Companies (OCs), each having responsibility for a geographical region (i.e. South West, North West, North East, and South East).

The OCs were commissioned by Transport Scotland to undertaken this data collection exercise. The OCs were provided with data sheets on which 57 separate ancillary asset types, as derived from the 3G contract[2], were listed. For each of the listed asset types, the OCs were requested to provide values for the following parameters:

- Inventory – quantities for each specified asset type (in the appropriate units)
- Asset lives – expected asset lives / replacement intervals
- Replacement costs – the unit cost of replacing each asset type (accounting for regional variations)
- Current condition – current condition of assets.

Due to the historical precedent described above, much of this data was not readily available. In the case of inventory data, asset quantities were obtained from the OCs' Routine Maintenance Management System (RMMS) or from other local sources. Where such sources were not available (or data was deemed to be unreliable), quantities were obtained either via a limited survey or by estimation. In the case of asset lives, values were either derived from the RMMS or other local data sources, or were based on expert engineering judgement. Unit rates for asset replacement were derived either from OC framework rates or were based on engineering experience.

Condition assessment

In the case of asset condition data, virtually no data was held by the OCs at the outset of the study. The lack of condition data for ancillary assets is a situation that is not unique to Transport Scotland. The majority of RMMS, are not typically configured to store asset condition, and inspection regimes typically do not cover asset condition. This is particularly the case for ancillary assets. Furthermore, because of the diversity of the ancillary asset stock there are no standardised approaches to assessing current condition.

In order to address this gap it was decided that a condition survey would be conducted. Whilst objective condition measures can be employed for certain assets (for example, retro-reflectivity readings for road markings and road traffic signs), for the majority of ancillary assets subjective visual assessment is required. In order to assist OC inspectors in making these subjective judgements when collecting condition data, a condition assessment manual was developed[4]. In an attempt to minimise the subjectivity of inspections, the manual makes extensive use of images to illustrate asset condition, as well as detailed description of assessment criteria. An extract from the condition assessment manual, relating to the condition of road markings, is shown in Figure 1.

A key tenet of the assessment method supported by the manual is the definition of asset condition (for all ancillary asset types) in terms of condition bands. Five conditions bands have been adopted, namely A, B, C, D and E, with A representing


Road Markings

Permitted Inventory Items

- Road Markings Hatched (LH)
- Road Markings Longitudinal (LL)
- Road Markings Transverse and Special (RM)
- Reference Marker Point (RF) (thermoplastic cores)

Condition Requirement	Assess and record the condition of road markings in paint or thermoplastic materials of all types and colours.	
Condition Reporting	Per 100m length	For Hatched (LH) and Longitudinal (LL) road markings the condition rating is based on an overall assessment of each 100m length.
	Per point item	For Transverse and Special (RM), Pedestrian Crossing (PX) and Reference Marker Point (RF) the condition is assessed for each point item.
Inspection Interval	1 year	

Condition Category	Condition Criteria[1]
A. Excellent	New or nearly new condition. Score of 5/5 on the visual assessment of 'Wear' (TD26/07).
B. Good	No visual defects and with few visible signs of deterioration. Good night time conspicuity and very little wear, score of 4/5 on the visual assessment of 'Wear' (TD26/07). >100mcd/m²/lux retroreflectivity.

Figure 1 - Extract from Condition Assessment Manual

Table 1 - Extract from data collection return from selected Operating Company

Asset Group	Estimated Quantity	Unit of Measure	Condition (% of overall quantity in Condition Bands A - E)					Revised Asset Life
			A	B	C	D	E	(yrs)
Covers, Gratings and Frames - Catchpit (CP)	1,296	No.	16.75	18.75	34.00	17.75	12.75	30
Covers, Gratings and Frames - Manhole (MH)	9,137	No.	16.25	20.25	32.50	18.50	12.50	30
Covers, Gratings and Frames - Interceptor (IN)	0	No.	0.00	0.00	0.00	0.00	0.00	30
Covers, Gratings and Frames - Gully (GY)	26,952	No.	15.50	20.75	30.50	20.00	13.25	25
Drainage Channels (CH) - V-Channels	8,432	per 100 m	17.00	21.50	17.50	25.00	19.00	30
Drainage Channels (CH) - Drainage Channel Block	4,129	per 100 m	18.00	11.50	23.00	30.00	17.50	30
Drainage Channels (CH) - Slot Drains	6,543	per 100 m	14.33	13.33	28.33	25.33	18.67	25
Drainage Channels (CH) - Combined kerb-drains	9,152	per 100 m	23.00	23.25	21.75	18.25	13.75	30
Drainage - Piped Drainage (PD)	800	m	22.00	26.00	33.00	12.00	7.00	40
Drainage - Piped Grip (PG)	542	m	10.00	10.00	25.00	35.00	20.00	20
Filter Drainage - Counterfort Drains (CD)	9,447	per 100 m	8.33	16.00	35.00	25.33	15.33	20
Filter Drainage - Filter Drains (FD)	403,165	per 100 m	12.75	17.50	31.25	21.50	17.00	15
Drainage - Ditches (DI)	24,404	per 100 m	13.25	16.75	35.25	20.25	14.50	15
Drainage - Culvert (CV)	12,372	No/m	12.25	19.75	35.00	20.25	12.75	40
Drainage - Balancing Ponds (BP)	6	No.	25.00	25.00	40.00	5.00	5.00	20
Drainage - Ancillary Items (1) - Aprons	33	No.	10.00	19.00	44.00	27.00	0.00	40
Drainage - Ancillary Items (1) - Grilles	180	No.	8.00	13.00	36.33	27.67	15.00	30
Drainage - Ancillary Items (1) - Headwalls	592	No.	6.33	11.33	44.00	24.67	13.67	40
Drainage - Ancillary Items (1) - Spillways	0	No.	0.00	0.00	0.00	0.00	0.00	40
Drainage - Ancillary Items (1) - Trash Screens	Inc in grilles	No.	0.00	0.00	0.00	0.00	0.00	30
Drainage - Ancillary Items (2) - Penstocks	0	No.	0.00	0.00	0.00	0.00	0.00	40
Drainage - Ancillary Items (2) - Sluices	23	No.	0.00	20.00	50.00	20.00	10.00	30
Drainage - Ancillary Items (2) - Tidal flaps	5	No.	10.00	30.00	30.00	15.00	15.00	30
Drainage - Ancillary Items (2) - Water gates	0	No.	0.00	0.00	0.00	0.00	0.00	30
Drainage - Ancillary Items (3) - Pumps	1	No.	0.00	0.00	100.00	0.00	0.00	20
Drainage - Ancillary Items (3) - Valves	22	No.	0.00	20.00	60.00	10.00	10.00	20
Drainage - Ancillary Items (3) - Other specialist equipment	0	No.	0.00	0.00	0.00	0.00	0.00	25
Fences and Barriers - Fences and Barriers (FB)	166,012	per 100 m	11.75	26.00	24.50	21.75	16.00	20
Fences and Barriers - Traffic Control Barriers (CB)	8	No.	0.00	0.00	50.00	25.00	25.00	15

new condition through to E representing the end of the asset's serviceable life. The condition of all assets is defined in terms of one of these five bands. The manual defines the asset condition that constitutes each band. A field trial was conducted by one of the OCs to determine its suitability for operational use. This trial identified a number of areas for improvement, and a revised version of the manual was then produced. Table 1 shows an extract of the data collection return from one of the four OCs. It can be seen in this table that in addition to the total number of each asset type being specified, the respective OC has also specified (as a percentage) the distribution of assets between the five condition bands.

It should be noted that the condition assessment manual and the associated condition survey were not instigated purely for the purposes of this study. It is the intention of Transport Scotland that the surveys will in future be conducted at the intervals specified in the manual. Amongst other benefits, this activity will provide a good base of time-series asset performance data which can be used in future to calibrate asset performance models. Whilst the condition data collection undertaken for this study primarily comprised desk-studies and visual surveys, this does not preclude the use of video surveys or other techniques in the future.

Model scenarios

At the outset of the study, Transport Scotland specified that the new lifecycle planning tool should support the investigation of two primary investment scenarios, namely:

- Defined Condition – what budget is required to maintain the ancillary asset network at a defined condition level?
- Defined Budget – what is the best network condition that can be achieved for a specified budget?

On-going discussions with Transport Scotland as the project progressed identified two further investment scenarios:

- Prioritisation 1 – where the budget is insufficient to achieve the defined condition, allocate funding to assets on a user-defined priority basis. What is the resulting impact on network condition?
- Prioritisation 2 – where the budget is insufficient to achieve the defined condition, endeavour to ensure that user-defined minimum performance (condition) targets are met, before allocating the remaining budget on a user-defined priority basis. What is the resulting impact on network condition?

The 'Prioritisation' scenarios described are a reflection of the financial climate in which many road agencies currently operate. Whilst funding has not yet reached the point inferred implied by these scenarios, Transport Scotland, as responsible asset managers, are keen to investigate the likely impact should this scenario arise. It should be noted that whilst the tool resulting from this study is capable of handling each of the above scenarios, it is also capable of investigating a wider range of scenarios, in addition to those listed here.

Choice of modelling approach

A number of life-cycle planning tools, both deterministic and stochastic in nature, are available to develop life-cycle plans. A variant of the stochastic modelling tool developed by Costello et al.[5] has been used to develop lifecycle plans for a number of UK road agencies, for both carriageway and structure assets. Stochastic models are also widely used elsewhere and therefore provide a relatively well-proven approach. Prominent among these are the Transport Research Laboratory's network condition model[6], the Finnish National Road Administration's highway investment programming system[7] and the network optimisation system[8], as used by a number of state highway administrations in the USA.

The stochastic-based approach to lifecycle-planning typically involves a number of common, fundamental concepts, as follows:

- Asset Groups – the arrangement of assets into groups of assets with homogeneous characteristics (based on performance and reporting requirements)
- Condition Bands – the adoption of a single condition measure which is rated in terms of a discrete number of condition states or bands.
- Condition Distribution – the range of condition states for the various assets in each asset group is defined in terms of a condition distribution.
- Transition Matrices – the deterioration of asset condition is modelled using transition probability matrices which model the transition of assets from one condition band to another, over time.

Each of these concepts is described in more detail later in this paper. Ancillary assets are deemed to be highly suited to the stochastic-based approach as follows:

- Asset Groups - ancillary assets naturally lend themselves to being modelled at a group level due to their diversity (the range of different asset types) and the homogeneity within each group.
- Condition Bands/Distribution - the condition assessment approach adopted for this study (based on bands, and as embodied in the Trunk Road Condition Assessment Manual) naturally lends itself to this form of modelling.
- Transition Matrices – in the context that there is negligible empirical data on the performance of ancillary assets, the transition matrix approach provides a pragmatic means of establishing initial deterioration models, based on expert engineering judgement.

As asset performance data becomes available through the condition monitoring programme, more sophisticated deterioration models can be developed.

Model implementation

Having selected the modelling approach to be employed, and having undertaken the data collection exercise required to populate the model, the next phase in the study was the development of the actual model. The following sections describe the overall model build process, and in particular, how the various stated components of the stochastic-based approach (namely asset groups, condition distribution, and transition matrices) were implemented.

Asset groups

Under the stochastic-based approach to life-cycle planning, individual assets are typically aggregated into asset groups. These groups should be homogeneous in nature (particularly in terms of performance) thereby allowing them to be modelled as a group. At stated above, the Transport Scotland 3rd Generation Contract[2] defines the list of 57 ancillary asset types to be managed by the Operating Companies. Given that this list of asset types is well-established in the Transport Scotland context, it was decided to retain this classification in the lifecycle model. Further discussions established that in addition to reporting model outcomes by asset type, Transport Scotland also wanted outcomes to be reported by region (or OC). Therefore, it was decided that the model would employ 228 asset groups (57 x 4). Whilst this number is relatively high when compared with the number employed in other implementations of the stochastic approach, such as that described by Costello et al.[5], the number was judged not to be excessive due to a) the short model run-times, and b) flexible reporting facilities included in the model which allow the end-user to decide the groupings to be used in output reports.

Current condition

Current network condition is represented in the model as a distribution of condition, defined as the proportion of each asset group lying in each condition band. This provides the base year scenario or starting point for life-cycle planning.

Mathematically, the initial state of any process may be described by a starting vector, in which the sum of all α_i should be equal to one, and all entries should be positive values. This starting vector is represented by:

$$a_0 = [\alpha_1 \ \alpha_2 \ \alpha_3 \ldots \ \alpha_n] \quad (1)$$

In the Transport Scotland context, the Operating Companies have supplied the condition distribution for each asset type. as illustrated in Table 2 above. Taking the first asset type in this table as an example (i.e. Catchpit), it can been seen that at the time of data collection, 16.75% of catchpits were adjudged to be in condition band A, 18.75% in B, 34% in C, 17.75% in D and 12.75% in E. Using these percentages, the starting vector for this asset group would be represented by:

$$\boldsymbol{a}_0 = [0.1675 \ \ 0.1875 \ \ 0.34 \ \ 0.1775 \ \ 0.1275] \quad (2)$$

In this way starting vectors for all 228 asset group were derived. These starting vectors were used to populate the model.

Deterioration modelling

In the stochastic-based approach, asset deterioration is modelled by means of the transition probability matrix (TPM), denoted by P. The general form of P is given by:

$$\boldsymbol{P} = \begin{bmatrix} p_{11} & p_{12} & \cdots & p_{1n} \\ p_{21} & p_{22} & \cdots & p_{2n} \\ \vdots & & & \vdots \\ p_{n1} & p_{n2} & \cdots & p_{nn} \end{bmatrix} \quad (3)$$

This transition probability matrix contains all the information necessary to model the deterioration of the respective asset group. The transition probabilities, p_{ij}, indicate the probability of the portion of the asset group in condition i moving to condition j in 1 year due to the damaging effects of traffic and environment, as applicable.

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Similar to the starting vector, for every TPM the sum of the entries in each row is equal to one, and all entries are non-negative. In matrix notation, the probability distribution at a specific elapsed time in years, say t = 1, is given by:

$$a_1 = a_0 P \tag{4}$$

Or more generally, the probability distribution at any time t may be calculated using:

$$a_t = a_o P^t \tag{5}$$

Asset deterioration can therefore be modelled using the above equation, where a_t is the distribution of condition at time t, a_0 the distribution of condition at time 0 (that is the starting vector), and P^t the TPM raised to the power of t, the elapsed time in years.

Two more conditions apply to the process when used to simulate asset deterioration. First $p_{ij} = 0$ for $i > j$, reflecting the general belief that assets cannot improve in condition without first receiving some form of treatment. Secondly, $p_{nn} = 1$, signifying a holding state whereby assets that have reached their worst condition cannot deteriorate further. Consequently, in asset deterioration the general form of the transition matrix P is denoted by:

$$\mathbf{P} = \begin{bmatrix} p_{11} & p_{12} & p_{13} & \cdots & p_{1n} \\ 0 & p_{22} & p_{23} & \cdots & p_{2n} \\ 0 & 0 & p_{33} & \cdots & p_{3n} \\ \vdots & \vdots & \vdots & & \vdots \\ 0 & 0 & 0 & \cdots & 1 \end{bmatrix} \tag{6}$$

Initial deterioration models

Given that there was insufficient historical data available to Transport Scotland and its OC to be used in deriving performance models for ancillary assets, expert engineering opinion was instead used. As described above, estimates of asset lives were captured during the data collection exercise. An interesting finding arising from this exercise was the range of service life estimates for certain asset classes in the various regions. This finding reflects the very diverse nature of the Transport Scotland network, ranging from heavily-trafficked urban environments in the south of the country, to relatively lowly-trafficked rural environments in the north. This diversity in service lives is also due to the diversity of environmental conditions experienced in the various regions, with weather conditions often very severe in northern regions (both in terms of temperature and rainfall). It should be noted that in estimating asset lives, it was assumed that routine maintenance is undertaken on all assets at the specified intervals, for example, washing of sign-faces. Examples of estimated asset lives are given in Table 1.

The estimates of asset life obtained by this process were then used to generate a separate TMP for each asset group. For the time being, linear deterioration rates were assumed for all asset types. It should be noted that the assumption of a linear deterioration rate is purely a starting point from which to carry-out an analysis in the first year of the newly introduced condition assessment regime. As deterioration information becomes available through the condition monitoring programme, these initial assumptions can be challenged and the performance models refined accordingly.

The generic form of the transition probability matrix is given below, where n is the number of condition bands and L is the asset life as supplied for each asset type:

$$P = \begin{bmatrix} 1-\left[\frac{n-1}{L}\right] & (n-1)/L & 0 & 0 & 0 \\ 0 & 1-\left[\frac{n-1}{L}\right] & (n-1)/L & 0 & 0 \\ 0 & 0 & 1-\left[\frac{n-1}{L}\right] & (n-1)/L & 0 \\ 0 & 0 & 0 & 1-\left[\frac{n-1}{L}\right] & (n-1)/L \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix} \tag{7}$$

The generic form of the transition probability matrix is best demonstrated by means of an example. Taking an imaginary asset group as an example, where n is 5 and L is 7, the resulting transition probability matrix would be as given below:

$$P = \begin{bmatrix} 0.43 & 0.57 & 0 & 0 & 0 \\ 0 & 0.43 & 0.57 & 0 & 0 \\ 0 & 0 & 0.43 & 0.57 & 0 \\ 0 & 0 & 0 & 0.43 & 0.57 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix} \tag{8}$$

Using the above generic TPM form, and assuming a linear deterioration rate, TPMs can be manually-derived. Alternatively, the tool developed as part of this study has been provided with a facility to automate the generation of TPMs for a user-specified L value (in the case of the Transport Scotland model, the value of n is fixed at 5). Figure 3 shows a screenshot from the relevant part of the user interface.

Future refinements to deterioration models

Although the approach described above enables transition probability matrices to be determined with minimal data (in this case from just n and L values), the more comprehensive the data-set used, the greater the confidence in the resulting performance models. As actual asset lives and deterioration rates become available through the condition monitoring programme instigated as part of this study, the initial transition probability matrices will be refined accordingly.

Performance monitoring and continual feedback into the performance models is necessary in order to ensure that increased levels of confidence in model forecasts are attained. The standard approach is to observe, from historical data,

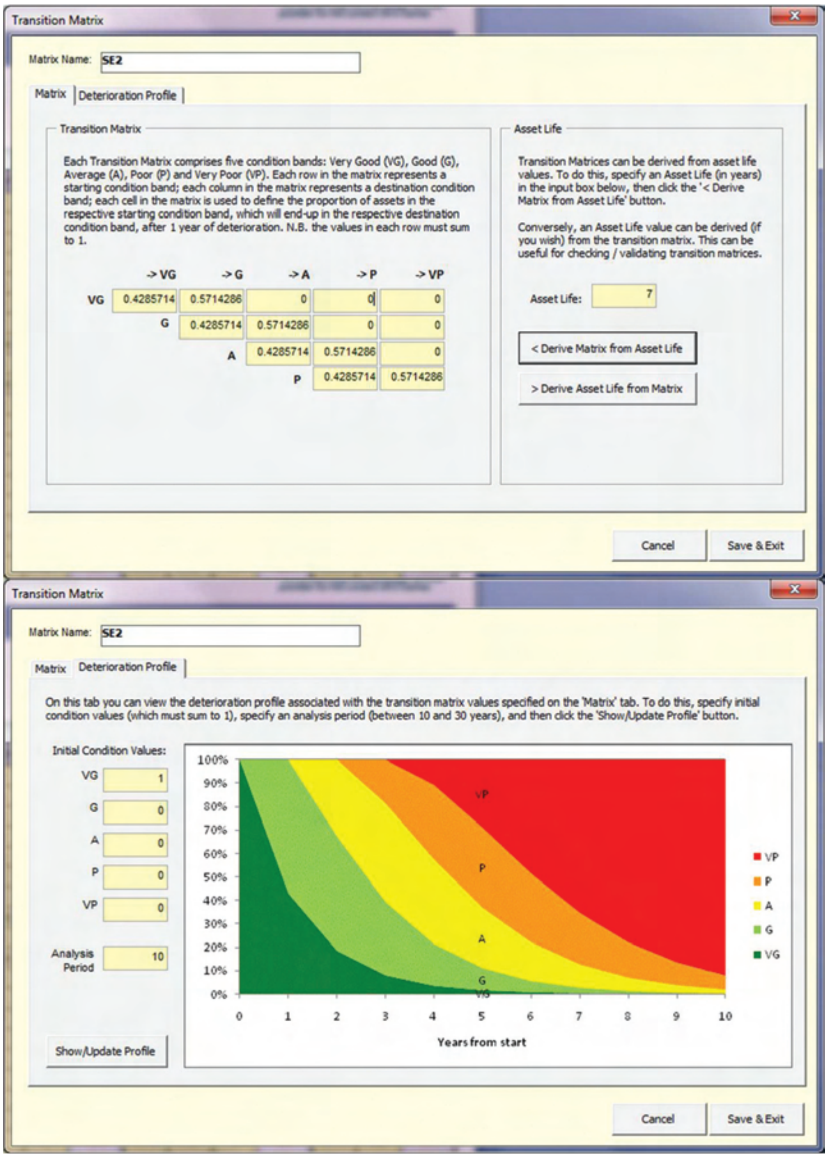

Figure 3 - Facility for deriving TPM from L value

the way in which an asset group deteriorates over time and use this to estimate p_{ij} using equation[9] below. N_{ij} is the number of assets in the asset group that moved from condition i to condition j during 1 year and N_i is the total number of assets that started the year in condition band i:

$$p_{ij} = \frac{N_{ij}}{N_i} \tag{9}$$

The proportions are likely to vary from year to year thereby requiring an average to be determined over time for each p_{ij} to ensure accuracy in the model. Alternatively, as deterioration curves are developed over time from historical data, the transition probability matrices can be determined using the transition matrix calculator developed by Costello et al[9]. based on the methodology defined by Ortiz-Garcia et al[10]. The transition matrix calculator determines the probabilities in the transition matrices based on a deterioration curve, coupled with an estimate of scatter or confidence in the model.

Analytical framework

The next phase in the study was the development of an analytical framework in which the various model elements established as described above (asset groups, starting condition vectors, and transition probability matrices) could be encapsulated. In simple terms, the modelling process employed is based on two main elements:

i) the calculation of the annual progression of condition distributions, followed by

ii) simulation of the effects of renewals applied in each year.

This iterative process is outlined in Figure 4 and detailed in the steps listed alongside it.

The deterioration modelling mechanism which is at the heart of this process can be best demonstrated by means of an example. Using the example given above, the distribution of current condition for catchpits reported above is taken as the starting vector, a_0, and the TPM for road markings: longitudinal (LL) developed above is taken as P. Then, to simulate 1 year's degradation, the two matrices are multiplied together. This is represented below:

$$a_0P = [0.1675 \quad 0.1875 \quad 0.34 \quad 0.1775 \quad 0.1275] \times \begin{bmatrix} 0.43 & 0.57 & 0 & 0 & 0 \\ 0 & 0.43 & 0.57 & 0 & 0 \\ 0 & 0 & 0.43 & 0.57 & 0 \\ 0 & 0 & 0 & 0.43 & 0.57 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix} \tag{10}$$

Each of the elements in the resulting matrix, a_1, is calculated by multiplying the first matrix, a_0, by each of the columns in the second matrix in turn. The resulting distribution of condition, a_1, is as follows:

$$a_1 = [0.072 \quad 0.176 \quad 0.253 \quad 0.270 \quad 0.229] \tag{11}$$

This means that after 1 year's deterioration 7% of the network will be in condition band A, 18% in B, 25% in C, 27% in D and 23% in E. This process can be continued indefinitely to provide a distribution of condition in future years.

Modelling of interventions

The scenario (demonstrating year on year deterioration), assumes that no maintenance interventions are enacted. In practice renewals are carried out on a yearly basis as dictated by the intervention levels and available budget.

Assuming that catch-pits are renewed when they reach condition band E, then 22.9% of the network will require renewal at the end of year 1. When a renewal is applied it is assumed to restore the asset to its original condition. This is simulated by returning the proportions renewed to

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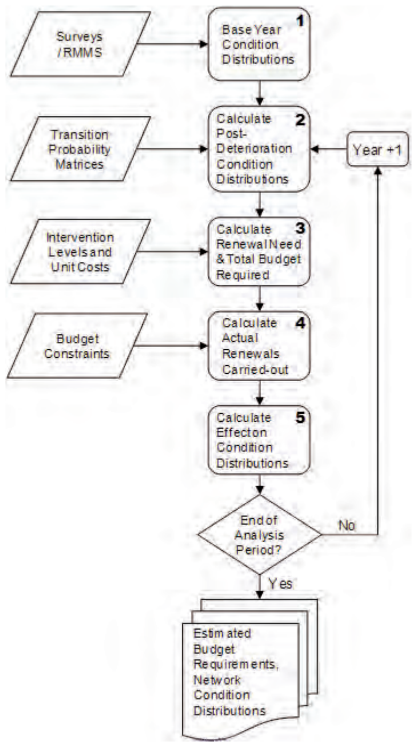

1) The base year condition distributions or starting vectors, a_0, are derived from current condition data. Where inspection intervals are not annual, as may be the case for asset types deemed to be low-risk, then the condition distributions can be projected forward to the base year, if required, using the appropriate TPM, to derive the base year condition distributions.

2) The condition distribution of each asset group post deterioration is predicted using the TPM.

3) The proportion of each asset group requiring renewal is then calculated based on the post-deterioration condition distributions and the specified intervention levels.

4) The percentage of each asset group that will actually be renewed is calculated based on the specified budget constraints.

5) The effect that the renewals will have on the condition of each asset group is calculated.

The above process is repeated for each year of the analysis period until the end of the analysis period is reached.

Figure 4 - Analytical framework

condition band A. Assuming sufficient funds are available, then 22.9% of catch-pits will be renewed at the end of year 1, resulting in 30.1% of the network in condition band A, 17.6% in B, 25.3% in C, 27% in D and 0% in E. This then becomes the starting vector for year 2, that is:

$$\boldsymbol{a}_2 = [0.301 \quad 0.176 \quad 0.253 \quad 0.270 \quad 0.0] \tag{12}$$

The budget required in year 1 to achieve this target performance can then be calculated by multiplying the unit cost for catch-pits by the quantity then by 30.1%. The budget required in subsequent years can be obtained by repeating the above process as required. This essentially answers the 'what-if ' question, 'what budget is required to support a target network performance?'.

Budget allocations

The above example assumes that sufficient funds are available to renew all catch-pits in band E. However, in practice this is rarely the case. Consequently, renewal budget limits can be specified within the model. This will also allow the 'what-if' impact of different levels of funding on network performance to be modelled.

Initially, the annual budgets are set, followed by the onwards allocation of the annual budgets to asset groups. Renewals are then applied until the relevant budget is exhausted, assuming the required expenditure exceeds the available budget. This results in a shortfall and parts of the network remaining untreated. This can be demonstrated by an example.

Assuming only a proportion of the funds required are available, say enough for only 3% of catch-pits, compared with the 22.9% requiring renewal at the end of year 1, then this will result in 10.2% of catch-pits in condition band A, 17.6% in B, 25.3% in C, 27% in D and 19.9% in E. This then becomes the starting vector for year 2, that is:

$$\boldsymbol{a}_2 = [0.102 \quad 0.176 \quad 0.253 \quad 0.270 \quad 0.199] \tag{13}$$

As before, the budget required in subsequent years can be obtained by repeating the above process as required. However, if the available budget exceeds the budget required in a particular year, then a surplus in the budget occurs. This surplus can then be reallocated to a different asset group. This can be carried out either through a manual iterative process whereby the user maintains complete control over the process or using the automated capability built into Transport Scotland system.

Conclusion

The stochastic-based lifecycle planning methodology described in this paper has shown itself to be appropriate for the modelling of ancillary assets on the Transport Scotland network. Such assets naturally lend themselves to being modelled at a group level due to their homogeneity. In addition, the assessment criteria defined in the condition assessment manual naturally lend themselves to this form of modelling.

The approach as applied in this study has some limitations as it stands. The deterioration models are based on engineering estimates of asset life and an assumption of linear deterioration. However, as deterioration information becomes available through the condition monitoring programme, these initial assumptions can be challenged and the performance models amended accordingly, thereby improving model accuracy.

Although subjective visual assessments are commonplace in inspections of highway assets, the reproducibility of assessments conducted using the condition assessment manual has yet to be determined and further research is required. Consequently, until such time as actual deterioration rates become available through the analysis of historical time-series data and the reproducibility of the assessment method is determined, the results of the model should be used with caution.

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References

1. Atkins. Review of Road Management Process, Work Package 12: Specification of a Computerised Decision Support Tool, Asset Management Improvement Programme, Transport Scotland, 2008.
2. Management, Inspection And Cyclic Maintenance, 3rd Generation Term Contract for Management and Maintenance of the Scottish Trunk Road Network - North West Unit, Schedule 7 Part 1, Transport Scotland, 2005.
3. Atkins. Specification and Performance Models for Ancillary Assets, Work Package 12: Specification of a Computerised Decision Support Tool, Asset Management Improvement Programme, Transport Scotland, 2009.
4. Transport Scotland. Transport Scotland Trunk Road Condition Manual, Asset Management Improvement Programme, Transport Scotland, 2010.
5. Costello, S. B., M. S. Snaith, H. G. R. Kerali, V. T. Tachtsi, and J. J. Ortiz-Garcia. Stochastic model for strategic assessment of road maintenance. Proceedings of the Institution of Civil Engineers, Transport 158(4): 203–211.
6. Kerali, H. G. R., and M. S. Snaith. NETCOM: The TRL Visual Condition Model for Road Networks. Transport Research Laboratory, Crowthorne, UK, Contractor Report 321, 1992.
7. Thompson, P.D., L. A. Nuemann, M. Miettinen, and A. Talvitie. A micro-computer Markov dynamic programming system for pavement management in Finland. Proceedings of the Second North American Conference on Managing Pavements, Toronto, Vol. 2. pp. 2.241–2.252, 1987.
8. Kulkarni, R. B. Dynamic decision model for a pavement management system. In Transportation Research Record: Journal of the Transportation Research Board, No. 997, Transportation Research Board of the National Academies, Washington, D.C., 1984, pp. 11-18.
9. Ortiz-Garcia, J. J., S. B. Costello, and M. S. Snaith. Derivation of transition probability matrices for pavement modelling. Journal of Transportation Engineering ASCE 132(2): 141–161, 2006.
10. Costello, S.B., J. J. Ortiz-Garcia and M. S. Snaith. Development of the transition matrix calculator. In Proceedings of the Tenth International Conference on Civil, Structural and Environmental Engineering Computing, Rome, Italy, 2005.

Paul Fraser

Senior Landscape Architect

Water & Environment

Atkins

Abstract

The scale and pace of global urbanisation is truly remarkable. A recent report estimates that India alone needs to build the equivalent of a new Chicago every year until 2030 to meet their demand, investing over $1trillion in the process[1]. Urban change on this scale coupled with the need to decarbonise the global economy challenges both clients and professionals to take a holistic approach to sustainability, from the fundamentals of land use planning to building typologies and infrastructure.

Under Atkins Carbon Tools programme, a team from across the Masterplanning Technical Network has developed a platform that will help us meet these challenges. A parametric 'BIM-style' (Building Information Modelling) platform has been developed that enables all data variables embodied in a masterplan to be tracked and optimised in real time - including both embodied and operational carbon. This allows the design team to understand the complex interrelationships between factors affecting sustainability outcomes in urban systems at the level of a masterplan. This paper explains the core capabilities of the tool, how it will revolutionise our approach to masterplanning, and the benefits of its use for our clients.

Introduction

The BIM revolution in architecture started in the 1980s, redefining the process of designing a building envelope through the use of an integrated design data model. By attaching metadata to drawn objects, it became possible to move beyond simple digital drafting to a dynamic design package that generates schedules, visualisation and procurement information in real time. The scope of these packages is often thought of as '5D' - the three traditional dimensions of design, plus time (4D) and cost (5D).

Masterplanners have over time made limited use of architectural BIM packages for their own ends, occasionally building bespoke tools, but there is very little currently on the market by way of parametric capability specifically for masterplanning. Although masterplanning and architecture both rely on interrelationships between drawn data and analytical/reporting tools, the parameters are very different.

Figure 1 - Suhar Urban Masterplan (Oman): City centre development proposals

Successful masterplans, see Figures 1 and 2, balance physical form with a broad range of factors that will ultimately determine its success as a place. In parallel with the investigation of BIM style solutions for our masterplanning business by the London studio, the brief was being developed under Atkins Carbon Tools programme for a tool to enable the forecasting of the carbon profile of a masterplan. The decision was taken to join the two workstreams together, and Holistic City Ltd were commissioned to build a bespoke Atkins carbon tool for their CityCAD platform.

Software architecture

For the tool to be globally applicable, it required a centrally hosted database for the carbon factors and the use of Sequel Server facilitated connection to desktop machines. The database was developed by a specialist third party supplier, Accelero Digital, with the data specification devised by the Sustainability team to make the final calculations as robust as possible in the context of the masterplan scale of design. Integration with the CityCAD application required the development of two new components to the core platform - one to facilitate retrieval of information from the database, and a second to define the changes to the user interface, including the new calculations and outputs. The application testing was done by a joint UK/China team. Development of the tool is now complete and the London, China and India teams are discussing its application with a range of potential and existing clients. Licences are available through the Atkins Carbon Tools Portal[2].

How do you calculate the carbon impacts of a Masterplan?

The tool allows the design team to arrive at aggregated values for both embodied and operational carbon across a major scheme of up to 50km x 50km (depending on the complexity of modelling). It is possible to use the tool in a detailed way, modelling collections of buildings with ancillary spaces, or in a more strategic mode – analysing representative development mixes in super blocks. This latter method makes allowances for, but does not actually model, 3D buildings.

Figure 2 - White City, Baku (Azerbaijan): Illustrative Masterplan

In order for the carbon calculation to be meaningful, the outputs are broken down by four key themes - land use, transport, waste treatment and renewable energy, for both operational and embodied carbon. This enables the team to see where design decisions have the most impact, see Figure 3, (either positive or negative), understanding the impact of design changes in real time. The software works by pairing assumptions (or factors) provided by the database with actual data measured from the 3D model. An example might be the potential for a building to accommodate electricity-generating technologies. Taking photovoltaic panels as an example, the database provides assumptions on what type of technology would be appropriate, the percentage of roof area available for it, its mounting angle etc. The model provides the available roof area, and the two are used to calculate the predicted electrical output. This is done for each building, with further positive contributions from stand alone generation kit (wind turbines etc.) to build a profile for the whole masterplan. In parallel with generation, the model also considers electricity demand, allowing the Masterplanning team to understand the net energy demand/generation of the scheme, and the resulting load on the local grid supply. This is one example of a whole range of metrics that can be used to 'balance' the masterplan, making it truly sustainable. The tool also includes a spatial trip distribution module to analyse carbon emissions associated with transport. This module considers the relative locations of key traffic generating and attracting land uses to determine a value for transport operational carbon. It incorporates a range of variables to enable the Transport Planning team to calibrate the model to its context, and allows the design team to 'test' the relative locations of key land uses such as hospitals and schools. Also important for the fundamental efficiency of the masterplan, in carbon terms, is the building orientation and block efficiency. The tool enables the user to see instantly the relative efficiency and resulting density of different block forms, see Figure 4, combined with other factors such as the resulting waste generated. The database supplies a value for the optimum orientation of a given building type in its geographic location, and the model provides measured quantities (for instance the actual roof area or orientation of a block. These factors are then combined in the model's database, combining across the masterplan to give a complete picture of the development's properties. Because the database uses a relatively simple (yet powerful) method of using parameters as multipliers on core values, such as Gross Floor Area (GFA), it is capable of tracking a host of numbers that are based on the development mix – much like a traditional methodology. One example of this is its ability to generate a cost model for the development by allowing the team to input cost per square metre construction costs alongside income projections to arrive at an initial understanding of the margins likely to be achieved

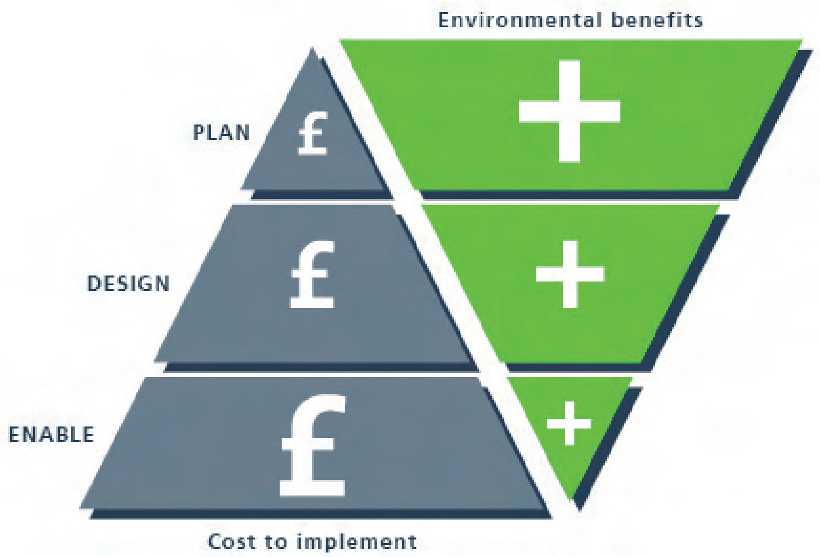

Figure 3 - Diagram showing the importance of land use decisions for sustainability

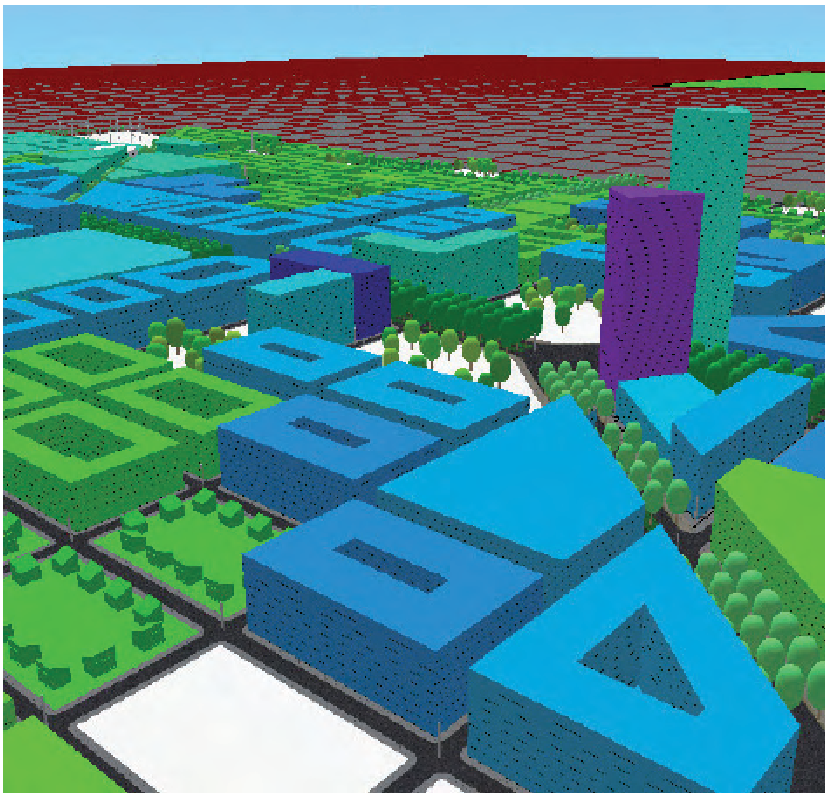

Figure 4 - Heat mapped view for anlysis of key factors

and the resulting viability thresholds of the site. This means we have the capacity to build in the client's cost model from the outset, working with them to optimise the development's layout including cost considerations - rather than it being measured by the client (or his Quantity Surveyor) after the design has been completed.

Benefits for our clients

The parametric capability of the Carbon Critical Masterplanning tool gives the Masterplanning team the capability to navigate complex interrelationships between city metrics in real time. This is the key to the power of the system. The questions of the impacts of relative activities in a proposed city could have been answered before, but through separate workstreams and with time delays while information was processed by different teams. The masterplanning carbon tool mitigates these possible delays to provide a more holistic solution.

It also places the decarbonisation of the masterplan at the heart of the design process, see Figure 5, meaning that both the design team and the technical specialisms involved can see the impact of design decisions on the masterplan's carbon footprint. By being highly visual, is also makes these factors accessible to a wide range of lay stakeholders, including the client, members of the public, local authority officers etc.

Alongside carbon optimisation, typical scenarios of interest to clients might include:

(1) What will be the impact on the financial model of the use of enhanced building standards?

(2) Can I optimise the phasing and distribution of floorspace against the capacity of the local property market? What will the impact be in terms of new infrastructure delivery?

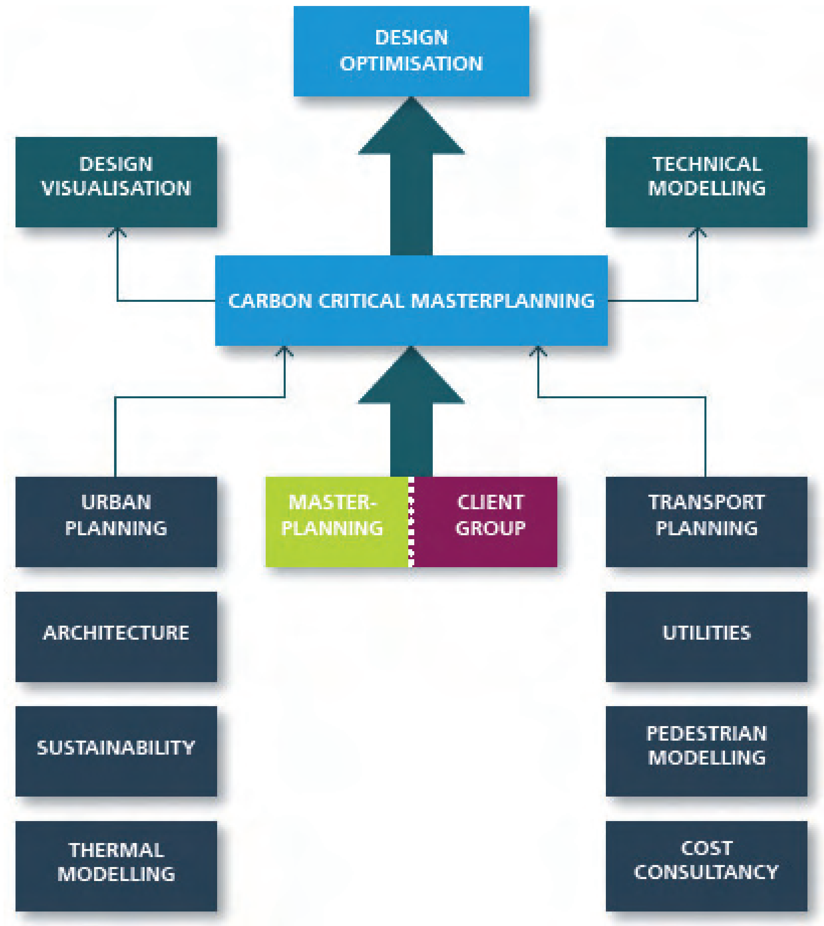

Figure 5 - Parametric design process using Carbon Critical Masterplanning

(3) What social infrastructure will I need to provide for the new population, and when will it need to be built?

(4) How can we optimise the energy use of the development to minimise the load on the local utilities networks?

(5) What will the population of this new city be? How many jobs will it create?

(6) How much waste will the development produce? What happens to the energy demands if we handle some of it on site?

Summary

Atkins' Carbon Tools were developed to place carbon optimisation at the heart of our business. By integrating the Carbon Critical Masterplanning tool with a BIM style masterplanning platform, we are able to offer carbon optimisation as a cost effective part of our core process, rather than a bolt on requiring parallel resource.

Although the implementation of the tool is in its early stages, initial feedback from our international design teams has been positive, and it is being piloted on live projects. Also fundamental to its success is its ability to enable smarter and more efficient working, allowing the masterplanning team to analyse design iterations more quickly and ultimately deliver a better service to our clients.

References

1. Mckinsey Global Institute (2001) India's urban awakening: Building inclusive cities, sustaining economic growth
2. http://www.holisticcity.co.uk

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Steven Fraser
Business Manager
Highways & Transportation
Atkins

Abstract

Reducing emissions from transport is one of the Scottish National Transport Strategy's three key strategic outcomes. On December 5th 2008 the Scottish Government published the Climate Change (Scotland) Bill, which includes a commitment to reduce emissions by 50% by 2030, and by 80% by 2050. The finalised Act also includes an interim target of a 42% reduction by 2020. These targets demonstrate a bold commitment by the Scottish Government. It signifies the importance Scotland places on playing its part in mitigating one of the most serious threats facing our world.

The Scottish Government's Transport Directorate wanted to improve its evidence based on how it can contribute to meeting emission reduction targets and appointed Atkins, in partnership with the University of Aberdeen, to undertake a study to identify, analyse and report on the policy options available to the Scottish Government. The analysis involved an assessment of individual policy options (including the ultimate production of marginal abatement cost curves including each policy) and also an assessment of central and ambitious scenarios that were formed by packaging together complementary policies.

The study has identified 22 devolved policy options that are available to the Scottish Government. A number of broad patterns have emerged in relation to the relative performance of different types of policy options. The analysis suggests that the Car Demand Management (Smart Measures) category has the greatest potential to reduce CO_2 emissions. The fiscal policy options also offer significant abatement potential, however, the analysis suggests most of the infrastructure policy options would offer significantly less potential.

The model results suggest that the combined effect of the policy options in the Central Scenario would achieve an annual abatement of around 1.35MtCO_2 p.a. in 2022, whilst the Ambitious Scenario would achieve an additional 0.80MtCO_2, representing a total of 2.15Mt CO_2 p.a. in 2022. These represent an 8% and 12% reduction respectively against projected transport emissions for that year.

Introduction

The Scottish Government and climate change

The Scottish Government (SG) published its Government Economic Strategy in 2007. This states that the purpose of the SG is to:

"focus the Government and public services on creating a more successful country, with opportunities for all of Scotland to flourish, through increasing sustainable economic growth" (The Government Economic Strategy, p1)[24].

In support of the Strategy, the Climate Change (Scotland) Act received Royal Assent on August 4, 2009. The Act sets in statute the Government Economic Strategy target to reduce Scotland's emissions of greenhouse gases by 80% by 2050, one of the sustainability purpose targets[8]. This covers the basket of six greenhouse gases (as recognised by the United Nations Framework Convention on Climate Change), and includes Scotland's share of emissions from international aviation and international shipping.

The Act also establishes an interim target for 2020 of at least 42% reductions in emissions, and allows Ministers, by order, to vary the reduction figure for the interim target based on expert advice from the advisory body. Progress towards these targets will be driven by a framework of annual targets.

The Act is a key commitment of the Scottish Government, and is the most far-reaching environmental legislation considered by the Parliament during the first ten years of devolution.

The bold targets signify the importance Scotland places on playing its part in mitigating one of the most serious threats facing our world. Figure 1 graphically illustrates the scale of this challenge.

Transport's contribution to climate change

In addition to the wider Scottish targets referenced above, reducing emissions from transport specifically is also one of the SG's National Transport Strategy's three key strategic outcomes.

In 2007, Scottish transport, including international aviation and shipping, accounted for 14.7 mega-tonnes of carbon dioxide equivalent (MtCO_2e), or 25.9% of total Scottish greenhouse gas emissions.

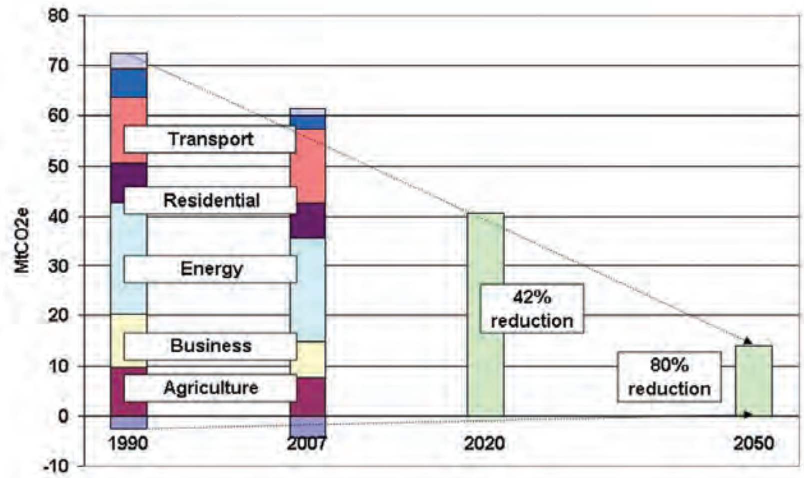


Figure 1 - The scale of the challenge

This figure, both in terms of absolute emissions and the proportion of total emissions, continues to grow on an annual basis. Surface transport is the area with the greatest emissions, with road transport alone accounting for over two-thirds of all Scottish transport emissions during the year. Figure 2 shows the growth in transport emissions by sector, and the proportion of total Scottish emissions accounted for by transport (measured on the right hand axis), since 1990.

As a result of this trend, the Scottish Government's Transport Directorate wanted to improve its evidence based on how it can contribute to meeting emission reduction targets, and appointed Atkins, in partnership with the University of Aberdeen, to undertake a study to identify, analyse and report on the policy options available to the Scottish Government. The subsequent report, Mitigating Transport's Climate Change Impact in Scotland: Assessment of Policy Options (MTCCI) will be used as an integral part of the evidence base to be considered in determining the most appropriate action for the Scottish Government to take to reduce greenhouse gas emissions from transport. In doing so, it will help inform transport's contribution to the 2010 statutory report on proposals and policies, which will formally set out how the Scottish Government overall will meet the emissions targets.

Table 1 - Devolved policy options

Sub Category		Policy Option
A	Technology	Electric car technology and network development Procurement of low carbon vehicles
B	Driving style	Active traffic management National motoring package Speed reduction on trunk roads
C	Car demand management (fiscal/ infrastructure)	Bus/rapid/mass transit infrastructure investment (including bus priority) Cycle infrastructure investment High speed rail links National network of car clubs National road user charging Introduction or increase in public parking charges Rail investment Introduction/increase in residential/private parking charges Bus /LRT fares reductions Walking infrastructure investment Workplace parking levy
D	Car demand management (amart measures)	Bus quality contracts / statutory partnerships Widespread implementation of travel plans Provide community hubs
E	Freight	Freight best practice
F	Land use planning	Urban density increases
G	Aviation	Improve public transport surface access to airports

Study methodology

Key objective:

The key objective of the study was to identify, analyse and report on the devolved policy options available to mitigate transport's climate change impact in Scotland.

Seven stages

The study took eight months to complete and was undertaken using a seven stage methodology:

(i) Establishment of a preliminary list of potential policy options

(ii) Identification of 'ownership' of options

(iii) Finalisation and filtering of the policy option list

(iv) Comparison of Scottish and UK transport use and requirements

(v) Establishment of the baseline 'business as usual' emissions scenario

(vi) Detailed assessment of policy options

(vii) Packing of complementary policy options into two alternative scenarios:

  (a) Central Scenario - a package of policy options that could feasibly be deployed with a politically or publicly acceptable degree of 'forcefulness'

  (b) Ambitious Scenario - a package of policy options, including all the measures from the Central Scenario which could be applied more forcefully, and also some policy options considered too ambitious for the Central Scenario.

Policy options

The study has identified a broad range of devolved policy options that are available to the Scottish Government to mitigate transport's climate change impact in Scotland. In total 22 policy options have been identified and divided into seven sub-categories, these are summarised in the Table 1.

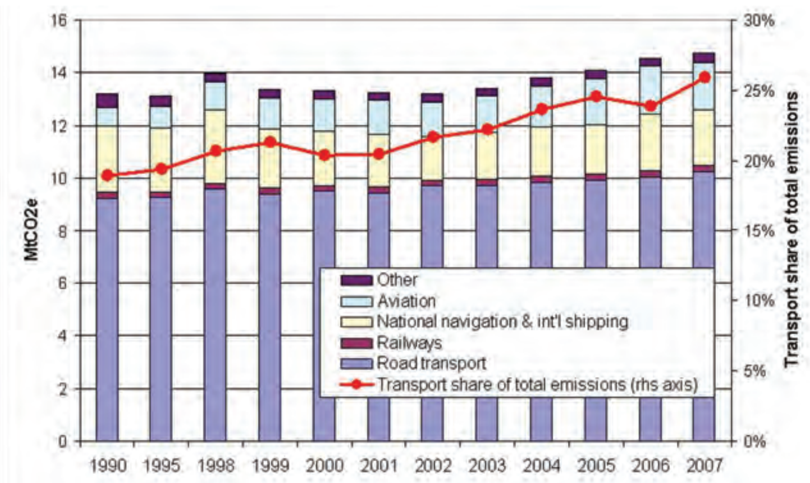


Figure 2 - Emissions from Scottish transport, 1990 - 2007

Baseline and BAU scenario

Establishment of Baseline and Business as Usual (BAU) scenario

The first stage in the detailed assessment of the potential impact of the identified policy options was to establish a BAU baseline scenario for transport carbon emissions throughout the study period. This scenario was intended to provide a reference level of carbon against which to appraise the impacts of each measure. As such it needed to take account of likely changes in key determinants of transport volumes and associated emissions. Such determinants include: economic, planning and land use changes (influencing levels and distances of travel); transport infrastructure developments (influencing levels and distances of travel); and action at the national and European level, in particular measures and legislation to instigate changes in vehicle technology and efficiency.

Table 2 presents the forecast baseline emissions for 2022 (the end of the third Committee on Climate Change (CCC) Carbon Budget timescale). For comparison, the estimates are set against the 1990 emissions out-turns (as measured in the National Atmospheric Emissions Inventory) and the consequent reduction required in order to meet the targets identified in the Climate Change (Scotland) Act.

The table also shows the further potential impact on forecast emissions levels of the measures proposed by the Strategic Transport Project Review (STPR) and national and regional action on vehicle technology represented through the supply side of the CCC's Extended Ambition scenario in their 2008 report 'Building a Low-Carbon Economy - the UK's Contribution to Tackling Climate Change'[5].

Table 2 - BAU Baseline with UK/EU measures

Source	1990	2006	2022
Cars, vans and HGVs	8.7	9.6	11.7
Bus	0.4	0.4	0.4
Rail (electric and diesel)	0.3	0.3	0.4
Aviation (domestic and international)	0.7	1.8	3.1
Shipping (domestic and international)	2.5	2.1	2.1
Total land transport	**9.3**	**10.3**	**12.5**
Target emissions required (56% of 1990 level)			5.2
Reduction required for 44% proxy target			-7.3
STPR measures			-0.3
Reduction in tailpipe emissions achieved through national vehicle technology measures			-1.3
Increase in electricity generation emissions due to national technology measures			0.5
Net required reduction to meet target			-6.1
Total transport inc shipping/aviation	**12.5**	**14.3**	**17.7**
Target emissions required (56% of 1990 level)			7.0
Reduction required for 44% target			-10.6
STPR measures			-0.3
Reduction in tailpipe emissions achieved through national vehicle technology measures			-1.3
Increase in electricity generation emissions due to national technology measures			0.5
Net required reduction to meet target			-9.5

The net effect is that to meet the target of a 44% reduction in emissions relative to 1990 in 2022 (acting as a proxy for the 42% reduction target by 2020 as specified in the Climate Change (Scotland) Act), a further reduction of around 6.1MtCO$_2$ p.a. in emissions from land transport would be required or 9.5MtCO$_2$ if total transport emissions are considered (including domestic and international aviation and shipping).

Policy assessment

Abatement potential by policy option

The annual abatement potential of each policy option varies with both scale of implementation and the year under consideration as illustrated in Table 3.

The table shows estimated annual abatement potential in 2022 (the end of the third carbon budget period) for each policy option assuming the intensity of implementation defined for each one in firstly the Central and

Table 3 - Abatement potential by policy option in 2022 (Central and Ambitious Scenarios)

Policy option	Intensity 2022 ($MtCO_2$ p.a.*)	
	Central	Ambitious
Electric car technology and network development	0.08	0.16
Procurement of low carbon vehicles	0.00	0.01
Active traffic management	0.02	0.02
National motoring package	0.11	0.17
Speed reduction on trunk roads	0.18	0.30
Bus/rapid/mass transit infrastructure investment (including bus priority)	0.01	0.01
Cycle infrastructure investment	0.05	0.12
High speed rail links	0.00	0.02
National network of car clubs	0.04	0.10
National road user charging	0.00	0.33
Introduction or increase in public parking charges	0.02	0.13
Rail investment	0.00	0.00
Introduction/raise in residential/private parking charges	0.02	0.02
Bus/LRT fares reductions	0.00	0.01
Walking infrastructure investment	0.02	0.05
Workplace parking levy	0.22	0.22
Bus quality contracts/statutory partnerships	0.15	0.18
Widespread implementation of travel plans	0.66	0.95
Provide community hubs	0.14	0.14
Freight best practice	0.09	0.09
Urban density increases	0.01	0.02
Improve public transport surface access to airports	0.00	0.01

* Rounded to the nearest 0.01Mt and including emissions generated by electricity required to power electric and plug-in hybrid vehicles

then the more Ambitious Scenario.

The figures show that the scale of abatement varies considerably from those policy options that achieve less than 0.01$MtCO_2$ p.a. of abatement in both Central and Ambitious Scenarios to several that achieve over 0.1$MtCO_2$ and travel planning which generates an estimated 0.9$MtCO_2$ reduction in CO_2 emissions in 2022 if implemented on an ambitious scale.

The difference between the central and ambitious figures illustrates the significance of intensity of implementation. More ambitious implementation in each case involves a combination of more rapid progress (so that implementation is further advanced by 2022) and/or a more stretching overall target, either in terms of geographical coverage or level of intended change. Table 3 shows that the combination of these factors has a considerable impact on the annual abatement potential with the ambitious implementation generating more than twice the abatement potential of the central implementation scale in most cases.

A number of broad patterns can be identified in the abatement figures in terms of the relative performance of different types of scheme. The forecast potential for travel planning considerably exceeds that for all other policy options, reflecting the range of approaches covered in the policy option (from workplace travel plans to individual travel marketing) and the associated scale of the target population. This will be discussed further, but suffice to say here that the way in which we have evaluated this option assumes a suite of travel planning activities will be implemented which simultaneously and comprehensively targets almost every journey purpose, social group and most geographical areas. In addition, we assume that the abatement potential is achieved in the context of supporting policies which enable and lock-in the behavioural change.

Other policy options towards the top of the list of annual potential include those that promote behaviour change through enforcement (speed limit reductions) or charges (parking charges and road user charge). Schemes involving extensive investment in the public transport network generally lie towards the bottom of the list in abatement terms.

A number of issues should be noted when interpreting the results. The estimates of abatement presented by policy option cannot simply be summed to provide an estimate of the total abatement potential generated if all policy options were introduced together. In some cases overlaps exist, for instance eco-driving assumes some compliance with speed limits on trunk routes and so there would be some double counting with enforcement of speed limits and the introduction of active traffic management. Similarly synergies are important. For example, the presence of policy options such as extensive cycling provision would be necessary to support the high levels of response forecast to policy options such as travel planning.

The figures presented should be considered as indicators of relative scale rather than detailed estimates, given the inevitable uncertainty in the forecasting process.

The impact of the policy options is strongly influenced by the underlying assumptions on vehicle technology. The estimates presented assume that national and European action have resulted in improvements in the vehicle fleet in line with an Extended Ambition scenario set out by the CCC. This results in a considerable reduction in emissions per vehicle kilometre compared to current conditions meaning that those policy options that achieve abatement through reductions in vehicle kilometres travelled or improved efficiency have less effect per vehicle kilometre than they would with the current fleet mix (between 10% and 20% less in 2022). Similarly, the abatement impacts per vehicle kilometre removed decrease through time as the fleet's average efficiency improves.

The abatement figures include an allowance for the 'rebound effect', which occurs when policy options intended to reduce emissions also reduce travel costs (through increased efficiency). The cost reduction can encourage increased travel, offsetting some of the abatement achieved. In line with the CCC report, this effect is assumed to reduce the abatement achieved by relevant measures (such as eco-driving) by 15%.

The estimated totals include emissions produced in the generation of electricity used to power electric vehicles. This is an unusual form of presentation, by convention multi-sectoral analyses (such as the CCC report) present only tailpipe emissions for the transport sector, allocating those from electric vehicles to the electricity sector. However, for a single sector analysis such as this study, it is important to include all emissions generated by transport operations, wherever they occur to provide an overall view of the net impact.

The assumptions on the energy mix used to generate the electricity powering electric vehicles and plug-in hybrids therefore influence the scale of impact of the policy options. The results presented are based on the assumptions included in the CCC Extended Ambition scenario, which assumed a continuation of the current mix of energy, resulting in a relatively high carbon emissions rate per kilometre travelled by electric or hybrid cars. If electricity could be assumed to be generated from a lower carbon source, the relative effects of the policy options would alter. Those acting to accelerate fleet turnover would have a larger effect as the emissions savings caused by switching to hybrid and electric vehicles would be greater. However, those acting to reduce demand and improve efficiency would have a relatively smaller effect in later years due to the lower emissions associated with each vehicle kilometre. Halving the assumed carbon emissions from electricity generation would reduce the abatement potential of demand management or efficiency improvements by 3% to 4% in 2022, with the impact growing in later years with the growth of the electric fleet. However, the lower emissions levels would also mean that the emissions reductions achieved by

Table 4 - Abatement potential by policy option, 2012-2030 (Ambitious scenario)

Policy Option	Annual by Year (Ambitious) ($MtCO_2$*)				Cumulative
	2012	2017	2022	2030	2010-2030
Electric car technology and network development	0.00	0.04	0.16	0.16	1.89
Procurement of low carbon vehicles	0.01	0.01	0.01	0.01	0.16
Active traffic management	0.01	0.02	0.02	0.02	0.30
National motoring package	0.12	0.24	0.17	0.06	2.86
Speed reduction on trunk roads	0.22	0.33	0.30	0.30	5.60
Bus/rapid/mass transit infrastructure investment (including bus priority)	0.01	0.01	0.01	0.01	0.17
Cycle infrastructure investment	0.04	0.10	0.12	0.11	1.84
High speed rail links	0.00	0.00	0.02	0.03	0.25
National network of car clubs	0.01	0.06	0.10	0.08	1.32
National road user charging	0.00	0.00	0.33	0.33	3.62
Introduction or increase in public parking charges	0.09	0.15	0.13	0.12	2.35
Rail investment	0.00	0.00	0.00	0.00	0.03
Introduction/raise in residential/private parking charges	0.01	0.02	0.02	0.02	0.33
Bus/LRT fares reductions	0.00	0.01	0.01	0.01	0.09
Walking infrastructure investment	0.01	0.04	0.05	0.04	0.70
Workplace parking levy	0.00	0.25	0.22	0.22	3.61
Bus quality contracts/statutory partnerships	0.04	0.10	0.18	0.28	3.05
Widespread implementation of travel plans	0.43	0.71	0.95	0.92	15.09
Provide community hubs	0.07	0.11	0.14	0.14	2.23
Freight best practice	0.07	0.11	0.09	0.08	1.72
Urban density increases	0.00	0.02	0.02	0.02	0.25
Improve public transport surface access to airports	0.00	0.00	0.01	0.01	0.05

* Rounded to the nearest 0.01Mt and including emissions generated by electricity required to power electric and plug-in hybrid vehicles

national action to promote electric/hybrid vehicles would be greater, leaving a smaller 'gap' to be met by action by the Scottish Government.

The impacts of some of the policy options are potentially understated as they are only fully implemented towards the end of the period and so their full effects do not materialise by 2030. The three key examples are road user charging (introduced in 2022) and the policy options to increase urban density and support extension of the electric car network which continue to expand in scale beyond 2022.

Year of abatement

Table 4 shows estimated annual abatement for each policy option assuming the ambitious level of implementation for each of the years of 2012, 2017, 2022 and 2030. The figures show that the potential for most policy options grows to a maximum in either 2017 or 2022 and then decreases thereafter.

This pattern is the net effect of the timescale related influences raised above which have opposing impacts on abatement levels.

- Influences acting to increase abatement potential include: increasing level of implementation - the extent to which each policy option has been implemented builds up to 100% in either 2017 or 2022 for most policy options increasing levels of reference case traffic - underlying growth in traffic is forecast meaning that each percentage reduction in vehicle kilometres achieved represents a greater absolute number and associated level of emissions.

Those acting to decrease potential include: increasing efficiency of the vehicle fleet. As outlined above, the underlying assumptions on vehicle fleet composition and efficiency (drawn from the CCC's Extended Ambition scenario) involve increasing use of more efficient vehicles such as hybrids. Consequently the average emissions per vehicle kilometre travelled reduce with an associated decrease in the scale of absolute emissions reduction achieved by a given percentage improvement in efficiency or reduction in vehicle kilometres travelled.

Mechanisms for abatement

Each policy option achieves the estimated abatement potential shown in the tables through its impacts on one or both of the following two key mechanisms for emissions reduction:

(1) reductions in the amount of travel (particularly by highway vehicles);

(2) and improvement in the emissions efficiency of travel, achieved through both vehicle technology and driver behaviour (including speed).

Reduced travel is the main abatement mechanism for several of the policy options. For example, the 0.1MtCO_2 abatement forecast to be generated by the ambitious implementation of cycling measures is largely the result of a 2% reduction in car kilometres relative to the baseline, focused particularly on roads in built-up areas (offset to an extent by increased congestion due to road space reallocation). Similarly, the 0.9MtCO_2 (in 2022) of abatement achieved by the ambitious implementation of travel plans is largely the result of a 12% reduction in vehicle kilometres (equating to a 15% reduction in car kilometres).

The short list also includes several examples of policy options for which improved efficiency is the key mechanism for abatement, particularly through alterations to driver behaviour. For example, the national motoring package achieves an estimated average reduction in emissions per vehicle kilometre of nearly 2% across the vehicle fleet, as a result of a reduction of nearly 3.5% for the car fleet.

The enforcement of a 60mph speed limit also achieves abatement through alterations to behaviour. The limit forces drivers on trunk roads to drive at more fuel efficient speeds, reducing average emissions per car vehicle kilometre by around 10% on affected roads and up to 20% for light goods vehicles (which experience a much more rapid deterioration in fuel efficiency at higher speeds). These effects result in a net reduction in average emissions per vehicle kilometres of 3% across the whole network, offset to an extent by a 0.3% increase in vehicle kilometres (due to drivers rerouting as a result of increased journey times).

Cumulative abatement

The patterns of abatement through time are also reflected in the final column in Table 4 which shows the estimated cumulative abatement potential for each policy option over the 20 year interval between 2010 and 2030.

The relative ranking of abatement potential for each policy option is broadly the same when considered in cumulative terms as when annual abatement in 2022 is considered. However, some minor reordering does occur. Those policy options that are suitable for more rapid implementation perform slightly better and those with a longer build up time perform slightly less well.

Cost effectiveness and Marginal Abatement Cost Curve (MACC)

Combining the estimated cumulative abatement potential between 2010 and 2030 with the present value (PV) of costs incurred over the same interval provides an indicator of cost-effectiveness for each policy option, defined as follows:

$$\frac{\text{PV of total capital and operating costs between 2010 and 2030 discounted to 2008 prices/values}}{\text{Estimated abatement potential between 2010 and 2030 in Mt}CO_2\text{.}}$$

This can be broadly viewed as the cost in PV terms of each tonne of abatement achieved in total over the 20 year period by each policy option and forms the basis of the Marginal Abatement Cost Curve (MACC) presented in Figures 3 and 4.

A number of points are relevant in interpreting the curve:

(i) Policy options are arranged across the graph horizontally in order of descending cost effectiveness

(ii) The width of each bar represents the broad annual abatement potential of the policy option in 2022 (in MtCO_2 pa)

(iii) The height of each bar represents the measure of the cost effectiveness of each policy option

(iv) Costs were based on implementation and ongoing operating costs only. Wider social impacts were not included and revenue gains were not offset against operating costs as they represent transfer payments (from user to operator)

(v) Given the inevitable uncertainties in both the cost and abatement measures, the figures presented should be considered as indicative of scale rather than detailed estimates.

The curve illustrates that the policy options considered vary widely in cost effectiveness from less than £10 (present value) per tonne abated (between 2010 and 2030) to over £3000. Four broad categories of policy option can be identified within this range:

(i) Those with a cost effectiveness ratio of less that £35 per tonne are largely policy options intended to affect behaviour which require the introduction of only very limited new physical assets and infrastructure (including demand management through parking charges)

(ii) Those in the next band up of less than £200 per tonne are largely policy options that require a fairly modest level of infrastructure or asset provision (charging points for electric vehicles, vehicles for car clubs and cycle provision)

(iii) Two of the four policy options in the third category of up to £1100 per tonne entail the provision of fairly complex (and therefore expensive) technological infrastructure (ATM and Road User Charging). The other two involve encouraging increased purchase rates for new, more efficient vehicles. The provision of infrastructure to encourage walking falls just above this category with a ratio of £1500 per tonne

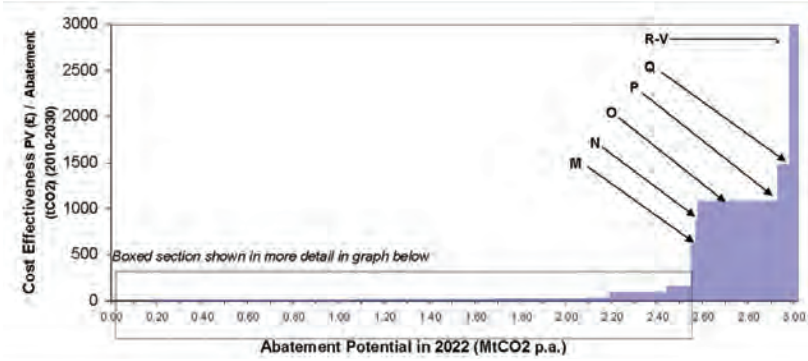
(iv) Finally, most of the policy options in the fourth category (with a ratio of over £3000 per tonne) are likely to involve the provision of significant physical infrastructure on the public transport network. The only exception is the policy option to support concessionary bus fares.

Embodied carbon

The analysis presented focuses on carbon emissions during the operation of the transport system. The embodied carbon implied in the provision of structures and infrastructure required to support each policy option has not been directly included because of the lack of available evidence and degree of uncertainty on the subject.

However, a number of general observations can be made. Several of the policy options have embodied carbon implications, particularly those involving the provision of new transport infrastructure (public transport, cycling and walking). The policy options relying on technology also imply a physical infrastructure and associated (smaller) level of embodied carbon and policy options to accelerate fleet turnover also imply the acceleration of the production of new vehicles with their embodied carbon (estimated to represent 10% of total carbon emissions associated with an average cars life span).

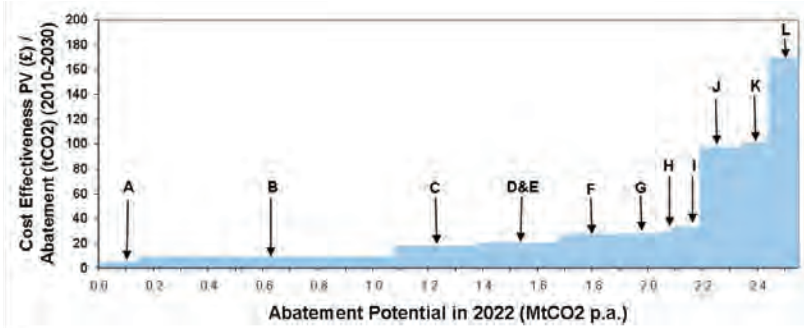
The level of embodied carbon implied by each policy option is therefore closely related to the level of supporting physical infrastructure and assets required. As noted above, the policy options already fall into cost-effectiveness bands that can be broadly described in terms of levels of supporting physical infrastructure and assets required. On this basis, including embodied carbon in the calculation of cost-effectiveness would generally tend to reinforce the ranking in the list above.



Key

Code	Measure	Measure
M	125	Introduce/ raise residential/private parking charges
N	109	Procurement of low carbon vehicles
O	99	National road user charging
P	1	Active traffic management
Q	131a	Walking Infrastructure Investment
R	205c	Improve public transport surface access to airports
S	15	Bus/rapid/mass transit infrastructure investment (including bus priority)
T	127	Bus/LRT fares reductions
U	75	High speed rail links
V	115	Rail investment

Figure 3 - Marginal Abatement Cost Curve - Ambitious implementation of policy options



Key

Code	Measure ID	Measure
A	204	Provide community hubs
B	173	Widespread implementation of travel plans
C	143	Speed reduction on trunk roads
D	103	Introduction or increase in public parking charges
E	98	National motoring package
F	172	Workplace parking levy
G	18	Bus quality contracts / statutory partnerships
H	158	Urban density increases
I	63	Freight best practice
J	53	Electric car technology & network development
K	97	National network of car clubs
L	37	Cycle infrastructure investment

Figure 4 - Marginal Abatement Cost Curve - Most cost effective policy options - detailed view

Abatement potential by scenario

The scenarios tested were built up from combinations of policy options identified and analysed above, implemented at either central or more ambitious levels of intensity. However, as discussed the cumulative abatement effect of the scenario cannot be viewed as simply the combined effect of each of the policy options included as interlinkages, overlaps and synergies that exist between them.

Allowances have been made for these effects in modelling the estimated combined effect of the policy options in each scenario, and on this basis the model results suggest that the policy options in the Central and Ambitious Scenarios would achieve significant abatement, generated through both of the key mechanisms identified above for the individual policy options (reduction in travel volume and improved emissions efficiency of travel).

For instance, the Central Scenario is forecast to cause a reduction in vehicle kilometres of approaching 10%, focused particularly on car kilometres. The equivalent estimate reduction caused by the Ambitious Scenario is 15%, again largely the result of reductions in car kilometres.

Viewing impacts from an efficiency perspective, the impact of the Central Scenario is to reduce average emissions per vehicle kilometre by around 5% across the road network, again focused particularly on a reduction in emissions from cars. The equivalent reduction for the Ambitious Scenario is 8% (largely resulting from reduced car emissions).

These combined effects would achieve an estimated combined annual abatement of approximately 1.35MtCO_2 p.a. for the Central Scenario in 2022. The Ambitious Scenario would achieve an estimated additional 0.80MtCO_2, representing a total of 2.15MtCO_2 p.a. in 2022.

The estimated abatement potential of the Central Scenario therefore accounts for approximately 15% of the difference between the Baseline emissions (including action at the EU/UK level) and the proxy 2022 level of a 44% reduction from 1990 total transport emissions. The contribution is approximately 25% if the comparison is restricted to emissions from the land transport modes targeted by the scenario. The equivalent figures for the Ambitious Scenario are just over 20% of the target difference for 2022 if all transport emissions are considered, and 35% if the focus is restricted to land transport alone.

As highlighted for individual policy options, the estimated abatement potential of the scenarios should be considered as indicative and are dependent on a range of factors, including: the assumed scale of implementation and response to each option, the inclusion of a rebound effect (of 15% for relevant options), the inclusion of emissions produced in generating the electricity required for electric vehicles; and the assumption of the energy mix used to generate the electricity.

Varying these assumptions would alter the results presented. For instance, if it was assumed that the rebound effect could be avoided through extra measures, the abatement potential of the Ambitious Scenario would increase by nearly 5%.

In contrast, assuming that electricity was generated from lower carbon energy sources (producing half of the level of CO_2 per KWh assumed in the main tests), the abatement potential would actually slightly reduce (by 2%). This reflects the fact that more of the abatement potential in the main scenario test is derived from reducing travel by electric vehicles than is derived from increasing the proportion of travel undertaken by electric vehicles. However, lower carbon electricity generation would increase the abatement potential of the national measures (to a net total of 1.1MtCO_2, as opposed to 0.8MtCO_2 otherwise), reducing the size of the 'gap' to be met by devolved action.

Abatement beyond 2022

The modelling results suggest that the total abatement potential from the Ambitious Scenario will be very similar in 2030, although the balance between the contributions from different policy options will have changed. For instance, those policy options focusing on efficient driving will have become less significant (as the vehicle fleet becomes increasingly dominated by electric and hybrid vehicles) and those with longer term effects (such as land use planning) will become gradually relatively more significant.

Forecasts of emissions levels and the impact of abatement policy options over the longer term to 2050 inevitably have to be less detailed than those for shorter timescales due to the uncertainties involved in attempting to forecast travel patterns, behaviour and technology in 40 years' time.

Conclusion

It is possible to anticipate future trends that are likely to be important. The key influence is expected to be the anticipated increasing use of electricity to power the vehicle fleet, either directly or through the production of hydrogen. Sources such as the CCC report and the King Review[25] suggest that such vehicles could feasibly be the standard by 2050.

In this case, emphasis will increasingly be on energy policy and technology and the nature and viability of the electricity network and vehicles rather than the direct reduction in emissions from vehicle exhausts. Suitable vehicle technology and the provision of very low carbon electricity (generated for instance by renewable energy) could potentially result in very low transport carbon emissions levels. However, the role of supporting transport policy options will remain important. Although some of the policy options assessed about will become less relevant as they are related to current technology (particularly those encouraging more efficient driving), the emphasis on improving efficiency and reducing demand will continue to be important. This will potentially be aimed less at reducing carbon emissions directly and more at ensuring demand remains at a level and in a form that could be viably served by both the transport and electricity networks.

Bibliography

1. Advanced motorway signalling and traffic management feasibility study (2008), a report to the Secretary of State for Transport, Department for Transport.
2. Allen H. (2007) Energy savings and increased passenger satisfaction with eco-driving: examples from signatories to the UITP Sustainable Development Charter, Public Transport International.
3. Anable, J. & Bristow, A. L. (2007), Transport and Climate Change: Supporting document to the CfIT report, Commission for Integrated Transport.
4. Atkins, (2003) High Speed Line Study, Milestone 7: Transport Case, SRA.
5. Building a Low-Carbon Economy - the UK's Contribution to Tackling Climate Change (2008), Committee on Climate Change.
6. Cairns et al. (2004) Smarter Choices – changing the way we travel, Department for Transport.
7. Carbon Savings from Active Travel Interventions (2008), Sustrans Report to the CCC, Sustrans.
8. Climate Change (Scotland) Act (2009), The Scottish Government.
9. ECMT 2007 (2008), Cutting Transport CO_2 Emissions - What Progress?
10. Written submission to Scottish Parliament from RMT.
11. Gower, P., Shearn, S. and Mitchell, J. (1998). Motorway tolling: Modelling the impact of diversion, TRL Report TRL. 349, Transport Research Laboratory.
12. Gross et al. (2009) What policies are effective at reducing carbon emissions from surface passenger transport? A review of interventions to encourage behavioural and technological change. UKERC.
13. Halden, D. (2006) Scoping the Impacts on Travel Behaviour in Scotland of E-Working and other ICTs.
14. Low Carbon Transport Innovation Strategy (2007), Department for Transport.
15. Low Carbon Vehicle Procurement programme – discussion paper (2007), Department for Transport.
16. Myers, D. and Cairns, S. (2009) Carplus Annual Survey of Car Clubs 2008_ final draft report. Transport Research Laboratory.
17. Nottingham City Council (2008), Workplace Parking Levy Business Case.
18. Scenarios and Opportunities for Reducing Greenhouse Gases and Pollutant Emissions from Bus Fleets in PTE Areas (2008), Study Report for PTE.
19. Scotland's National Transport Strategy (2006), The Scottish Executive.
20. Scottish Association for Public Transport (2008) - High Speed Rail in Britain
21. North South Line Carbon Impact: BAH Report, Department for Transport.
22. Scottish Transport Statistics (2008), The Scottish Government.
23. STPR Summary Report (2008), Transport Scotland.
24. The Government Economic Strategy (2007), The Scottish Government.
25. The King Review of Low Carbon Cars. Part II: recommendations for action (2008), HM Treasury.
26. UK Air Passenger Demand and CO_2 forecasts (2009), Department for Transport.
27. Update of UK Port Demand Forecasts to 2030 and Economic Value of Transhipment Study (2007), Department for Transport.
28. Wardman, M., Tight, M., & Page, M. (2007) Factors influencing the propensity to cycle to work, Transportation Research Part A: Policy and Practice.
29. Wiegmans, B. W., Beekman, N., Boschker, A., Van Dam, W., & Nijhof, N. (2003), ICT and sustainable mobility: From impacts to policy, Growth and Change.

CARBON CRITICAL DESIGN

UAE Holistic plan: Kadra power infrastructure development

Dr Vinesh Thiruchelvam

Senior Electrical Engineer

Atkins Middle East

Abstract

The United Arab Emirates (UAE) has been far sighted in its approach towards developing an infrastructure that meets the demands of the future. The expansive urban development in the UAE has caused the power demand to increase and there will be an upwards shift for many years to come. Although the population pressures differ between the seven Emirates, Abu Dhabi and Dubai lead the way for current demands being met.

The concerns, however, have shifted to the Northern Emirates of Sharjah, Ajman, Umm al-Qaiwain, Ras Al Khaimah and Fujairah, which are all in the throes of developing and expanding their infrastructure to facilitate residential and tourist development. In this technical paper, we take a specific look at the Kadra development located in the Emirate of Ras Al Khaimah. The current Kadra catchment would be considered as a simple village and it will move into a sustainable community living zone by 2015. This is the vision of the central government as part of its target for urban development.

Introduction

Approximately 97% of the UAE's electricity production is fuelled by natural gas, with the remaining three per cent produced by diesel generation and steam turbines (primarily in the Northern Emirates). The major players in the UAE's electricity generation sectors are Abu Dhabi Water & Electricity Authority, which currently accounts for 53% of the capacity; Dubai Electricity and Water Authority (DEWA) (29%); Sharjah Electricity and Water Authority (SEWA) (11%) and the Federal Electricity and Water Authority (FEWA), which operates in the Northern Emirates (7%).

The escalating scale of urban development is impacting significantly on the demand for electricity. The demand for electricity in the UAE has been growing at double digit rates for many years and according to all estimates it will continue to do so until at least 2020. Although they are not experiencing quite the same population pressures as Abu Dhabi and Dubai, the Northern Emirates of Sharjah, Ajman, Umm al-Qaiwain, Ras Al Khaimah and Fujairah are all in the throes of developing and expanding their infrastructure to facilitate residential and tourist development.

In fact, the UAE and Saudi Arabia have the highest projected increase in demand within the Gulf Co-operation Council (GCC) region, which is expected to continue to grow at a minimum rate of 10% per annum, far outstripping the world average of 3% per annum.

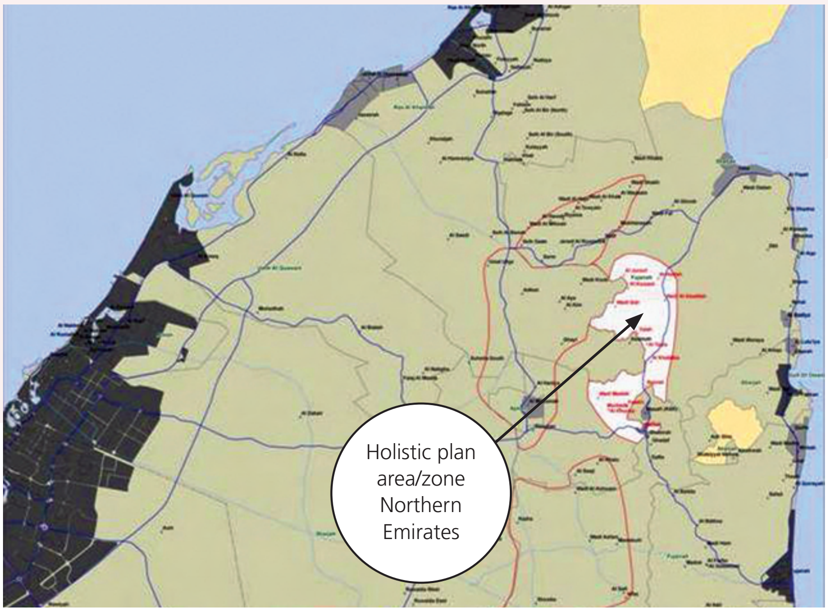


Figure 1 - Holistic development zone

There is currently an insufficient capacity of power generation in some parts of the UAE. Combined with the non-existence of the national grid, this has resulted in the shortage of electricity in certain areas. Some of the new real estate development projects and industrial zones have not been able to secure sufficient power supply and have had to resort to power generators fuelled by diesel or gas. Security of energy supply is an area of increasing visibility and acknowledged importance for maintaining the UAE national economy and welfare. The UAE itself has a growing energy demand.

Assuming primary energy resources are available in full capacity to cater for the upcoming demand, secure power distribution into future developments, such as the Kadra Catchment Area in the Northern Emirates, will enhance the town's growth. Interruption of service of this power infrastructure may cause inconvenience or harm to the Kadra community and economy. It may also snowball into other systems that depend on continuous electricity supply. This approach of secure power is essential in development of Kadra not only in terms of the population growth but also from an industrial growth or a sustainable living point of view.

UAE power distribution - Fujairah, Ras Al Khaimah and Sharjah

The current power supply distributed into the Fujairah and Ras Al Khaimah region predominantly is from two main power stations:

(i) 1150MW via 400kV transmission network lines distributed from Taweelah Power Station (ADWEA-Abu Dhabi) into Dhaid 400kV Substation.

(ii) 861MW via 132kV transmission network lines distributed from Qidfa Power Station (FEWA-Fujairah) into Ras Al Khaimah and Fujairah regions including the 132kV substations at Dhaid and Tawain.

The general distribution within a region or emirate is traditionally at 132kV unless it is direct from the power station at 400kV. This is via overhead transmission lines. From a 132kV substation, the power is stepped down to a medium voltage of 11kV which is then distributed to any local development. Within the towns or local housing areas the 11kV is further stepped down to 400V usually via pocket substations. The 400V low voltage is distributed into each household distribution board and for industrial in most cases it is a direct 11kV supply. For medium or low voltage the distribution is normally done via power cables direct buried in the ground.

FEWA energy sourcing

In 2008, Abu Dhabi Water & Electricity Authority (ADWEA) signed a deal with FEWA to supply it with increasing imports of electricity, reaching up to 2,500MW by 2015. Electricity demand in the Emirates covered by FEWA peaked at 1,840MW in the summer of 2009, 3% higher than the maximum consumption of 1,790MW recorded in 2008. Peak exports to FEWA from Abu Dhabi in the summer of 2009 were about 909MW, compared with 758MW in 2008.

FEWA has about 1,120MW of generation capacity, but most of the power plants are small, ageing and inefficient.

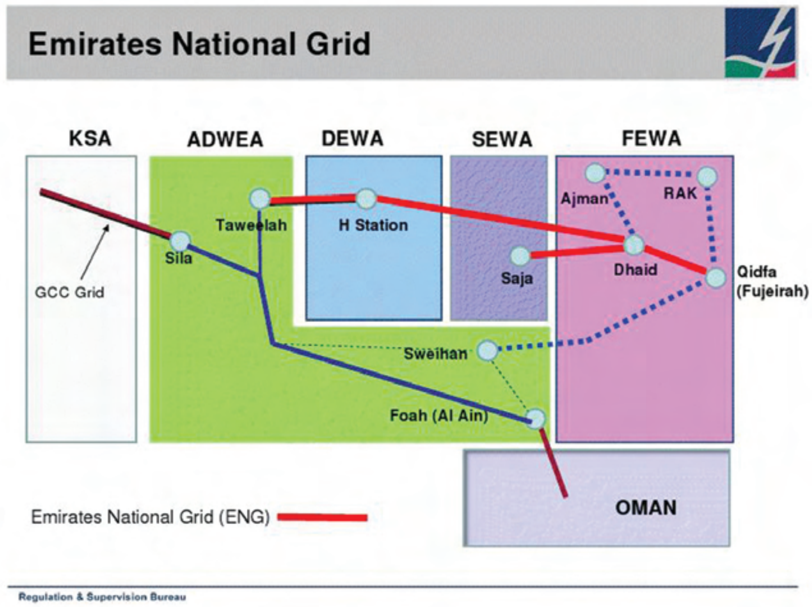


Figure 2 - Emirates National Grid (Regulation & Supervision Bureau, 2009)

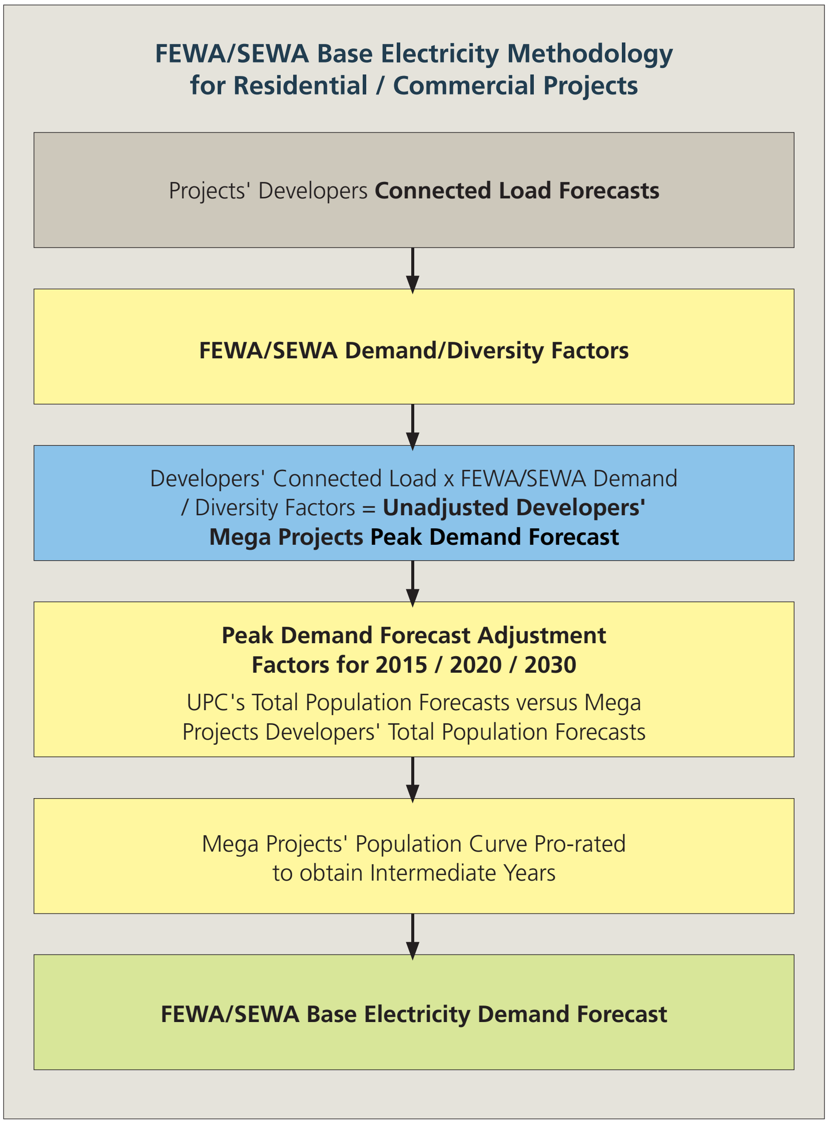


Figure 3 - FEWA power development methodology

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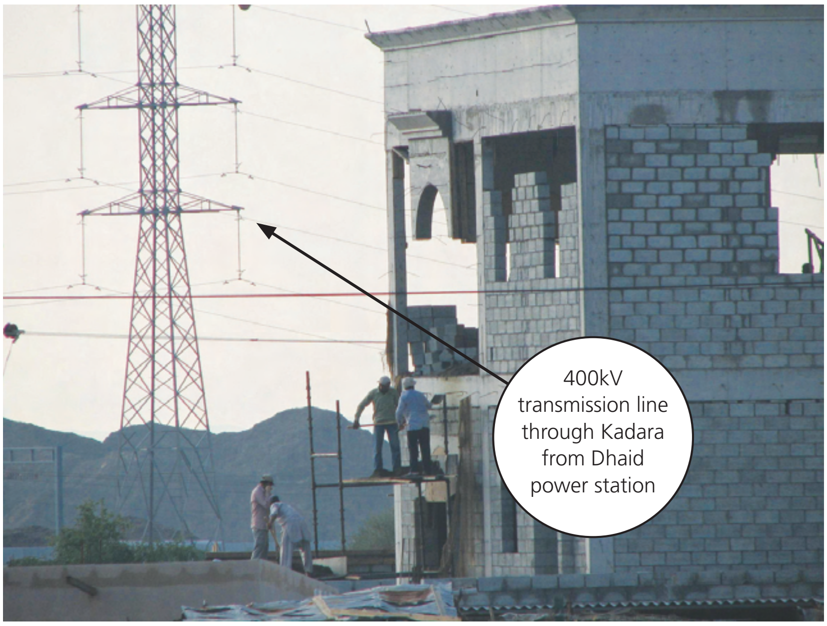

Figure 4 - Existing distribution to and in Kadra

Since Abu Dhabi is the main contributor to the UAE federal budget, and the federation funds FEWA, the UAE government has taken the view that it makes economic sense to export electricity produced by Abu Dhabi Water & Electricity Company's (ADWEC's) more modern and cost-effective power plants to FEWA.

Most of the electricity supplied to FEWA and SEWA by ADWEC is generated at the 890MW Fujairah 1 power and desalination complex at Qidfa. A second plant, Fujairah 2, which will also supply the Northern Emirates, is under construction nearby. It will have a capacity of 2,000MW and is scheduled to start up in late 2010. A new 240-kilometre-long pipeline is being installed to take Qatari Gas from receiving facilities at Taweelah in Abu Dhabi to Fujairah to fuel the facility. Figure 2 illustrates the distribution flow direction within the Emirates National Grid (ENG).

At the moment, Union Water & Electricity Company (UWEC) will also be supplying up to 500 MW of power to FEWA's network and ultimately to the ENG network. The ENG project, set up by the Abu Dhabi Government in 2001, is being jointly undertaken by UWEC, ADWEA, DEWA, FEWA and SEWA.

A new greenfield power generation and water production plant, adjacent to the existing ADWEA Fujairah F1 plant, is being implemented as an IWPP at Qidfa near Fujairah on the Gulf of Oman coast. The project was scheduled to be completed by July 2010 with total net power generation capacity of 2000MW. This is an example of how the utility board is providing for the growth within the Northern Emirates region. Figure 3 is the typical flowchart of how FEWA formulate their methodology against the growing energy demand in the region.

Future demand assessment

Based on the development plan for Kadra, an electrical energy demand was projected from 2010 till 2030 in five phases. Projection was done for the residential units and all public service units. The demand was calculated based on the total built up area using locally benchmarked watts/m² norms. The final projected demand for 2030 is calculated as 40.28MW. This was based on conventional cooling and not district cooling. To cater for this estimated load a 132kV substation will be built. The location is to be finalised in agreement with the utility provider FEWA. The preliminary estimation is a requirement for 60m x 40m plot area. The power output capacity of the substation is 42.5MW and this allows for 2.22MW spare power availability. The substation will be built based on the N+1 system. It is estimated that the transformers will be sized at 25MVA and three units will be housed in the substation. In the next stage of the study diversity application will be detailed further. The expected diversity target will be at 0.7 which should give the development an After Diversity Maximum Demand (ADMD) of approximately 28.2MW.

Considerations and appropriate options

From the site visits conducted (Figure 4) and communications with the local residents it was apparent that the power received is continuous but they do experience power cuts mainly during heavy rain. As such an investigation will be needed on this matter in relation to the network quality to and within Kadra.

The new site power distribution will be integrated with the extensive on-site generation systems to enable the import and export of power under differing load and environmental conditions. The following voltages will be provisionally utilised for the power distribution network design throughout the Kadra development:

132,000 Volts - (132kV)
High Voltage (HV)

11,0000 Volts - (11kV)
Medium Voltage (MV)

400 Volts - (0.4kV)
Low Voltage (LV)

FEWA will provide supplies to the site at 132kV from an existing nearest 400/132kV sub station. The supply will be provided via underground cable circuits to the perimeter of the site and will terminate in a new 132/11kV sub station.

The power infrastructure for the future development will include the following:

- Cable corridor reservations for 132 kV cabling from the FEWA network to the 132kV substation plot
- Corridor reservations and network infrastructure for the MV reticulation from the 132kV substation to the 11/0.4 kV pocket substations throughout Kadra
- Corridor reservations and network design for LV services (400 volts) supplying street lighting, pumping stations, residential units, telecoms equipment, traffic control equipment and any other infrastructure facilities.

ENERGY

The pocket substation consists of a bunded base to allow the placement of the transformer and outgoing LV switchboard that is placed directly on the bund and then covered with glass-reinforced plastic (GRP) housing that is positioned over the top of the equipment providing a basic shield from direct sunlight, rain, sand and unauthorised personnel. The LV switchboard provides 400V supplies to villas' distribution boards and also to feeder pillars that reticulate to local supplies such as for the street lighting or any public service buildings.

Community integrated energy

It is possible to save energy by generating and converting energy for electricity, cooling and heating at a neighbourhood level where the urban density is high enough, rather than the building level. Some parts of the UAE have seen considerable application of district cooling that has demonstrated good energy savings, provided more flexibility at development/building level and has resulted in lower health risks.

Thermal energy storage can easily be incorporated to reduce peak cooling and electrical loads. The next technical development is likely to be district cooling using interconnected grids under municipal ownership where providers all compete for custom on the basis of efficiency and therefore breakdown potential monopolies. Such technology could be well suited to the Kadra, and it could be implemented in a phased manner either as a public or private venture.

Issues and options

Issue No. 1

The main issue at the moment for this study is the clarity of power availability within the existing Kadra area. Information is still lacking from FEWA to formulate future requirements.

Options for Issue No. 1

- To progress further discussions with both FEWA with the assistance of the client to bring awareness to the authorities on the Kadra development
- To use current demand estimation as a benchmark for the 132kV substation design and Atkins to plan and locate the substation as commercially and strategically possible.

Issue No. 2

The client has requested a study on the feasibility of micro power generation. A more detailed study needs to be undertaken to collate meteorological data to justify application feasibility available via the relevant research bodies. The evaluation of renewable energy and suitable technologies is included in this study under the various disciplines covered.

Options for Issue No. 2

Renewable energy technology offers a realistic power source alternative (or supplement). The energy technologies termed as 'renewable' are often up for debate. In general, renewable energy does not provide consistent power supply as it is typically dependent on energy sources that peak and wane (i.e. sunshine, wind, rainfall, etc.) hence renewables tend to suit a power grid whereby renewable energy systems augment a primary power source. Convention is a solution whereby renewables generate a proportion of the developments energy.

For the purposes of this assessment we have highlighted the applicable/feasible areas for micro generation which are as listed below:

- Photovoltaics (PVs) - solar energy

  There are two forms of renewable energy that use sunlight directly.

  (i) Solar photovoltaic cells (solar- or PV cells) convert radiation from the sun into electricity

  (ii) Solar thermal panels produce hot water using sunlight absorbers.
- Wind turbines - wind energy

Issue No. 3

In an integrated power system, rural electrification is challenging for two reasons. Firstly as large capital expenditures are required to connect remote areas due to the distance to be covered through overhead lines, connecting remote areas with small consumption might prove uneconomical. This effect is amplified when taking into account transmission and distribution losses because both tend to increase with the distance covered. Rural electrification such as to Kadra is thus costly. It often proves more economical to rely on distributed generation in such cases. This has often been the case for mountain areas or low density areas remote from the main cities.

Option for Issue No. 3

One option to minimise the energy demand and incorporate a distributed network would be in terms of the cooling methodology for Kadra. To meet this requirement district cooling would seem to fit. District cooling is centralised production and distribution of cooling energy. Chilled water could possibly be delivered via an underground insulated pipeline to residential buildings to cool the indoor air of the buildings within Kadra. Specially designed units in each building/villa then use this water to lower the temperature of air passing through the building's air conditioning system.

The output of one cooling plant is enough to meet the cooling-energy demand of many of the buildings. District cooling can be run on electricity or natural gas, and can use either regular water or seawater as the means to reject the heat from the cooling generation process. Along with electricity and water, district cooling constitutes a new form of energy service.

District cooling systems can replace any type of air conditioning system. This air conditioning system is subject to a difficult operating environment, including extreme heat, saline humidity and windborne sand. The feasibility of district cooling should be further investigated at the next study stage.

FEWA discussion

With regards to upstream power distribution, the utility board will do the network analysis to tie in with the existing network. This is a normal process and should be kept that way. With co-operation and assistance of the client, a joint discussion with FEWA will be undertaken to address the demand for the new development. Discussion with FEWA will also allow for an assessment on the following elements that could be implemented:

ENERGY

Figure 5 - Typical wind turbines in the desert area

- Consideration of a gas infrastructure versus electrical infrastructure
- Possibility of sharing existing 132kV substation, if load is reduced
- Assumptions on unit energy consumption for villas
- Investigate the possibility of smart e-grid distribution with FEWA
- Consider peak demand versus off peak demand and method of control (dynamic demand management)
- Metering and monitoring strategy - Smart/Intelligent metering.

Micro power generation

Wind power generation

Wind power generation could be a possibility if the energy towers, see Figure 5, are strategically located in relation to the highest wind catchment point. This requires a separate plot of land to assemble and erect the wind energy towers. A site specific feasibility analysis would be required to determine the viability of wind power. This study would determine year-round wind speeds, direction and frequency. Wind power technology (wind driven turbines) could potentially be installed on the site to augment the primary power supply.

Solar energy for homes

Given the high number of sunshine hours in the Kadra area, solar power is a realistic power source. Solar power is most commonly generated through photovoltaic cells, alternatives include solar thermal power and solar concentrators.

One of the biggest problems with PV panels in hot climates is that performance drops off significantly above 90°C. The other problem is dust; without regular cleaning a thick layer of dust soon builds up on any horizontal surface in the UAE, see Figure 6. A key drawback of solar power is the large amount of land area required to generate significant amounts of power. As mentioned above, it is unrealistic to target 100% of the development demand from renewables. This would require an enormous land area and therefore is discounted. Smaller solar farms could supplement the primary power source.

Solar power does not have to be isolated in one area within the development to work. An alternative or in addition to bulk renewable energy, is to integrate renewables into the urban fabric or into specific sites and rail/road designs. PVs can be installed on the roof of any building or villas in particular. Typically, an array is incorporated into the roof or walls of a building/villa, and roof tiles with integrated PV cells can now be purchased. Alternatively, an array can be located separate from the villa but connected by cable to supply power for a group of villas. This is simplified as the small solar farm concept.

Figure 6 - Typical solar farm in the desert

Solar street lighting

PV application can be utilised for the street lighting installation. The most recent developments with regard to the technology behind solar street lighting have been in connection with Light Emitting Diodes (LEDs). Firstly, these consume far less power than the older type of conventional sodium lamp. In addition, they have a much longer working life, better colour definition and require smaller solar components than sodium lamps. It can be appreciated therefore, that in order for solar street lights technology to be efficient and cost-effective, a thorough and meticulous assessment has to be carried out, paying the utmost regard to information concerning solar radiation, the amount of sunshine prevalent and general climatic conditions for Kadra. This needs to be done hand-in-hand with careful consideration of the exact requirements of the street lighting system. In other words, the number of hours of illumination required on a daily basis and other important data have to be precisely determined in order to provide the client with the system that will meet all those criteria. If these items are all properly assessed, then there will be the ability to provide an efficient, reliable, economically sound, purpose-built solar street lighting network.

Recommendations

Based on current information, energy demand current or existing growth is 15.61MW in 2010 increasing to 40.28MW in 2030. The 20 year development and population rise is in parallel with a 61% energy demand maturity growth. In line with the energy demand growth the following is recommended:

(i) The energy demand will need to be further verified with the final development plan for Kadra. At this early masterplan stage the capacity of the network connection must encompass design development and changes during the construction period of this project. It is recommended that initial negotiations with the FEWA are based on a demand range of between 45-50MW

(ii) A new 132kV substation may need to be built with location and upstream network tie-ins to be coordinated with FEWA as required

(iii) Some of the basic meteorological data will be obtained, such as the kWh/m² of horizontal irradiation for solar energy and the annual wind mean velocity to study renewable energy possibilities

(iv) Recommend a detailed 'Community Integrated Energy' approach to meeting the developments power demand requirements, including (among others) integration of demand reducing schemes; incorporating renewables into the urban fabric (and power grid) and recycling and recovery of energy. We recommend a detailed study is commissioned to develop this exciting opportunity further

(v) For street lighting, further feasibility on type of lights and the integration will have to be discussed with the infrastructure designers.

The five points above, when undertaken, give the overall energy demand study considerable decision making technical data towards the next phase of development study. It also allows for a thorough database and benchmark setting for the correct design path.

It is recommended that the following three main steps be executed towards a sustainable development:

(i) Load reduction

(ii) System optimisation

(iii) Renewable substitution.

Load reduction is a passive design strategy which will be studied further in the next study stage incorporating part of the IDP. Some possible elements to study further are:

- Controlled daylight supply replacing artificial lighting
- Change of set point to >24°C instead of 22°C
- Buffer spaces and adjusted set point depending on use
- Most energy efficient equipment
- Reduced water consumption.

Optimising supply systems is part of energy demand strategies and the element that will be studied further is the intelligent metering with energy saving incentives

In addition to the above MoPW will have to raise awareness in the Kadra community of consumer initiatives towards saving energy and sustainability.

References

1. Regulations and Supervision Bureau, UAE Vision 2030, Pg 17, 2009
2. RAK Green Building Guidelines, Ras Al Khaimah Municipality, 2010.
3. ADWEA Demand Manual, Pg 23, 2010.

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David Knowles
Chief Engineer
Energy
Atkins

Michael Sheridan
Lead Engineer
Energy
Atkins

Oliver Montgomery
Lead Engineer
Energy
Atkins

Nick Terry
Mechanical Engineer
Energy
Atkins

This paper describes a programme of work undertaken to assess the integrity of stainless steel boiler tubes in both their current state and after tube-plugging. Tube-plugging is potentially damaging as tubes in this particular boiler are coupled together by "detuning straps", potentially amplifying tube wall stress due to differential thermal expansion. Assessments have been carried out to R5 volume 2/3 Appendix 4, incorporating proposed updates to the procedure in consideration of welds. Assessments were undertaken for tubes in their current, unmodified state. Predicted damage levels were lower than the action level, and hence it was considered unlikely that tube-plugging will be required. Assessments were also undertaken with each tube plugged in-turn. The damage levels in tubes coupled to the plugged tube ranged from acceptable to in excess of the action level. When tube-plugging produced higher levels of damage, assessments were undertaken of simulated cutting solutions, to identify effective remedial actions.

Introduction

This paper describes the final phase of a three phase project. An overview of the two preceding phases of work is provided for context in this section.

During commissioning of a thermal power station boiler, concerns were raised regarding excessive resonance of the boiler tailpipes (serpentine steam-tube bundles in the boiler, shown in Figures 2 and 4). The vibration was believed to be excited by the blade frequency of the gas circulators. To reduce this vibration, "detuning straps" were welded between adjacent boiler tailpipes (see Figure 1).

The straps have a rectangular cross section and are fabricated from grade 316H stainless steel. In the majority of cases straps are welded onto a weld pad, which is, in turn, fillet welded to the surface of the tailpipe.

In the event of a tube leak in such a boiler, the normal procedure is to plug the effected tailpipe (and therefore the two boiler tubes it feeds), which has the effect of causing the metal temperature to rise to the surrounding gas temperature due to the loss of its steam flow. As the plugged tube is then unpressurised, the increase in metal temperature is not significant to the plugged tube itself.

However, the differential thermal expansion between the hotter plugged tube and the cooler adjacent tubes creates a bending moment in the detuning straps, potentially increasing tube wall stresses above their normal operating stress level. Any increase in local strain range imposed by the detuning straps will lead to higher accumulated creep-fatigue damage. Consequently, a leaking boiler tube cannot be plugged without considering the requirement for cutting a de-tuning strap, or combination of straps, to relieve potentially high stresses.

ENERGY

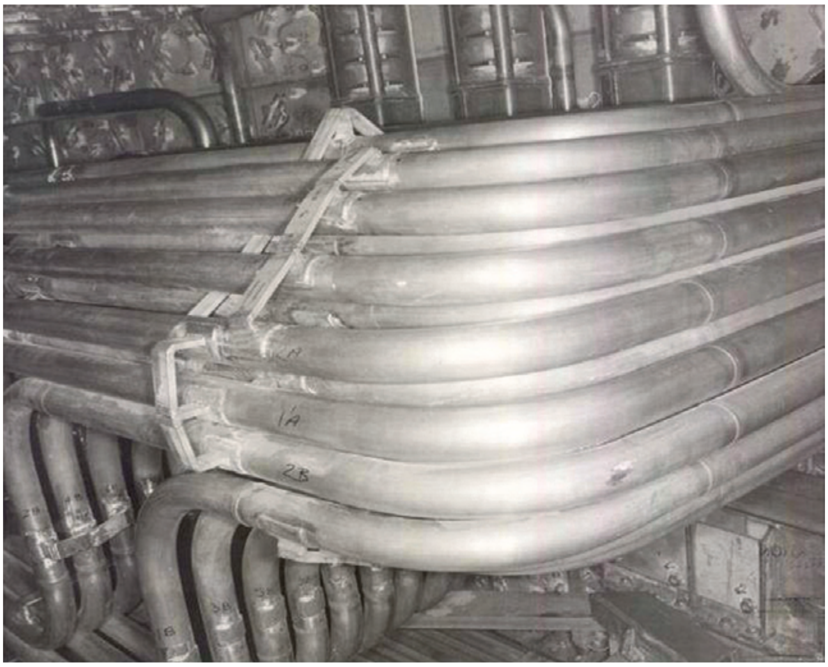

Figure 1 - Photograph of boiler tube bundle with detuning straps

The first two phases of the project were scoped to investigate which straps could practicably be accessed for cutting, given the situation where a tube fails and must be plugged. Phase 1 of this project was to create a complete 3D model of the boiler inlet and outlet, all the de-tuning straps, and boiler components. The model was created using CATIA (see Figure 2). Phase 2 of the project imported the model into Jack, a purpose specific human modelling software package. Using Jack, a human manikin, sized using standardised anthropometric values and encumbered with appropriate clothing and tools, can be simulated to carry out cutting tasks on each de-tuning strap (see images in Figure 3). The output of phase two is a complete list of every de-tuning strap that can be accessed for cutting manually, if required, to relieve tube stresses in the event of tube plugging.

The final phase of the project was to undertake an assessment to identify, given a tube failure of any given tube, whether cutting a de-tuning strap was required, and if so, whether cutting the de-tuning strap would sufficiently reduce stress levels. To do this it was necessary to evaluate the creep-fatigue damage that has accumulated in an unmodified (normal operation) tube to the current time and then the change in damage rate if an adjacent tube was plugged. These results can be used to ascertain whether in-vessel remedial work is required, and whether any proposed remedial work would sufficiently reduce predicted damage levels. This paper describes the methodology employed to carry out the creep-fatigue damage assessments.

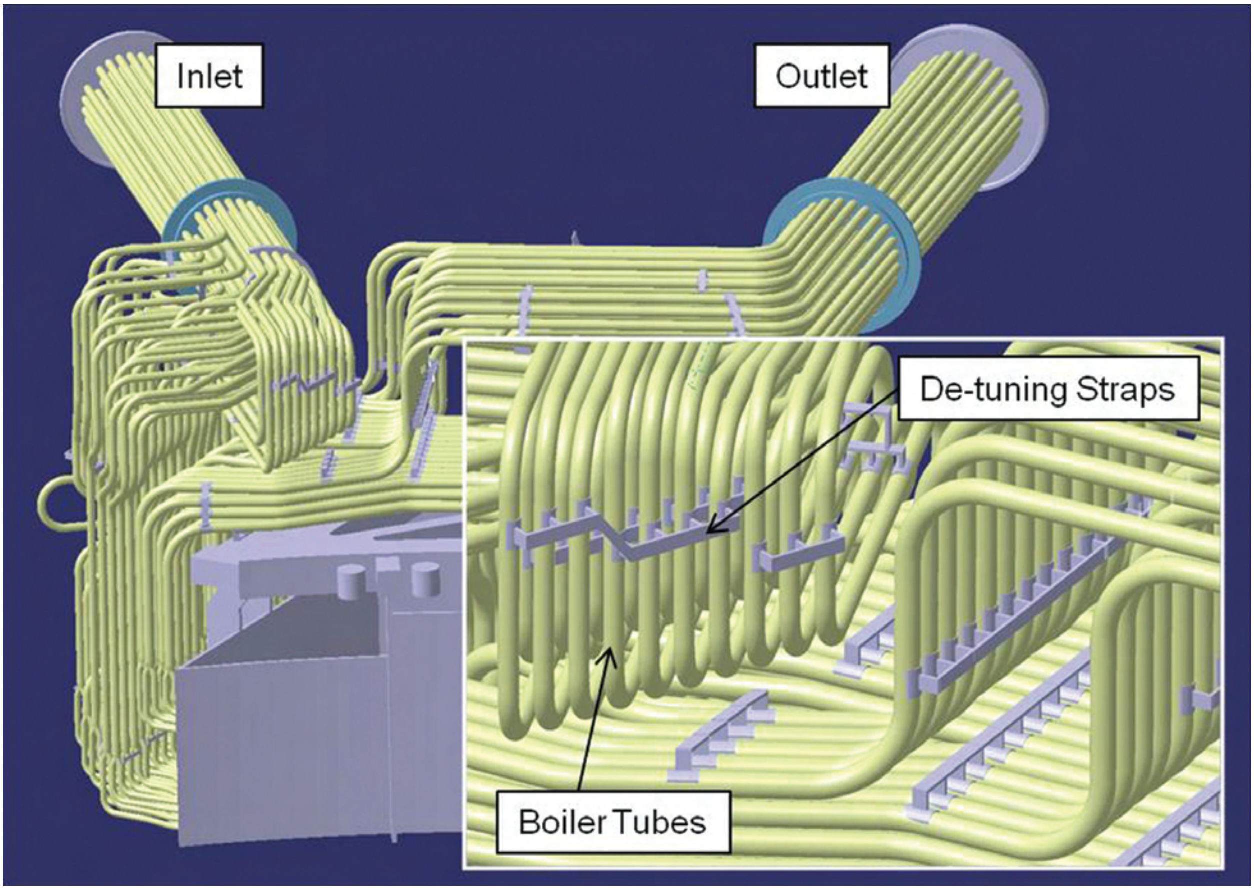

Figure 2 - CATIA model of the boiler bank (Inset – detail of boiler tubes and de-tuning straps)

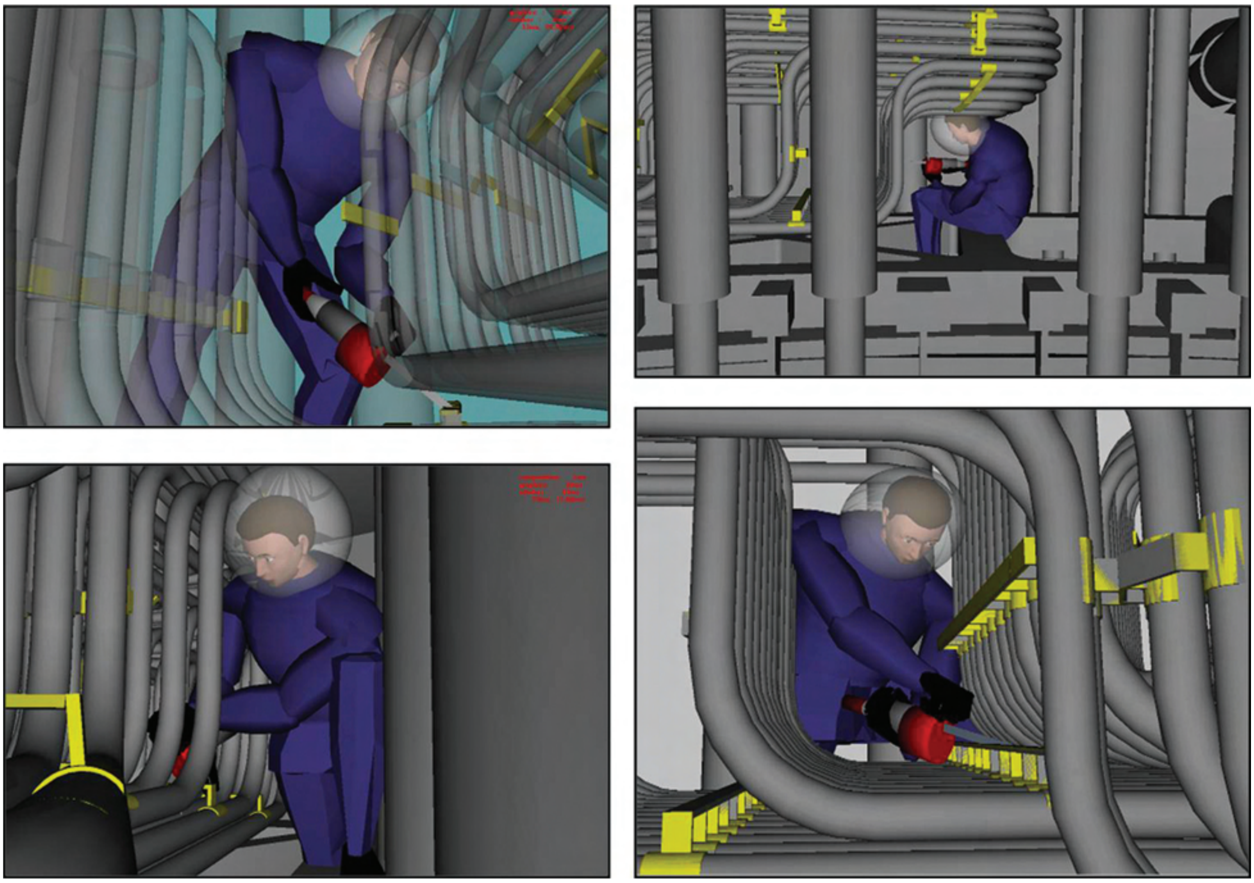

Figure 3 - Jack Simulation: Human manikin shown in various poses, assessing cutting accessibility to various de-tuning straps.

Finite element modelling of the Boiler Bank

Global beam-element models have been produced using Abaqus/CAE (version 6.9) of the complete boiler bank inlet and outlet (inlet shown in Figure 4). The boiler bank inlet and outlet consists of 62, 316H stainless steel, "tailpipe" tubes (referred to as tailpipes) which bifurcate to connect via 124 tubes which pass through the main gas pass. The global models are used to assess the global linear-elastic response of the boiler during normal operation, with a plugged tube, and also operation following strap/tube cutting.

The tailpipes, boiler tubes and bifurcations are represented using pipe (PIPE32) elements. The detuning straps are represented using beam (B31) elements. The effect of tailpipe plugging is modelled by raising the plugged tube metal temperature to the outside gas temperature. Strap cutting is modelled by removing the relevant strap elements from the model.

Detailed local models of the strap connection were produced using solid hexahedral elements to extract assessment stresses for specific strap/tailpipe connections. Like the global models, the local models employed linear-elastic analysis. They consist of a short extent of tailpipe, the detuning strap weld pad, a short extent of the strap, and each fillet weld (Figure 4).

The boundary conditions for the local models are extracted from the global model. In addition, other loads on the local model (e.g. internal and external pressure and gravity) are applied.

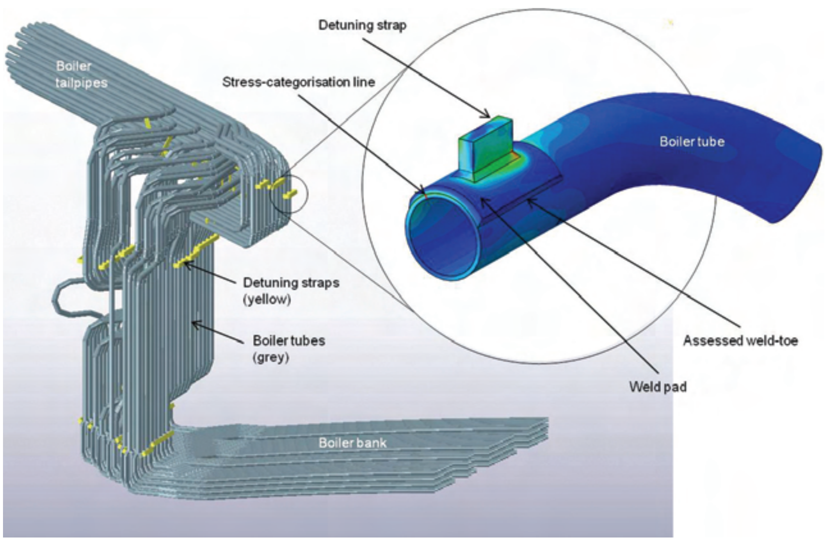

Figure 4 - 'Global' beam-element model of the boiler bank inlet (grey) and 'local' solid-element model of a de-tuning strap location (blue)

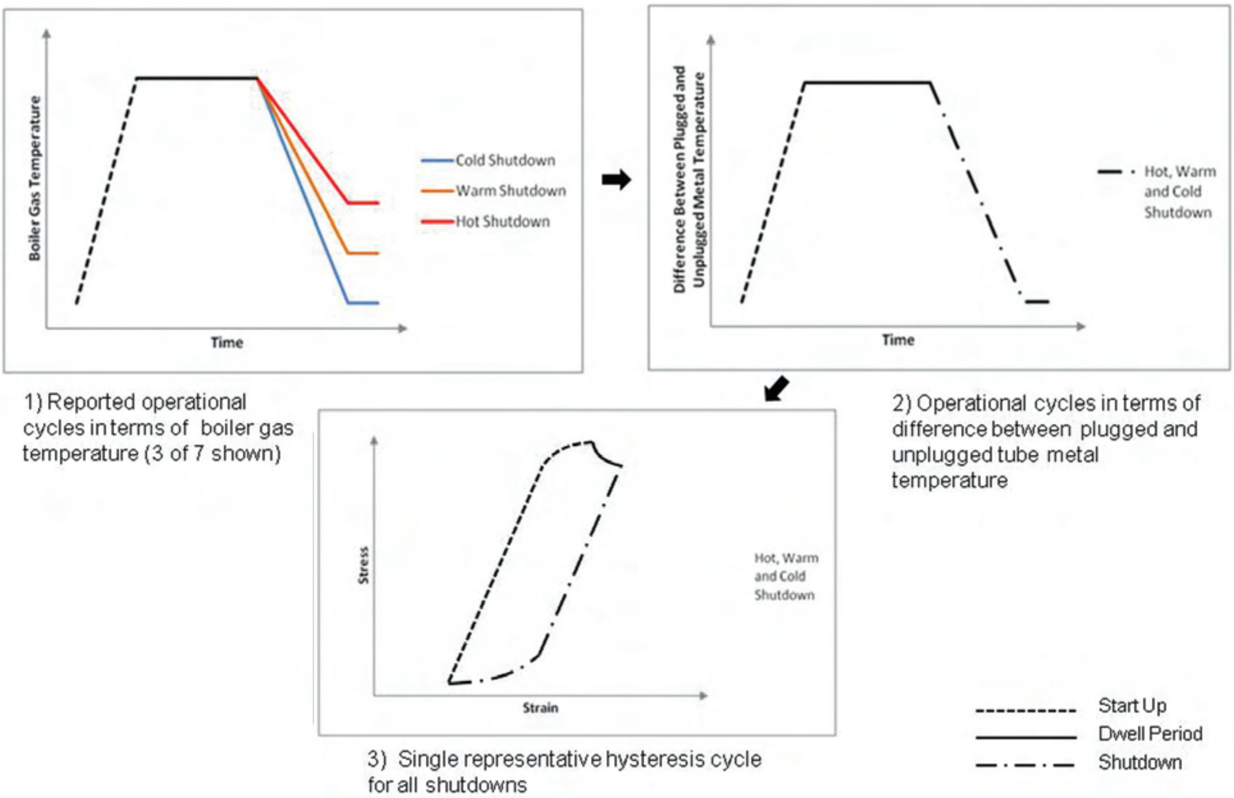

Figure 5 - Creation of representative hysteresis loop

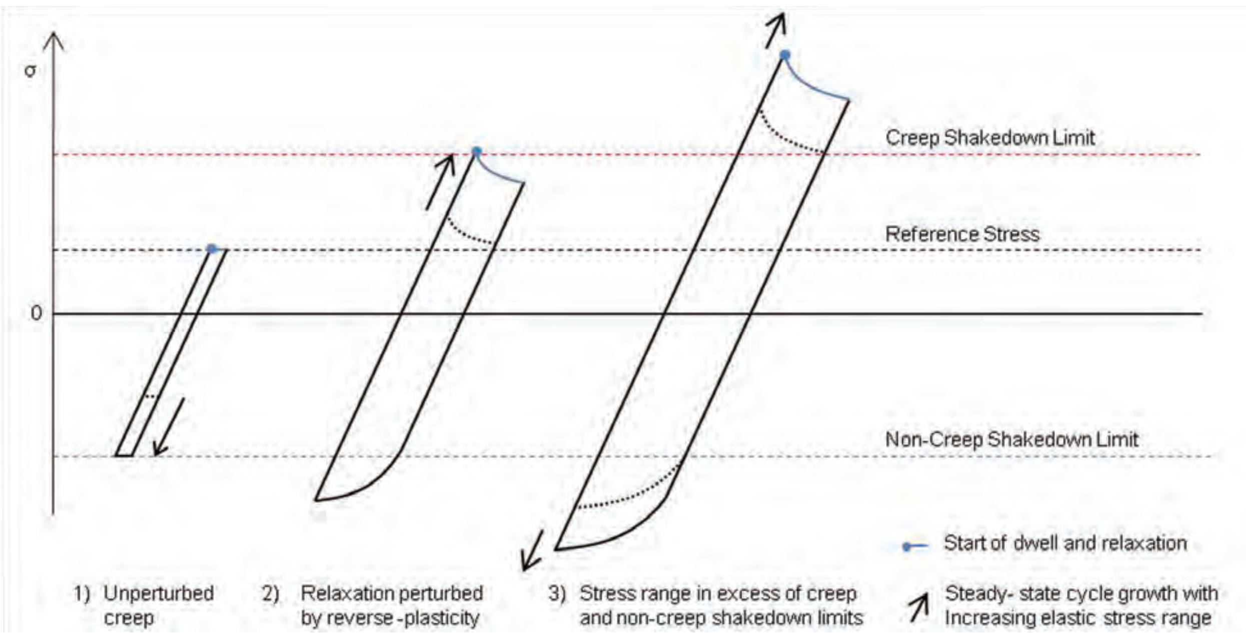

Figure 6 - Three R5 shakedown criteria

Assessment cycles

Historically, damage assessments of boiler components have been based upon seven shutdown cycles and two load change cycles. For simplicity, the shutdown cycles and the load change cycles have been related to the change in boiler gas temperature during the cycle. Boiler gas temperature is continuously logged and periodically all the cycles are identified from the data, tallied and reported for use in damage assessments.

The reported cycles were reconsidered in this assessment to ascertain whether they could be used directly to count the in-phase thermo-mechanical stress cycles. It was found that the reported cycles required re-evaluation for this assessment. This was due to the fact that the stress range was primarily driven by the difference in metal temperature between plugged and unplugged tubes, not by the magnitude of boiler gas temperature alone. Furthermore, it was shown that each of the seven categories of reported shutdown cycle accumulated similar damage when considering a boiler with a plugged tube; this feature is illustrated with simplified diagrams in Figure 5. Hence, for this assessment there was only the need to consider one (bounding) representative shutdown cycle. The occurrence of any reported shutdown category therefore resulted in the assessment of one additional representative cycle.

There was also a major and minor load change cycle to consider. Finite element analysis showed that both cycles generate relatively low stress ranges. These were considered during the assessment to see if they had any effect on the accumulated damage and thus if they required inclusion. As described later, it was found that they do not perturb the shutdown cycle relaxation and hence did not affect damage in the creep dominated regime.

Implementation of R5

The creep-fatigue crack initiation assessment undertaken herein follows the procedures in R5 Volume 2/3 Appendix 4 for undressed welds[1]. However, O'Donnell[2], issued in 2005, proposes updates for the treatment of weldments which will be present in the new revision of the procedure. They aim to reduce some of the conservatisms in the methodology and have been applied in this assessment, along with a comparison made to the historic approach.

In practice there are only few changes to the procedure, but they are significant in reducing damage levels. The Fatigue Strength Reduction Factor (FSRF) applied to amplify the local strain range has been divided into a weld strength enhancement factor (WSEF) and a weldment endurance reduction (WER). The FSRF for undressed fillet welds (type 3) has been replaced by the WSEF with an associated reduction in the strain enhancement value from 4 to 1.66 for the same weld type, resulting in a significant reduction in the assessed inelastic strain range. The WER is applied to the fatigue endurance data and is expressed by removing the number of cycles to nucleate a surface crack (to size a_i) from the base-line fatigue endurance curve. This is assumed on the basis that the weld inherently contains defects, of size ai, at the time of fabrication.

As recommended in O'Donnell (2005)[2], the assessments were based on mean cyclic stress-strain data, with conservatism built in to the creep and fatigue damage assessments through the use of lower bound creep ductility and fatigue endurance data. Creep ductility, fatigue endurance and all other material property data were taken from the AGR Materials Data Handbook R66[3].

This assessment was not concerned with a crack growing through the weld itself, as failure of the weld will simply release the detuning strap and not release steam. The considered crack is one which grows through the tube wall, causing a leak; hence parent material properties were used throughout.

The stress range for the damage assessments was taken at the weld-toe of the weld pad, at the point of maximum equivalent stress, from the local finite element model. Linearised stresses were extracted using a stress categorisation line (SCL), positioned at the weld-toe and orientated perpendicular to the tube surface (Figure 4).

The R5 procedure is based on the determination of a steady-state hysteresis loop. The first step was to adjust the elastic stress range to take account of plasticity. A Neuber transformation was used to evaluate the inelastic stress, based on the Ramberg Osgood representation of a cyclic stress-strain curve. The strain was then enhanced by the WSEF and a smaller enhancement made to take account of volumetric differences from uni axial test data. The equivalent inelastic stress range was then calculated, again based on the Ramberg Osgood relationship. Before the dwell period could be assessed, the start of dwell stress was modified by repositioning the inelastic stress range with respect to one of three shakedown criteria (Figure 6).

The rupture reference stress was calculated using the primary reference stress. The tube parent material is taken as creep ductile at failure and so the reference stress is derived from Equation (1). To take account of the stress raiser at the weld-toe, the concentration factor, χ, is conservatively assumed to be 1.66, equivalent to the WSEF.

$$\sigma^{R}_{ref} = \{1 + 0.13[\chi - 1]\}\sigma_{ref} \qquad (1)$$

One of two separate creep relationships were used to calculate the creep relaxation accumulated during the cycle dwell period, both dependent on the start of dwell stress. If the start of dwell stress was above 125MPa then the Feltham relationship was used to calculate stress drop as a function of time, independent of strain. The relationship does not consider primary and secondary creep discretely and, as it implicitly determines stress reduction with time, it does not invoke a hardening law. When the start of dwell stress was below 125MPa, the RCC-MR forward creep strain constitutive relation was implemented, because the Feltham relations were not available for a start of dwell stress below this value. The relation was arranged to allow implementation of a strain hardening law. Primary and secondary creep was calculated discretely, with the switch over occurring when secondary creep rate exceeded that of primary.

It was assumed that the stress relaxation characteristic during the dwell period re-primes for every cycle, i.e. creep strain in the constitutive relationship returns to zero, thus promoting primary creep. This would not always be the case, as some assessments were within strict shakedown and unperturbed by creep. However, re-priming did occur at stress ranges approaching the limit of acceptable damage level, and so re-priming was assumed by way of a conservative approach for all assessments. The basis for this assumption was founded on the analysis, O'Donnell (2005)[2], which demonstrated increased accuracy in forward creep laws when primary creep is reset to zero at the start of each cyclic dwell period for temperatures above 550°C and for large stress ranges. During the dwell period, if the stress relaxed to the rupture reference stress, the stress was held at the rupture reference stress until the end of dwell, i.e. stress was not allowed to fall below that expected due to primary loading.

Table 1 - Damage fractions for most highly stressed unmodified tube

Assessment period	**Damage from initial cycles**		**Cyclic damage per assessment period**		**Total cumulative damage to end of assessment period**
	Initial shakedown	Relaxation of residual welding stress	Creep	Fatigue	
Commissioning to 2009	0.074	0.090	0.113	~0	0.277
2009 to end of a/c life			0.0535	~0	0.331

Elastic follow-up (Z) was applied to creep strain sustained from the relaxation of high stresses. Z is defined as the creep strain increment divided by the elastic strain decrement. Using an approximate power law creep relation, Z was calculated by FEA as 1.16, though in line with common engineering practice, a value of 3 was conservatively chosen for the assessment.

Creep and fatigue damage fractions for a steady-state cycle were evaluated in the assessment. Fatigue damage per cycle was calculated using Equation (2) where N_0 is based on lower bound fatigue endurance data and adjusted for the crack initiation size (10% of wall thickness). The total strain range used with the fatigue endurance data was a summation of the inelastic strain range plus the additional creep strain, modified by Z, incurred during the dwell period. Creep damage per cycle was evaluated using a ductility exhaustion approach using Equation (3). In this assessment, $\bar{\varepsilon}_f$ was taken to be independent of strain rate based on data from AGR data handbook[3].

$$d_{fatigue} = \frac{1}{N_0} \tag{2}$$

$$d_{creep} = \int_0^{t_h} \frac{\dot{\bar{\varepsilon}}_c}{\bar{\varepsilon}_f(\dot{\bar{\varepsilon}}_c)} dt \tag{3}$$

It was conservatively assumed that all stress states are primarily bi-axial, defined in R5 as a stress state where the first principal stress is less than twice the second principal stress. The effect of a biaxial stress state was represented by halving the creep ductility and therefore effectively doubling creep damage.

In addition to the assessment of creep-fatigue damage accumulated each cycle, the damage incurred during the first few operating cycles due to shakedown and residual weld stress relaxation were also calculated as one-off damage fractions. The shakedown damage was found by calculating the total strain accumulated due to the stress relaxation from start of dwell stress in the initial cycle to the start of dwell stress in the steady-state cycle. A creep ductility exhaustion method was used to calculate the accumulated damage, where $\bar{\varepsilon}_f$ was assumed to be independent of strain rate at failure. The first cycle start of dwell stress was calculated using Neuber to translate elastic stress-strain to the Ramberg Osgood monotonic stress-strain curve. A bounding damage fraction has been calculated to take account of the relaxation of potentially high residual weld stress. The damage fraction was conservatively calculated assuming that the stress relaxation due to residual stress is equal to the yield stress, including consideration of elastic follow-up. A creep ductility exhaustion method was used to calculate the accumulated damage, again assuming that $\bar{\varepsilon}_f$ is independent of strain rate at failure.

Results and discussion

The total strain ranges and cycle numbers were relatively small at the weld-toe of the detuning strap weld pad and the fatigue damage accumulated per cycle was found to be insignificant in comparison to the creep damage. Creep therefore dominated cyclic damage. The level of stress relaxation calculated by the Feltham Law and RCC-MR law naturally correlate to the start of dwell stress which was found to be dominant in determining the creep strain and thus creep damage.

In the R5 procedure, the steady-state cycle can shakedown to one of three criteria (Figure 6) depending on the inelastic stress range. In this situation where creep dominates damage, the shakedown criterion becomes significant as it determines the start of dwell stress and hence the assessed damage.

When the stress range was low, and shook down to criterion one, the start of dwell stress was determined by the rupture reference stress, itself based on the primary stress. All assessments within criterion one developed insignificant damage due to the low primary stresses. The stress range started to influence tube life when it forced the steady-state cycle to enter a reverse plasticity regime, and moved to shakedown criterion two. In this regime, any increase in elastic stress range directly increases the start of dwell stress, and thus had a significant impact on the damage. Within this criterion, as the stress range increased, the assessed damage increased from insignificant to unacceptable levels.

It is of note that shakedown criterion two, which defines the significant damage levels in a plugged boiler, was not affected by the rupture reference stress and hence the primary stress. This allowed certain simplifications to be made in bounding acceptable stress levels.

The damage results for unmodified tubes were within shakedown criterion one, and hence the damage results, shown in Table 1, are low. If a tube was plugged at the time of the assessment, strap connections which developed elastic stress ranges of less than 160MPa would be limited to a tolerable damage fraction by the end of the current accounting life.

Table 2 - Effect of employing the updated strain enhancement (WSEF) on shakedown criterion and damage limit

	Assessment with original FSRF for undressed fillet weld = 4	Assessment with updated WSEF for undressed fillet weld = 1.66
Maximum allowable stress range that will shakedown to shakedown criterion 1	51MPa	124MPa
Allowable elastic stress range	67MPa	160MPa

This stress was used to bound acceptable strap connections in terms of the requirement for strap cutting.

In this type of assessment where the steady-state cycle positioning was very significant, the benefits of employing the updated WSEF as opposed to the historic FSRF were amplified. The more representative WSEF demonstrated that a higher stress range could shake down to criterion one and therefore accumulate insignificant creep damage in this assessment. It also dramatically reduced the enhancement of strain range within shakedown criterion two, significantly reducing start of dwell stress and hence damage levels. This is shown in Table 2. For this assessment, if the standard FSRF had been used, the results would have been much more conservative and many of the unmodified tubes would have had substantial predicted creep damage.

The results were used to validate the acceptability of tube-plugging and remedial cutting solutions on the basis of acceptable damage accumulation by end of accounting life. It was shown that of the 62 tail-pipe tubes, 34% can be plugged without causing the requirement for remedial in-vessel work. Simple, manual cutting solutions were identified for 37% of the tubes using current in-vessel cutting techniques (identified using Jack). The remaining 29% of tubes required more complex cutting solutions.

Conclusion

R5 Volume 2/3 Appendix 4 creep-fatigue crack initiation assessments for weldments have been undertaken on boiler tubes at detuning strap weld pad locations. The updates to Appendix 4 proposed by O'Donnell[2] were incorporated into the assessment procedure and these proved much less conservative than the standard code approach.

Assessment of unmodified tubes calculated low creep-fatigue damage levels such that damage would not be expected to reach the action level before the end of the current accounting life.

The damage accumulated in the boiler tubes at the detuning strap locations was dominated by cyclic creep damage. Primary stresses were low and this resulted in insignificant damage accumulation in all assessments with a steady-state cycle which was unperturbed by creep. Damage levels from steady-state cycles conforming to a shakedown regime which re-primed the start of dwell stress ranged from insignificant to significant levels. The damage from steady-state cycles which perturbed creep relaxation was dominated by the stress range and WSEF.

References

1. R5, Creep-Fatigue Crack Initiation Procedure for Defect-free structures, Issue 3, 2003.
2. O'Donnell M P, Proposed changes to R5 Vol 2/3 A4: Treatment of Weldments, E/REP/BDBB/0067/GEN/05,rev.0,QA Grade 4, British Energy, 2005.
3. BEGL, AGR Materials Data Handbook R66, Rev. 007.

Assessment of a bridge pier pile foundation subjected to bearing replacement

Omar Hamza

Senior Lecturer

Apello University

Angela Bellis

Principal Geotechnical Engineer

Water & Environment

Atkins

Abstract

This paper reports on the geotechnical aspects of highway maintenance work, which has been planned recently for A14 Orwell Bridge - Pier 5. The pier (one of nineteen) is supported by a group of 50 bored piles founded in chalk. Because the bearings were to be replaced for one side of the pier resulting in additional forces from a temporary supporting system erected on one side of the pile cap, a geotechnical assessment of the pier foundation was required. The investigations included the back analysis of a pile load test using PLAXIS to evaluate the engineering parameters of the chalk and the pile bearing capacity calculations, and hence helped to reduce the uncertainty usually encountered in the estimation of skin friction in chalk. To check for any potential for overstressing or excessive rotation, the load deformation response of the pile group under the new working load conditions was analysed using two different types of software: PIGLET and REPUTE. This paper provides simple procedures to assess an existing pile group in chalk.

Introduction

Bridges in the UK are routinely maintained by replacing their roller bearings when they have reached the end of their serviceable life. A14 Orwell Bridge has been subjected to this type of maintenance by the Highways Agency. The bridge shown in Figure 1 carries the A14 over the River Orwell just south of Ipswich in Suffolk, England. The construction of the bridge commenced in October 1979 and was opened to road traffic in 1982. The bridge is a 1.3km long, 24m wide, 18 span, continuous post-tensioned concrete twin box-section structure and supported on 19 piers[1]. Defects were initially observed on roller bearings during an inspection in 1992, resulting in their replacement. A remote monitoring system was set up in 2003 to detect any excessive movement and bearing deterioration at the bridge. Further maintenance work is planned at Pier 5; the pier (indicated by the arrow in Figure 1-(i) comprises two columns (north and south), which share piled foundation. Bearing replacement will be required only at the north column of Pier 5. A temporary steel frame is to be erected on the pile cap to support the bridge loads of the North deck while replacing the bearings[2].

Figure 1 - (i) Overview of Orwell Bridge, Pier 5 is indicated by an arrow

Figure 1 - (ii) Photo taken at Pier 5, where the height of the pier from the top of pile cap is 34.5m

Although the additional load is estimated to be less than 15% of the total existing loads applied on the foundation, the distribution being on one side of the pile cap may significantly increase the existing stresses under this side.

To ensure the safety and serviceability of the foundation under the new temporary supporting system (i.e. during the proposed bearing replacement) a geotechnical assessment is required to predict any potential over-stressing or excessive settlement under the piles. In the initial stage of the assessment, existing geotechnical information was collected from the as-built records[1] and reviewed on the basis of knowledge and experience reported in literature.

Ground investigations conducted for the bridge have shown that the site comprises various superficial deposits underlain by an extensive layer of upper chalk. The properties of these materials, used for the existing pile design, were evaluated against a historical pile load test, which was back analysed using PLAXIS-2D[3]. The analysis of the pile load test also provided a tool to evaluate the bearing capacity estimated for a single pile and particularly in reducing the uncertainty in estimating the skin friction of the chalk.

To calculate the deformations and load distribution among the piles in the group, under the 3-dimensional working load conditions, the load deformation response of the pile group was analysed. The pile group was modelled first using PIGLET software[4] adopting a linear elastic model for the ground. To ensure that the elastic assumption was reasonable at the load and deformation levels, a further check using REPUTE software[5] was performed.

Methodology

The method, carried out to assess the effects on the foundation system under the temporary loading system (produced by the proposed bridge maintenance e.g. steel work) included a number of steps, which can be summarised as follows:

- Desk study exercise to collect and review the existing geological and geotechnical data for the site
- Identify the pile group arrangement at Pier 5 from as-built records[1]
- Evaluate the material properties used for the assessment by conducting back analysis on a historical pile load test using finite element method in PLAXIS
- Estimate the bearing capacity and settlement of the piles
- Compare the outputs from above with that given in the 'as built record' and identify any significant difference
- Conduct structural/matrix analysis of the pile group subjected to the new loading system. This part of the assessment was achieved using two different commercially available software packages. PIGLET was used to check the elastic response of the system. Then the effect of plasticity of the ground materials on the analysis was checked by REPUTE software
- The axial forces in piles obtained from above were compared with the allowable bearing capacity of the piles. It is believed that by this step any critical condition that may be induced by the proposed temporary supports on the foundation can be identified.

Existing information

Geology of the site

The subsoil information used to establish the ground model was obtained from various sources, including:

- Ground investigation data used for the construction of Ipswich Bypass and Ipswich Southern Bypass Orwell Bridge reported in 1976 and 1979 by Sir William Halcrow & Partner[6, 7]
- As-built construction drawings and documents (1997-1984)[1]
- Recent site investigation conducted for Pier 9 remediation[8]
- In addition to the British Geological Survey 1:50,000 – Series, Sheet 207 Solid and Drift for Ipswich 9 and British Regional Geology – East Anglia and adjoining areas[10].

The information provided by the existing ground investigation was found generally adequate to establish the subsurface conditions for the purpose of the proposed assessment. A walkover survey and a number of trial pits[11] were conducted to confirm the pile cap size and provide a schedule of ground parameters that may be required to design a temporary access road.

According to the available data, it appears that the geology of the broader area has the arrangement of a typical river valley as shown in Figure 2, where a geological cross section has been plotted from a number of historical boreholes on the west side of the river. The geological sequence is the same on both sides of the valley and consists of a sedimentary succession of River Terrace Deposits, London Clay, Lambeth Group and Thanet Sands[12].

Table1 - Geological profile at Pier 5 of Orwell Bridge

Unit	Discretion	Thickness of the unit (m)
Topsoil	Soft to firm fine grained sandy gravelly Clay with occasional soft organic material	0.3
Made Ground	Orange brown gravelly, silty Sand with occasional pockets of stiff sandy clay	1.2
Alluvium	Soft silty sandy Clay	1.60
Sands & Gravels	Generally medium dense sand or silty gravelly sand with occasional pockets of stiff clay.	0.5
Chalk	Grade V to III	30+

The whole sequence has been deposited on upper chalk and eroded from the centre of the valley where the river currently flows and now comprises alluvium overlying glacial granular soil underlain by the chalk.

In the immediate vicinity of Pier 5, the ground investigation data have shown that 0.3m of topsoil is underlain by about 1.5m of granular made ground, which has been used to bring the soil to 0.4m above the pile cap level. Below the made ground, the natural geological sequence comprises alluvium deposits, glacial sands and gravels overlying the upper chalk. The ground summary at Pier 5 including the description of the geological units, their average thicknesses are given in Table 1.

The chalk is described as highly weathered Grade V at the top 15 metres and becoming Grade IV/III at deeper locations. According to the classification of chalk proposed by Ward, Burland and Gallois (1968)[13], Grade V is used to describe 'structureless remoulded chalk containing lumps of intact chalk', while Grade III is used for 'rubbly to blocky unweathered chalk'. Further description of grades and classes of chalk is presented in CIRIA report -C574[14]. Although the chalk formation is known in other regions to be subjected to dissolution features, there is no evidence at the surface of such features within the study area, nor from the ground investigation.

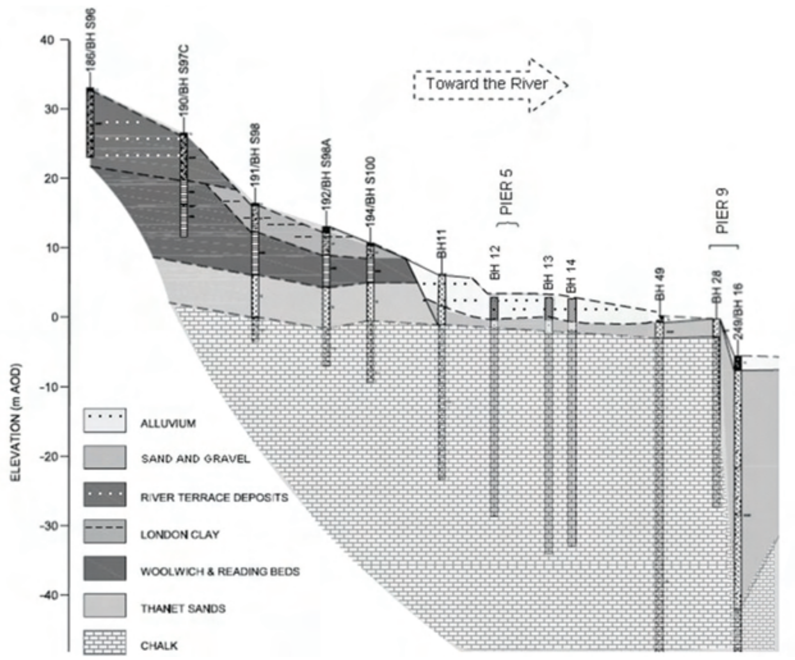

Figure 2 - Geological section of the west side of the bridge constructed from historical boreholes[5]

No ground water was encountered at shallow depths in the recent trial pits conducted in the vicinity of the scheme area, but the chalk is considered as a major Aquifer 15 and the presence of groundwater in the chalk has been confirmed in the piling record of the bridge foundation[1]. Therefore, for the purpose of this assessment, the phreatic surface was considered at the top of the chalk.

Ground condition and material properties

Chalk can be considered the main geological unit providing the bearing stratum required for the deep foundation system. As-built records[6,7] have shown an extensive investigation conducted on this stratum. The investigation comprised various common field and laboratory tests to determine the material properties. A summary of the test results is presented in Table 2. Moreover, cone penetration testing has been recently conducted for remedial works[8] at Pier 9 adding further confidence in the chalk properties.

Table 2 - Field and laboratory testing results conducted on the chalk[6,7]

Properties				Number of tests	Range (average)
SPT (results shown from borehole adjacent to Pier 5)		Elevation	0 to -15 mAOD	10	3-18 (9)
			-15 to -20	3	15-25 (20)
			-20 to -23	2	25-33 (29)
			-23 to -30	5	25-40 (33)
Triaxial tests on intact samples	Drained condition	Effective Shear strength, c', (kPa)		10 on Soft Chalk* 5 on Hard Chalk+	60* 320+
		Effective friction angle, ϕ' (Deg)			33* 39+
	Undrained	Cu, (kPa)		20 on Soft Chalk	60-260
Drained triaxial tests on remoulded samples		Effective friction angle, ϕ' (Deg) (c'=0)		8	39
Deformation Modulus, E', MPa		Insitu pressuremeter test		13	10-350
		Laboratory test		37	10-190

Table 3 - Summary of the engineering parameters of the chalk[6,7,12]

Reference	Bulk Unit Weight γ(kN/m^3)	Cu (kPa)	ϕ' (°)	c′ (kPa)	ν'	E′ (MPa)
Ref. 12	20	-	33	0	-	50 (to 2m brh**) 100 (2m to 13m) 175 (below 13m)
Ref. 7	18.5 - 20	60*-260	33*- 39	60* - 320	0.25	40 (to 10m) 80 (10m to 20m) 160 (below 20m)
Ref. 6	18.5 - 20	125*-800	39	320	0.30	17.5 (to 10m) 43 (10m to 20m) 121 (below 20m)

* Increases with depth
** brh: below rock head

Based on these ground investigations, interpretation was carried out to obtain the design parameters, of which a summary is presented in Table 3. Details on this interpretation have been discussed elsewhere[6,7,12].

The engineering parameters recommended in 1976 by Sir William Halcrow & Partner[6] are the lowest value of stiffness, which was adjusted later by the same author[7] who suggested higher values after conducting supplementary site investigation and trial pile tests in the scheme area. The highest values[12] were based on CPT (conducted by the river), where the effective Young's Modulus E was determined from relationships suggested by Meigh[16] and Lord[14].

Arrangement of the foundation system

As-built construction drawings[1] showed that the foundation system at Pier 5 contains 50 bored piles (10 x 5), with 3m centre to centre spacing and 1.05m in diameter. The pile cap is 28.5m long and 13.5m wide as shown in Figure 3. Two types of piles were identified including:

- Type B: 20 pile distributed around the edge of the pile group, and
- Type C: 30 pile in the internal area.

The two pile types have identical geometries and both are 24m long and reach -23m AOD, but different allowable working load and steel reinforcement. The difference in pile capacity may be attributed to group efficiency, where an outer pile with fewer neighbouring piles will perform better[17].

Pile load test

A static load test was conducted on an internal working pile of Pier 5 during the construction[1] (1980), where the axial load was applied to the top of the test pile with the use of hydraulic jacks. The reaction force was provided by two adjacent anchor tension piles. The test was carried out for three days where the applied load increased progressively over this period to a maximum value of 5787kN ≈150% of the working design load for a single internal pile (3850kN).

The loading produced a maximum settlement of only 3.6mm at the pile head level, with the load maintained for 12 hours.

The pile test is a valuable tool to understand the settlement characteristics and performance of the pile when under load. Therefore, the test was back analysed to evaluate the material properties and the estimated bearing capacity.

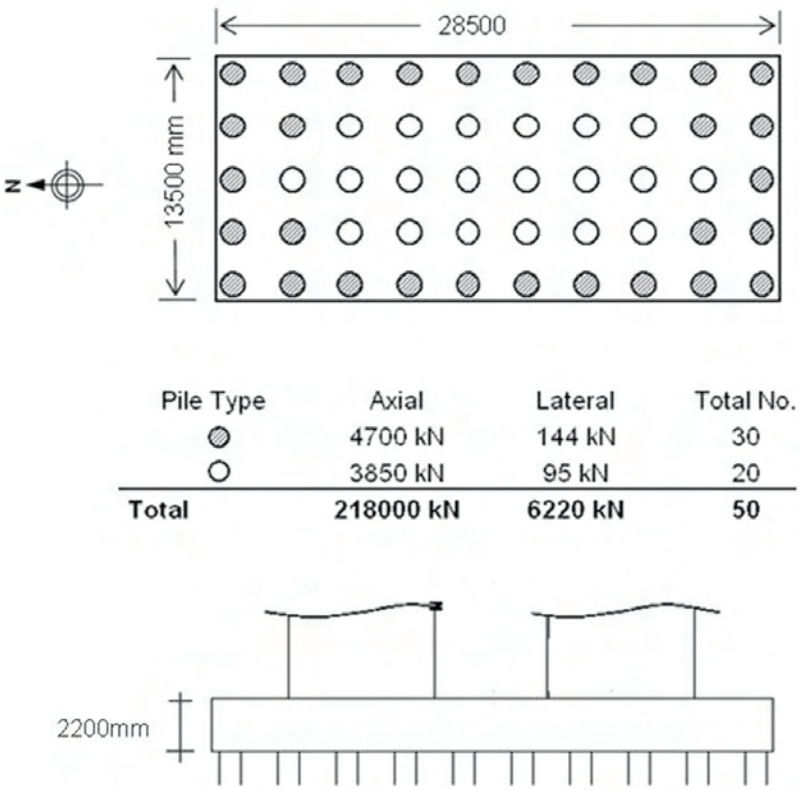

Figure 3 - Schematic diagram showing the pile group arrangement at Pier 5[1]

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Evaluation of material properties and pile capacity

Evaluation of material properties

Before conducting the pile group analysis, it was important to evaluate the properties of the chalk. For this purpose, the historical pile load test was back analysed by finite element analysis using PLAXIS. In particular, 2-D axisymmetric model of a single pile embedded in the geological sequence explained above was constructed and based on Mohr-Coulomb constitutive model.

Using the range of strength and stiffness parameters suggested in Table 3, the model predicted a maximum settlement of the pile head approximately 3.3 times the value reported in the actual test. The model was initially analysed for drained condition because the permeability of the chalk is relatively large compared with the actual loading time in the test (3 days). The permeability of chalk in the field is normally high enough to be considered free-draining[14], however, at critical state the conceptual models for the shaft resistance in chalk explained by Burland[18], suggested that a partial drained condition develops due to a thin layer of remoulded chalk formed around the shaft during boring.

The sensitivity of the model to the undrained condition was investigated; the analysis showed a better result, with a smaller gap between the actual and the predicted settlement, but still overestimated by two times, indicating that the chalk parameters used in the model (shown in Table 3) are conservative, particularly the stiffness values considered for the chalk at and below the pile toe level, where Grade III and IV were found. Compared to Matthews scale[19], these grades fall into Class B - a medium density - with secant modulus usually ranging from 1500 to 2000 MN/m^2 for stresses up to 500kPa; after this stress level the stiffness decreases[20] to 50-80MN/m^2. This proposed stiffness is up to six times larger than the values used for the chalk adjacent to the pile toe level (i.e. 20m below the chalk head) in the initial PLAXIS model.

By incorporating the new higher range of stiffness, the same settlement values reported in the pile test were obtained.

Evaluation of pile axial bearing capacity at Pier 5

The estimation of bearing capacity of bored piles in chalk is usually based on the Standard Penetration Tests (SPT) N value[21]. For a bored pile with a base area A_b, the ultimate end bearing Q_b may be estimated from:

$$Q_b = 240\ N\ A_b \text{ kN (for } N < 30) \qquad (1)$$

While there was a general agreement about the calculation of the end bearing of bored piles in chalk based on SPT, the estimation of skin friction (or shaft resistance Q_s) received more debate. Two methods of determining the ultimate skin friction in chalk were proposed: the first method was discussed by Hobbs and Healy[21] based on empirical relationships with SPT blow-count N. However, re-analysis of the case histories used for the first method indicated that the shaft resistance of bored piles should not be related solely to the SPT N value[20]. On the contrary the second method indicated that the skin friction of piles in chalk is controlled by frictional behaviour and the average effective overburden pressure along the pile shaft[22,23,24].

In the current assessment, ultimate skin friction Q_s, was calculated using both methods. Thus a smaller value was obtained using the empirical correlation with SPT. According to the calculations the total ultimate capacity of a single pile (Q_b+Q_s) varies from 10,900 and 13,500kN. Indeed, the difference was primarily caused by the uncertainty of estimating the skin friction Q_s, where the lower bound value was based on SPT. However, this method has many shortcomings[20] including that the test was not able to distinguish between a medium density chalk at Mundford and a low density chalk at Norwich for both of which N is typically 8 to 15. This was attributed to the nature of the chalk, where discontinuities in higher density or grade of chalk may ease the spoon penetration.

To reduce the uncertainty in the bearing capacity estimation, the PLAXIS model validated by the historical pile load test was rerun to find out the load required to achieve maximum pile head settlement of 10mm, where the base resistance is likely to be mobilised i.e. when shaft resistance has been or is close to being fully mobilised[14]. The result showed that the pile in the model required an axial force of about 14000kN to achieve the proposed criteria, which is close enough to the upper bound value estimated by the second method suggesting that this method may provide more economic pile design.

The working load, P_w , may be estimated from the ultimate pile capacities Q_s and Q_b divided by factor of safety: F_b= 10 and F_s =1.5 as suggested by CIRIA Report PG6[21] for bored cast in place piles with large diameter:

$$P_w = Q_b/F_b + Q_s/F_s \qquad (2)$$

The large factor applied to the ultimate end bearing was chosen to restrict settlement and to guard against risk from solution features immediately beneath a highly stressed pile base. Accordingly, the estimated working load was 5600kN, which is 20% larger than its allowable capacity (4700kN) given in the as-built construction records (see Figure 3) implying that the existing pile design is conservative. Nevertheless, the bearing capacity and the engineering parameters (i.e. in Table 3 - Ref 7), which were identified from as-built, can be reasonable for the initial assessment of the pile group.

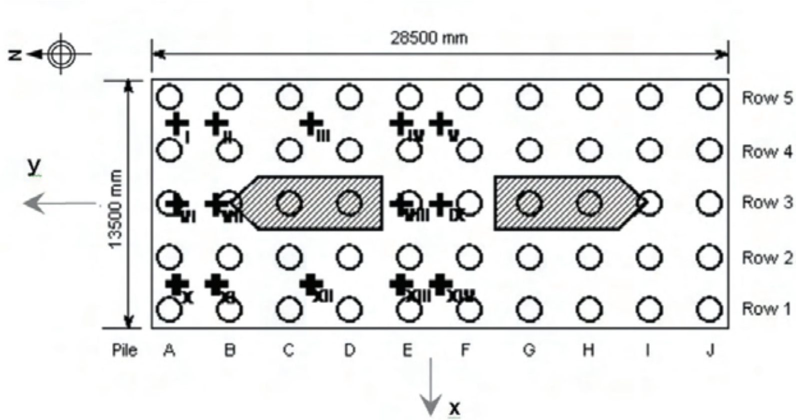

Figure 4 - Plan view of the pile cap showing pile notations and the proposed location of trestle legs (denoted +) of the steelwork

Table 4 - Load combinations used for the initial assessment of the pile group

Load case	Fz (vertical), kN	Fx, kN	Fy, kN	Mx, kN.m	My, kN.m
1	82385	685	0	83006	16780
2	81760	0	954	111561	5648
3	82074	343	954	111142	11214
4	72178	1753	1753	126532	65847

Structural analysis of pile group

Bridge load combinations

Details about trestle legs to be used for the steel work and the load breakdown for the geotechnical analysis were provided by the bridge engineers. This information is presented in Figure 4 and Table 4, where four critical load cases were identified and considered in the assessment herein.

In addition to the effect of the eccentricity of the loads applied by the trestle legs (shown in Figure 4) with an additional 800mm thick concrete slab placed on the north side of the pile cap, the bending moments also include other effects such as wind and jacking forces.

Pile group analysis using PIGLET

The pile group was initially analysed using PIGLET[4]. The program is Excel based software which is an approximate closed form solution allowing analysis of the elastic response of pile groups under 3D working load conditions. In the analysis the soil is modelled as a linear elastic material, with stiffness varying linearly with depth. The solution provides stiffness and flexibility matrices for the pile cap, axial, lateral and moment loading at the head of each pile and profiles of bending moment and lateral deflection down selected piles.

The predicted forces and deflection are dependent on the stiffness of the piles, soil and also on the fixity of the pile head and the flexibility of pile cap. In this analysis, the Young's modulus of the concrete of the piles was taken equal to 15GPa, the piles were assumed fixed to a rigid pile cap, and the loads were applied on the centre of the pile cap. Based on the parameters recommended in Table 3, the axial shear stiffness of the soil was considered equal to zero at the ground surface increasing linearly by approximately 2.5MPa per metre depth.

The deflection and rotation of the pile cap predicted by the software was insignificant. However, the axial load was noted to be relatively large. Figure 5 (i) shows the maximum axial loads predicted in all piles by PIGLET, where clearly the piles at the north side are subjected to a larger value as a result of the load combinations (shown in Table 4) produced by the proposed supporting system, with the greatest load of 4900kN in the corner pile A5, which exceeded the as-built allowable axial pile capacity, P_w, by 5% (where P_w = 4750kN).

Discussion and further analysis

To evaluate the PIGLET analysis results, further assessment was carried out by predicting the response of the pile group by considering a perfect plastic behaviour for the chalk in addition to the linear elasticity. This was carried out using REPUTE software, which is based on Boundary Element Method[5], and has the option of using shear strength criteria of Mohr-Coulomb. In particular, the effective friction angle of the chalk was considered in addition to the initial elastic modulus as given in Table 3.

The result of the axial loads obtained from REPUTE for each pile is graphically presented in Figure 5 (ii), indicating that the maximum vertical load is about 3500kN, which is almost 30% less than the allowable bearing capacity used for the design.

The diagram of the bending moment was also predicted and presented

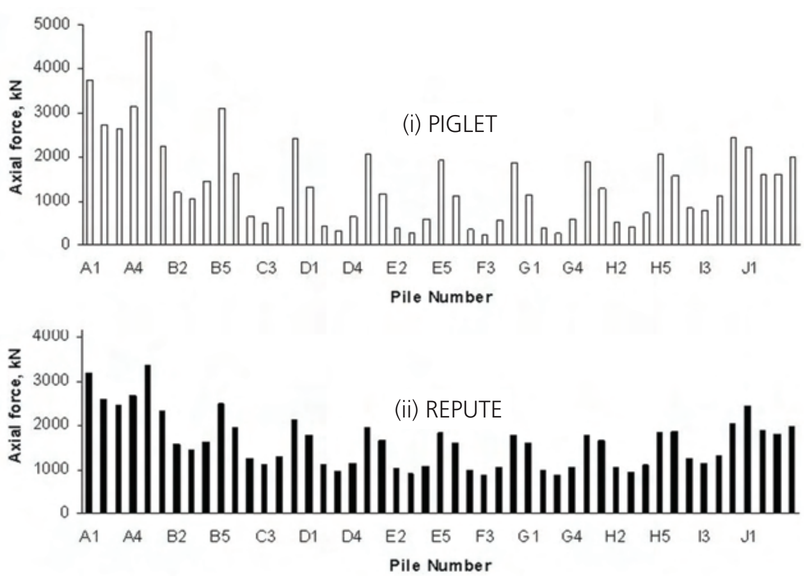

Figure 5 - Maximum axial loads in all piles obtained by: (i) Piglet and (ii) Repute

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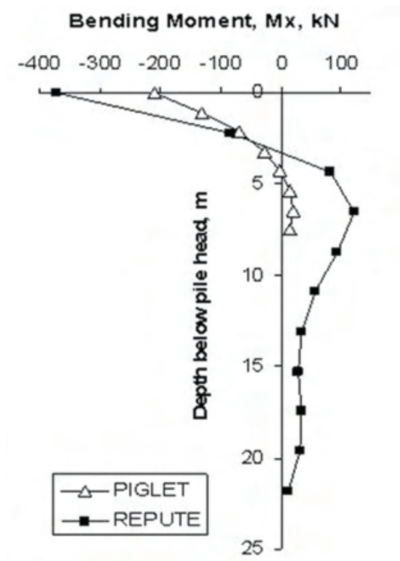

Figure 6 - Diagram of bending moment predicted by PIGLET and REPUTE.

in Figure 6. Comparing the results obtained from both softwares, the linear response predicted by PIGLET has resulted in 53% larger vertical loads at the corner piles; but 44% less maximum bending moment.

The difference in the results obtained from both softwares cannot only be attributed to the plasticity of the materials used in the analysis but also to the method and assumptions embedded in the software, into which further investigation is being carried out.

Conclusion and recomendations

An initial geotechnical assessment of a group of bored piles in chalk was presented in this paper discussing the stages implemented to ensure that safe working loads are applied during the bearing replacement of the bridge pier and that no excessive deflection or rotation will result from the new temporary load distribution.

To establish the ground conditions and determine the engineering parameters of the strata, the existing geotechnical data was collected from as built records and then evaluated against a historical static load test using PLAXIS software.

The bearing capacity of a single pile was calculated considering two different methods in obtaining the ultimate skin friction, where the larger value based on the effective overburden pressure was found to be in agreement with the results of the pile load test analysis.

The elastic response of the pile group was analysed using PIGLET software, which has predicted a significant increase in the vertical loads on the edge piles at the north side (where the bridge maintenance work will occur). However, this proved to be an over-estimation compared with the result obtained from REPUTE software where a perfect plastic behaviour of the chalk was considered.

The analysis of the pile group can be improved by using analytical tools that consider the non-linear mechanical behaviour of the chalk, i.e. adopting strain and stress dependent stiffness. Nevertheless, the analysis carried out was sufficient to satisfy the designers that the proposed method of bearing replacement would not cause an unexpected behaviour to the existing bridge sub or superstructures.

Acknowledgement

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References

1. Sir William Halcrow & Partners, Frederick Gibberd & Partnersand. Exploratory Boreholes, Soil Classifications, Piling record and Pile test loading record, Pile and pile cap details of Pier 5. As-built construction drawings and documentations, Ipswich By-Pass, Orwell Bridge, 1979-1984.
2. Atkins. A14/193.8 Orwell Bridge Bearing Replacement Pier 5 North Deck – Assessment Under Temporary Support Conditions. Approval In Principle, Study prepared for the Highway Agency, 2007.
3. PLAXIS BV. Manual of the Program Plaxis 2D, version 8.0, 2005
4. Randolph M F. PIGLET 5.1, Analysis and design of pile groups. 2006.
5. Geocentrix. User manual of the program Repute version 1.5. 2006.
6. Sir William Halcrow & Partner. Ipswich By Pass Orwell Bridge, Site Investigation Report. 1976.
7. Sir William Halcrow & Partner. Felixstowe-Weedon Trunk Road A45 Ipswich By Pass Southern Section Orwell Bridge, Supplementary site investigation report. 1979.
8. Lankelma Ltd. Orwell Bridge Pier 9 - Cone Penetration Test Results, Factual Report on Ground Investigation. April 2005.
9. British Geological Survey. Ipswich, Geological Map 207, 1:50,000, 1990.
10. British Geological Survey. East Anglia and adjoining areas, British Regional Geology, 1961.
11. Atkins GSI. A14 Orwell Bridge – Phase 4, Factual Report: F359/5042944, September 2007.
12. ATKINS. Orwell Bridge Pier 9 Remedial Works, Geotechnical report, May 2005.
13. Ward W H, Burland J B and Gallois R M. Geotechnical assessment of a site at Mundford, Norfolk, for a proton accelerator, Geotechnique 1968, 18, (No. 4), pp 399-431.
14. Lord J A, Clayton C R I, Mortimore R N. Engineering in chalk, CIRIA report C574, London, 2002.
15. Environment Agency. Groundwater Vulnerability of East Suffolk, 1:100,000 Map series, Sheet 33. 1996.
16. Meigh A C. Cone Penetration Testing - Methods and Interpretation, CIRIA, Butterworths. 1987.
17. Das M B. Principles of Foundation Engineering. 4th Edition. Pacific Grove, California, 1998.
18. Burland J B. Preface. Proceedings of International Chalk Symposium, Brighton, Thomas Telford, London, 1989, pp 1-4.
19. Matthew M C G, Clayton C R I and Russell C S. Assessing the mass compressibility of chalk from visual description, Proceeding of Engineering Group of Geological Society. Conference on The engineering Geology of weak rocks, Leeds, 1990, Balkema, Rotterdam, pp 351-368.
20. Lord J A, Twine D, Yeow H. Foundation in chalk, CIRIA report PR11, 1994.
21. Hobbs N B, Healy P R. Pilling in chalk, CIRIA report PG6, 1979.
22. Twine D. Discussion: Session III. Proceeding of International Chalk Symposium, Brighton, 1989. Thomas Telford, London, pp 415-416.
23. Clayton C R I. The mechanical properties of the chalk. Proceedings of International Chalk Symposium, Brighton, 1989. Thomas Telford, London, pp 213-232.
24. Burland J B, Hancock R J and May J. A case history of a foundation problem on soft chalk. Geotechnique 33, (4). 1983, pp 385-395.

Deployment and safety benefits of Open Road Tolling for mainline toll plazas in Florida

Eric Gordin

Assistant Traffic Operations Engineer

Florida's Turnpike Enterprise

Jack Klodzinski

Senior Transportation Engineer

Florida's Turnpike Enterprise

URS Corporation

Cristina Dos Santos

Tolls Engineering Specialist

Atkins North America

Abstract

In the search to improve mobility, increase safety and provide better service to the public, different toll agencies worldwide have upgraded their system to the Open Road Toll (ORT) designs. Based on the research analysis conducted by the authors, upgrading from conventional toll lanes to ORT has demonstrated measured improvements in traffic operations at toll plazas and subsequently conceivable benefits in driver safety from a reduction to potential crash exposure. A previous 2007 analysis of crash data from an ORT plaza in Orlando, Florida resulted in a 22% drop in total crashes at an ORT plaza and 26% drop in the plaza's area of influence. To further evaluate ORT with Electronic Toll Collection (ETC) benefits, additional crash data were collected from six toll plazas recently converted to ORT located on Florida's Turnpike Enterprise (FTE) system of toll roads. A before and after study analysis was conducted where frequency, crash rate and crash severity were analysed. The results supported the previous study showing a significant reduction in the number of crashes. The evaluation analysed crash data over three years (2007 to 2009) and there was a decrease in the total number of crashes, crashes in the non ORT lanes, number of total injuries and total injury crashes. The calculated crash rate at all the ORT plazas reviewed displayed as much as a 70% reduction. To quantify the benefits, average cost of a crash was obtained and when the ORT lanes were compared to the before study, a saving of $1m was shown. The future of ETC is now moving to All Electronic Tolling (AET) and if the statistics for crash reductions holds true, the toll road user benefits could be very significant.

Introduction

Transportation engineers and planners, through the use of innovative technologies, are seeking alternatives to improve performance at toll plazas and relieve traffic congestion on toll facilities in the United States and throughout the world. Improvements have been sought to decrease the time of each toll transaction so that users experience either reduced delays or no delays at all. This has been done through the introduction of automated payment systems such as Automatic Coin Machines (ACM) and later, implementing the ITS technology of Automatic Vehicle Identification (AVI)[1]. Though many toll authorities have already applied AVI by utilising it in the application of ETC, increasing traffic volumes have demanded toll authorities speed up the implementation of ORT to better serve their customers[2,3,4].

ORT allows toll collection at highway speeds and is sometimes referred to as express ETC which allows drivers to travel at speeds of 55mph (88.5 kph) or more. The current trend in the evolution of smart toll collection initiatives and toll road operations is AET collection. This toll collection concept combines ORT with video capture technology or Optical Image Recognition (OIR) to bill the drivers who do not have a local area transponder.

Prior to the concept of ORT, the method for improving operations at a toll collection facility was adding additional service lanes, modification of a conventional payment (cash, token, or ticket) lane to accept ETC, or conversion of a conventional lane to a dedicated ETC lane (typically with a speed limit imposed). Now ETC technology has been embraced by the general driving population of toll facilities and the technology has evolved, it can effectively be applied in a more efficient and economical manner. Thus, the advancement of ETC application to develop ORT continues.

Although mixed-use lanes (conventional toll lanes equipped with ETC technology) and dedicated lanes have been widely used on toll facilities, as the percentage of the vehicles equipped with ETC increases, the benefit of ETC decreases in these lanes due to increased congestion from recurring bottlenecks at the plaza. ORT eliminates existing plaza barriers and creates a new toll road design that mitigates this congestion. ORT is fully automated ETC in an open road environment, allowing vehicles to travel through toll collection points without deceleration.

Figure 1 - University mainline plaza

There are a number of agencies around the world that have implemented ORT, including Israel and Canada[5,6,7]. The US National Transportation Safety Board in 2006 called for toll plazas nationwide to

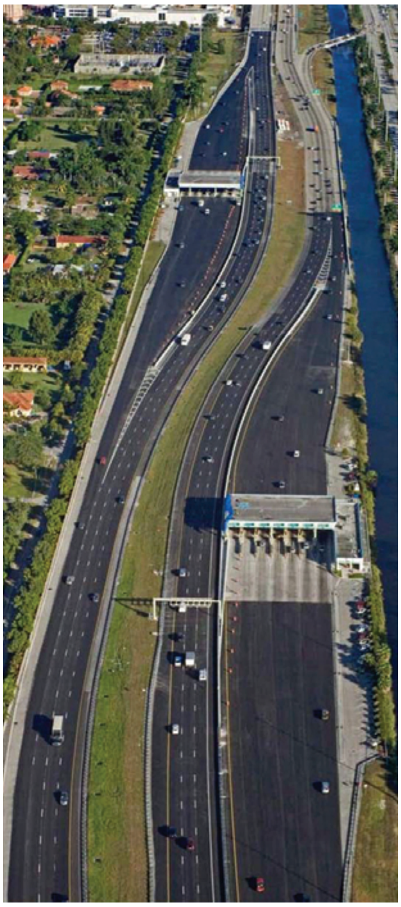

Figure 2 - Bird Road toll plaza

be revamped to reduce the risk of rear-end collisions. In an attempt to do this several toll agencies in the United States converted to ORT[8]. In the United States, some of the state agencies or toll authorities in Oklahoma, Georgia, Texas, New Jersey, California, and Florida have projects underway or in operation [2,9,10,11,12,13]. With the current deployment of ORT, two versions of this concept are being defined. One design is removal of all vertical structure and elimination of conventional toll service lanes (non-ETC payment). This design includes allowing drivers to continue travelling at the posted speed as they pass through a toll collection facility on the mainline and also does not require the driver to stop on ramps where a toll payment is required. The other version of ORT utilises both express ETC and mixed-use lanes into the design of the toll collection facility. Their current deployment of this ORT concept involves a retrofit of existing plaza structures. Once completed, the plaza has a physical separation between the express ETC lanes and the mixed-use lanes. There is an immobile barrier wall and median making the area for toll service virtually two separate facilities. The ETC equipped vehicles continue on the main highway lanes without a requirement to decelerate for toll collection.

Within the state of Florida, many agencies have completed ORT conversions. Miami-Dade Expressway has already begun their AET transition and Florida's Turnpike Enterprise (FTE) will begin theirs in 2011 [11,14,15]. FTE has completed most of their plaza conversions to this ORT design and the future work plan is to move forward with all electronic toll (AET) collection[16]. Florida's Turnpike mainline, also designated as State Road 91, is a limited-access toll road that runs 312 miles, through 11 counties, beginning near Florida City in Miami-Dade County and terminating near Wildwood in Sumter County, Florida[14]. FTE completed the first conversion to ORT in November 2007 at the Bird Road Mainline Plaza (Figure 2) in south Florida [16]. FTE manages about 606 miles (975 kilometres) of roadways in the state including the Sawgrass Expressway (Toll Road 869) in Broward County, Florida. FTE provides service to more than 1.6 million motorists every day and averages about 650 million transactions each year.

Orlando-Orange County Expressway Authority (OOCEA) provides more than 90 miles of interstate standard roadway to drivers in Central Florida. Currently the network consists of eleven mainline toll plazas with corresponding on and off ramps. The first road in the OOCEA system to be designed with their ORT concept was introduced in 2001 on State Road 429 (Western Expressway)[17] SR 429 is a 10.6-mile (17.06km), $237-million, limited-access tolled expressway that was the first in the state of Florida to feature express ETC lanes and ORT. Though benefits from ORT are obvious, quantifying those benefits requires data with and without ORT [18,19]. The first OOCEA mainline toll plaza retrofit was successfully completed in the summer of 2003 at the University Mainline Toll Plaza on SR 417 (Figure 1). OOCEA selected this plaza first since it had the highest percentage of ETC customers (greater than 60% during the peak hour). All of the OOCEA main-line toll plazas have now been reconstructed to incorporate express lanes[20]. This plaza was previously evaluated for operational benefits and some crash data were utilized to potentially quantify safety benefits.

University Mainline Toll Plaza evaluation (previous case study)

The University Mainline Toll Plaza was evaluated and operational improvements as well as potential safety benefits were concluded. Construction of the University Mainline Toll Plaza occurred between Spring 2002 and Summer 2003. Before implementation of ORT, speed limits of 35 mph (56.3kph) were imposed on all dedicated ETC lanes and these lanes are located adjacent to the cash paying lanes (manual and ACM). The ORT concept with express ETC lanes was completed in summer 2003 with a posted speed limit of 55mph (88.5 kph).

Prior to beginning construction, average throughput of the plaza was 6,000vph for both directions. In spring 2003 one additional approach lane to the plaza in each direction was completed and in summer 2003, after construction was completed, the delays at the plaza decreased substantially. The additional approach lane increased the level of demand at the plaza for the cash lanes because the previous upstream bottleneck on the approach to the plaza was eliminated. For the AM peak southbound, the average delay for a manual cash customer decreased by an average of 6 seconds and an ACM customer's average delay decreased by 2 seconds. For the PM peak northbound the average delay for a manual cash customer decreased by an average of 8 seconds and an ACM customer's average delay decreased by 10 seconds. The addition of an approach lane in the northbound direction to provide additional plaza approach capacity with the new plaza configuration proved to be an improvement for the plaza's performance. Another indirect benefit is the potential for improved safety based on the observed reduction in plaza turbulence from lane changing between all vehicle payment types. From the video analyses it was realized that throughout the before study analyses of upstream videotaped arrivals traffic turbulence from vehicle lane changing increased as demand

Table 1- OOCEA University Mainline Toll Plaza crash data before and after conversion to ORT

	Crashes at University Plaza [1]				Crashes in Area of Influence [2]			
	Before		After		Before		After	
Crash Group/Deception	Number	Percent	Number	Percent	Number	Percent	Number	Percent
Lane Type								
ORT	3	33%	0	0%	N/A	N/A	N/A	N/A
Non ORT	4	44%	7	100%				
Not recorded	2	22%	0	0%				
Direction								
Northbound	3	33%	5	71%	10	27%	11	44%
Southbound	6	67%	2	29%	21	57%	12	48%
Unknown	0	0%	0	0%	6	16%	2	8%
Total Crashes	9	100%	7	100%	37	84%	25	92%
Harmful Event 4								
Rear end	6	67%	4	57%	16	43%	7	28%
Backed into	0	0%	1	14%	0	0%	1	4%
Collision with pedestrian	1	11%	0	0%	1	3%	0	0%
MV Hit Guardrail	0	0%	2	29%	5	14%	4	16%
Hit Bridge/Pier/Abutment/Rail	1	11%	0	0%	13	35%	13	52%
All other	1	11%	0	0%	13	35%	13	52%
Contributing Cause 4								
No improper driving/action	2	22%	0	0%	4	11%	1	4%
Careless driving	6	67%	5	71%	27	73%	16	64%
Improper parking	1	11%	1	14%	1	3%	1	4%
Followed too closely	0	0%	1	14%	0	0%	1	4%
All other	0	0%	0	0%	5	14%	6	24%
Statistics								
Avg. vehicles per crash	2.1	N/A	1.7	N/A	1.9	N/A	1.8	N/A
Total injuries	10		4		49		15	
Fatal crashes	Data not availible				1	3%	0	0%
Injury crashes					30	81%	13	52%
Property damage only crashes					7	19%	12	48%
Total Crashes					37	100%	25	100%
AADT					46,716		62,020	
Vehicle miles (thousandths) [3]					45,755		60,743	
Estimated crash rate					0.81		0.41	
Crashes per mile					16.1		10.9	
Note: Table data from law enforcement crash reports								
[1] Does not incliude crashes away from plaza structure			[3] Estimated based on 2.3 miles					
[2] Does not include plaza crashes			[4] Harmful event and contributing cause are both reported on an accident report					

increased. This turbulence obviously increases exposure to the potential for a traffic crash. Furthermore, during video analyses an actual sideswipe traffic crash in the dedicated ETC lane was observed. Therefore, the separation of the ETC lanes from the cash lanes in itself provides a benefit.

Cost and benefit analysis

Previous study results

The conversion of University Mainline Toll Plaza was completed over 18 months at a budget of $25.5 million[21]. To evaluate some possible benefits, a preliminary evaluation of the operations and crash data were reviewed. Speed has been found to have a significant influence on traffic crashes[22-25]. Navon has found that accident-prone interactions occur more with lower average speeds[23]. Feng suggested that higher speed may not lead to more crashes[24]. Through research conducted by M. Abdel-Aty, speed variance increase can augment the probability of crashes[25-27]. This supports the probability of lower crash rates being achieved from plaza conversion to ORT lanes. The available speed data for the ORT lanes

Table 2 Turnpike annual crash analysis before, during and after conversion to ORT

Crashes at Turnpike Plazas [1]						
	Year 2007		Year 2008		Year 2009	
Crash Group/Description	**Number**	**Percent**	**Number**	**Percent**	**Number**	**Percent**
Lane Type						
ORT	0	0%	20	4%	39	12%
Non ORT	526	100%	514	96%	280	88%
Not recorded	0	0%	0	0%	0	0%
Direction						
Northbound	227	43%	239	45%	138	43%
Southbound	252	48%	239	45%	134	42%
Eastbound	17	3%	32	6%	27	8%
Westbound	24	5%	23	4%	18	6%
Unknown	6	1%	1	0%	2	1%
Total crashes	526	100%	534	100%	319	100%
Harmful event [2]						
Rear end	121	23%	107	20%	95	30%
Backed into	5	1%	2	0%	25	8%
Collision with pedestrian	1	0%	0	0%	1	0%
MV hit guardrail	61	12%	86	16%	10	3%
Hit Bridge/Pier/Abutment/Rail	1	0%	2	0%	0	0%
All other	337	64%	337	63%	188	59%
Contributing Cause [2]						
No improper driving/action	82	22%	85	22%	22	22%
Carless driving	174	67%	201	67%	109	67%
Improper parking	6	11%	1	11%	32	11%
Followed too cLosely	15	0%	2	0%	11	0%
All other	249	0%	245	0%	145	0%
Statistics						
Avg. vehicles per crash	1.8		1.7		1.9	
Total injuries	464		390		180	
Fatal crashes	5	1%	3	1%	2	0%
Injury crashes	276	52%	240	45%	102	19%
Property damage only crashes	245	46%	291	54%	215	40%
Total crashes	526	99%	534	100%	319	60%
AADT	**Data not availible**		**Data not availible**		**Data not availible**	
Vehicle-miles (thousandths)[3]						
Estimated crash rate						
Crashes per mile						
Note: Table data from law enforcement crash reports						
[1] Does not include crashes away from plaza structure						
[2] Harmful event and contributing cause are both reported on an accident report						
[3] Estimated based on 2.3 miles						

at the plaza show a 61% increase of speed after the ORT lanes were open.

The crash data covered 14 months prior to the beginning of construction at the plaza and then after conversion was complete. This data is summarized in Table 1. No crashes were recorded at the plaza in the new ORT lanes. The number of crashes at the plaza dropped by 22%. The average number of vehicles involved in a crash at the plaza dropped from 2.1 to 1.7 vehicles. The number of reported injuries was the most significant drop at 60%. The full crash data set is shown in Table 1.

The 2.3 mile segment of road between the two closest interchanges north and south of the mainline plaza was also upgraded to accommodate the additional toll lanes and future traffic. This is considered to be the area of influence for the plaza conversion in this study. The

Table 3 Florida Turnpike ORT Plaza crash analysis

Total Crashes - Before ORT versus After ORT

Crash Analysis - Before ORT versus After ORT Conversion

Toll Plaza			AADT		ORT Opening Day	After ORT Time Period (Days)	Total Crashes		Crash Rate (MEV)**		Reduction
Location	Roadway	Mile Post	FY 2007*	FY 2008*			Before ORT	After ORT	Before ORT	After ORT	
Cypress Creek	Turnpike Mainline	63	101,200	94,200	2/4/2008	365.0	54	16	1.5	0.5	-68%
Homestead	HEFT	10	72,800	72,300	12/21/2007	365.0	18	9	0.7	0.3	-50%
NB Bird Road	HEFT	22	122,600	114,600	11/13/2007	365.0	8	7	0.4	0.3	-6%
SB Bird Road	HEFT	22	122,600	114,600	11/15/2007	365.0	12	5	0.5	0.2	-55%
Lantana	Turnpike Mainline	88	71,300	66,000	3/17/2008	365.0	38	11	1.5	0.5	-69%
Sunrise	Sawgrass Expwy	1	76,500	72,300	4/28/2008	365.0	9	4	0.3	0.2	-53%
Beachline West	Beachline Expwy	6	70,000	69,600	8/13/2008	365.0	10	3	0.4	0.1	-70%
							149	55			-53%

* FY 2007 was obtained from planning - Traffic Trends and FY 2008 was obtained from Finance - Traffic Engineer's Annual Report FY 2008

** Crash Rate for the Spot (MEV = Million Entering Vehicles)

~ Crash Rate for Bird N and S was calculated using AADT divided by 2

Crash Analysis - Before ORT versus After ORT Conversion

Toll Plaza			Total Crashes															
			Before ORT				After ORT											
							ORT				CASH				TOTAL			
Location	Roadway	Mile Post	Fatal	Injury	PDO	Total	Fatal	Injury	PDO	Total	Fatal	Injury	PDO	Total	Fatal	Injury	PDO	Total
Cypress Creek	Turnpike Mainline	63	0	6	48	54	0	2	6	8	1	2	5	8	1	4	11	16
Homestead	HEFT	10	0	1	17	18	0	2	4	6	0	2	1	3	0	4	5	9
NB Bird Road	HEFT	22	0	0	8	8	0	2	1	3	0	2	2	4	0	4	3	7
SB Bird Road	HEFT	22	0	0	12	12	0	0	1	1	0	1	3	4	0	1	4	5
Lantana	Turnpike Mainline	88	0	2	36	38	0	1	2	3	1	0	7	8	1	1	9	11
Sunrise	Sawgrass Expwy	1	0	1	8	9	0	0	1	1	0	0	3	3	0	0	4	4
Beachline West	Beachline Expwy	6	0	0	10	10	0	0	0	0	0	1	2	3	0	1	2	3
Total			0	10	139	149	0	7	15	22	2	8	23	33	2	15	38	55

Toll Plaza			Before ORT		After ORT			
					ORT		CASH	
Location	Roadway	Mile Post	Injury	PDO	Injury	PDO	Injury	PDO
Cypress Creek	Turnpike Mainline	63	$303,072	$144,000	$101,024	$18,000	$101,024	$15,000
Homestead	HEFT	10	$50,512	$51,000	$101,024	$12,000	$101,024	$3,000
NB Bird Road	HEFT	22	$ -	$24,000	$101,024	$3,000	$101,024	$6,000
SB Bird Road	HEFT	22	$ -	$36,000	$ -	$3,000	$50,512	$9,000
Lantana	Turnpike Mainline	88	$101,024	$108,000	$50,512	$6,000	$ -	$21,000
Sunrise	Sawgrass Expwy	1	$50,512	$24,000	-	$3,000	$ -	$9,000
Beachline West	Beachline Expwy	6	$ -	$30,000	$ -	$ -	$50,512	$6,000

Toll Plaza			Before	After	
Location	Roadway	Mile Post		ORT	Cash
Cypress Creek	Turnpike Mainline	63	$447,072	$119,204	$116,024
Homestead	HEFT	10	$101,512	$113,024	$104,024
NB Bird Road	HEFT	22	$24,000	$104,024	$107,024
SB Bird Road	HEFT	22	$36,000	$3,000	$59,512
Lantana	Turnpike Mainline	88	$209,024	$56,512	$21,000
Sunrise	Sawgrass Expwy	1	$74,512	$3,000	$9,000
Beachline West	Beachline Expwy	6	$30,000	$ -	$56,512
TOTAL			$922,120	$398,584	$416,584

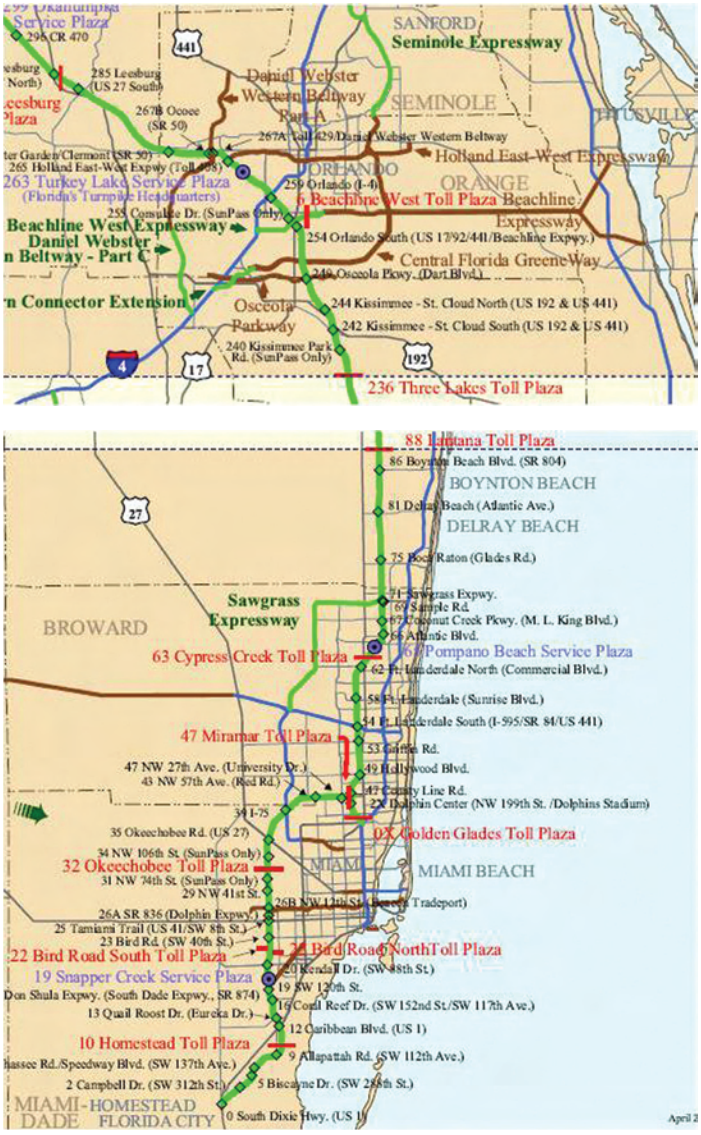

Figure 3 - Florida Turnpike study plaza locations

number of crashes in the area of influence dropped by 26%. The number of reported injuries in the area of influence also had the most significant change, a decrease of 69%. The total crashes including those that were reported but did not have details about the location or type of crash dropped 32%, from 37 to 25 crashes. Also, not only was there a drop in crashes but the Annual Average Daily Traffic (AADT) increased by 33%. Based on the total crashes, average AADT, and estimated vehicle-miles (in the area of influence), a calculated crash rate shows an overall reduction of 49%.

Current study results

To further expand upon the investigation of effects from implementing ORT at a toll plaza, traffic crash data were procured from FTE. The Turnpike speed change is similar to the OOCEA's University plaza with an increase from the previous 25mph to posted highway speeds (between 55mph and 70mph). A before and after study analysis was conducted to analyse the Turnpike's toll plaza crash data. Crash data for years 2006 to 2009 were used to complete this analysis. The before period includes 12 months of data before ORT opening day and the after period includes 12 months of data after ORT opening date. Because the ORT conversions were done during different dates, the before and after period dates are different for each FTE toll plaza being analysed in this paper. In addition, an annual crash analysis for years 2007 to 2009 that summarises details of total system crashes (e.g. harmful event, contributing causes) and ORT plaza crash summary data were also created to compare previous OOCEA study. The data have been summarised in Table 2 and Table 3.

The annual crash analysis covers the entire Florida Turnpike System's toll plazas including the conventional and the ORT toll plazas. A summary was compiled of the same data used in the evaluation of the OOCEA University Mainline Toll Plaza. Year 2008 can be considered a transitioning year where a number of plazas had ORT lane conversions completed by August of 2008, and 2009 would be a complete after period for ORT plaza conversions. When comparing the "before" period to the "after" period, there is a significant reduction in total crashes and injuries overall. When the transition year, 2008, is considered though there is a slight increase in total crashes, the potential effects of implementing ORT may be discerned from the reduction in fatal crashes, injury crashes and total injuries for the year. Also, with the increase in "careless driving" contributing causes, it may be attributed to construction and a newly implemented design to which drivers were becoming accustomed.

The ORT plaza crash summary data were used to evaluate the toll plazas on Florida's Turnpike System where ORT was implemented between fall 2007 and summer 2008. All the plazas, except the Beachline, are located in the southern part of Florida whereas the previous study looked at a plaza in central Florida where the Turnpike's Beachline plaza is located. Figure 3 is a map identifying the locations of the ORT plazas along Florida's Turnpike System that were part of the economic and crash rate study. Costs were not available in the previous study therefore the crash data were not quantified in terms of dollar value. The current data attaches an estimated cost per crash type to better quantify the results in terms of benefits. Values/costs for crashes at the time of analysis were approximately $3,000 for Property Damage Only (PDO) based on Florida related traffic crash reports and $50,512 for an injury[28]. Due to the infrequency, lack of information regarding fatal crashes and the sensitive nature of evaluating fatalities, they were not included in the cost analysis.

Six mainline toll plaza locations have been included in this evaluation of safety benefits from ORT using ORT plaza crash summary data. The Bird Road Plaza is denoted by direction and therefore evaluated by direction. Only crashes that occurred at the toll plazas were included with the evaluation covering only the ORT converted plazas. Crashes upstream or downstream that had no data supporting influence from the plaza or ORT lanes were not included. The data analysis includes twelve months before and twelve months after implementation of ORT. Table 3 indicates the opening day for each plaza. The analysis summarises total crashes by Fatality (Fat), Injury (Injur), and PDO where available.

In order to normalize the crash data, the crash analysis includes crash rate using 2007 and 2008 AADT for before and after ORT conversions. Crash rates are normally considered better indicators of risk than crash frequencies alone, because they account for differences in traffic volumes, hence exposure and time period of the analysis. The crash rate was based on Million Entering Vehicles (MEV) with total annual crashes factored to 10^6 and the total yearly traffic at the plaza (e.g. [Total Crashes * 10^6]/[AADT*365]). The total crashes for all plazas studied showed a reduction at each plaza. Cypress Creek and Lantana had high crashes compared to the rest of the plazas. They had double digit reductions in total crashes with the implementation of ORT. These two plazas with the highest total crashes before ORT had almost a 70% reduction in crash rates. Table 3 provides details of the crash analysis by plaza.

The analysis compares total crashes and also separates the after analysis into cash and ORT. The total number of crashes that occurred at the ORT converted plazas after implementation showed a significant reduction, less than half of the total before ORT was implemented. Also, the number of ORT Lane crashes compared to Cash Lane crashes was less, with no recorded fatalities and slightly fewer injuries. In addition, each crash report diagram was evaluated to determine the location of the crashes and it was noticed that the majority of crashes that took place on the ORT lanes were caused by drivers changing lanes at the last moment. When considering reduction in crashes after implementation of ORT, as well as ORT versus Cash, ORT clearly shows benefits, especially if you consider a fatality. The fatality crash records were reviewed to determine if a correlation between the crash as a fatality and the fact it was in a cash lane contributed to the severity of the crash, but there were not enough data to draw a conclusion. However, it may be inferred based on the lower variability in speeds for ORT Lanes (drivers continue through at posted highway speeds) compared to Cash Lanes that have a high variation in speed with approach, departure and stopping in order to pay the toll.

When comparing the crash severity and economic impacts to society of ORT versus Cash, the total estimated cost for the ORT crashes is less than the Cash lanes. The property damage only costs dropped drastically. The overall estimated costs from crashes after ORT was implemented are less than the before study.

Conclusions

ORT is the current application of ETC being utilised on transportation facilities throughout the world. Express ETC lanes can either be provided at new toll plaza collection points or constructed at existing plazas to replace current speed-controlled dedicated ETC lanes. The OOCEA recently completed the conversion of all plazas to ORT. The University Mainline Toll Plaza was the first of the OOCEA's existing toll plazas to undergo a retrofit in order to increase cash lane capacity and add express ETC lanes. A case study evaluated the plaza's operations before and after renovation was completed. The performance of the plaza was improved in terms of increased throughput and reductions in delay. A calculated crash rate from the AADT, vehicle-miles and total crashes showed a 49% reduction.

The additional data from Florida Turnpike Plazas continue to show a trend in the reduction of crashes at toll plazas that have been converted to include ORT lanes. Every plaza that had an ORT conversion showed a reduction in crashes for the twelve months following the completion of construction. Though the overall crashes during the year of transition, 2008, had a slight increase in total crashes, based on the reduction of injury crashes and increase of property damage only crashes, the construction at the plazas and newly implemented plaza design could have contributed to some crashes. This may be further supported by the fact that after full implementation of these ORT plazas, 2009 showed a reduction of all crashes except backed into and followed too closely. These may also be attributed to ETC usage in the Cash Lanes but without examination of details from individual crash reports, the individual causes are unknown.

Also, based on the crash rate at each plaza computed using total crashes, a spot count of vehicles entering the plaza (in millions), every plaza showed a reduction and all but one had more than a 50% reduction in the crash rate. The cost of crashes also gives some indication of the potential savings due to reductions in crashes based on a comparison of the ORT crashes to the before study crashes.

The future of ETC is now transitioning not only to ORT but to the elimination of any cash transactions. This upcoming trend is being referred to as AET. Based on the current data, it could be inferred that if AET was implemented, an estimated additional half million dollars could be saved. That is the total cost of crashes at the ORT plazas in the Cash Lanes.

The concluding results from the review of ORT and the use of express ETC lanes are positive from the previous study and the current analysis. Not only can the use of this innovative concept improve a toll plaza's operations, but it may also improve the safety for drivers approaching and departing from a toll plaza due to the reduction in possible conflicts from the ORT/Express ETC plaza design[29]. Preliminary analysis supports the probability of crash reductions from the conversion to ORT. With a continued progression in ORT technology, a reduction in plaza structures drops the potential exposure to crashes. Toll facilities around the world continue to improve their traffic operations and collection procedures to be more efficient which in turn improves the safety of their drivers.

References

1. Pietrzyk M.C. Electronic Toll Collection Systems. Transportation Research Board Special Report 242, Curbing Gridlock, National Research Council, Washington, D.C., 1994, Vol. 2, pp. 464-501.
2. ITS America Vol. 12, No. 5, May 2002, ITS View – Open Road Tolling.
3. Miller, P. The Bell Tolls for Tollbooths. Civil Engineering, August 1995, pp. 48-50.
4. U.S. Department of Transportation, Federal Highway Administration (2003). A Guide for HOT lane development, http://www.itsdocs.fhwa.dot.gov/JPODOCS/REPTS_TE/hot/glossary.htm. Accessed July 17, 2003.
5. Raytheon corp., http://www.raytheon.com/products/htms/, Accessed July 17, 2003.
6. Nassereddine, I. Toronto – Transportation Systems for the 21st Century, ITE Journal, May 1998, pp. 32-36.
7. Highway 407 Express Toll Route (407 ETR). http://www.407etr.com/. Accessed July 17, 2003.
8. Toll Road News, http://www.tollroadsnews.com/taxonomy/term/43., Accessed July 27, 2010.
9. Gilbert, P.R. NJ Turnpike's Exit 1 to have high-speed E-ZPass lanes. ITS America, October 5, 2000, http://www.itsa.org/etm.html. Accessed July 17, 2003.
10. 91 Express Lanes. http://www.91expresslanes.com/#, accessed July 17, 2003.
11. Miami-Dade Expressway Plan Includes Open Road Tolling. http://www.itsa.org/ITSNEWS.NSF/0/b5145e74e169288a85256c630069b30f?OpenDocument. Accessed July 17, 2003.
12. Sawgrass Expressway. http://sawgrass.urs-tally.com/main.asp. Accessed July 17, 2003.
13. Broward County, FL http://www.hhh.umn.edu/centers/slp/projects/conpric/projects/broward.htm. Accessed July 17, 2003.
14. Florida's Turnpike Enterprise, About Us: System Description. http://www.floridasturnpike.com/about_system.cfm. Accessed July 20, 2010.
15. Miami-Dade Expressway Authority. http://www.mdxort.com/. Accessed July 20, 2010.
16. Florida's Turnpike Enterprise. http://www.floridasturnpike.com/all-electronictolling/index.html. Accessed July 20, 2010.
17. Orlando-Orange County Expressway Authority (OOCEA), General Statistics http:www.oocea.com/, Accessed September, 2003.
18. Worrall, H.W. Central Florida Experiences Significant Benefits from Electronic Toll Collection, ITE Journal, June 1999, pp. 39-45.
19. Mohamed, A., M. Abdel-Aty and J. Klodzinski. Safety Considerations in Designing Electronic Toll Plazas: Case Study. ITE Journal, March 2001.
20. Klodzinski, J. and H. Al-Deek. New Methodology for Defining the Level of Service for a Toll Plaza. ASCE Journal, Vol. 128 No. 2, March 2002.
21. Hutchings, B. Construction Costs for University Plaza ORT Conversion. Orlando-Orange County Expressway Authority, E-mail response, July 26, 2006.
22. Transportation Research Board. "Highway Capacity Manual." Transportation Research Board Special Report 209, Washington, D.C., 2000.
23. Navon, D. "The paradox of driving speed: two adverse effects on highway accident rate." Accident Analysis and Prevention 35 (3), pp. 361-367, May 2003.
24. Feng C. "Synthesis of studies on speed and safety." Transportation Research Record No. 1779. Pg. 86-92. Transportation Research Board. Washington DC. 2001.
25. Abdel-Aty M. and Pande A. Identifying crash propensity using specific traffic speed conditions, Journal of Safety Research, Vol. 36 No.1, pp. 97-108, 2005
26. Abdel-Aty M., Uddin N. and Pande A. Split Models for Predicting Multi-Vehicle Crashes during High-Speed and Low-Speed Operating Conditions on Freeways, Journal of the Transportation Research Board, accepted Feb 2005.
27. Abdel-Aty, Mohamed; Uddin, Nizam; Pande, Anurag. Split Models for Predicting Multivehicle Crashes During High-Speed and Low-Speed Operating Conditions on Freeways Transportation Research Record No. 1908. Transportation Research Board. Washington DC. 2005.
28. AAA Newsroom, http://www.aaanewsroom.net/Main/Default.asp?CategoryID=7&ArticleID=601. Accessed June 2010.
29. Klodzinski, J., H.M. Al-Deek, and E. Gordin "Evaluation of Impacts from Deployment of an Open Road Tolling Concept for a Mainline Toll Plaza", Transportation Research Board, TRR 2012, Washington, D.C., January 2007.

The structural design of Almas Tower, Dubai, UAE

Ranjith Chandunni
Senior Structural Engineer and Associate
Atkins Middle East

Farshad Berahman
Senior Structural Engineer
Atkins Middle East

Abstract

The Almas Tower is a 360m high office tower in Dubai, UAE. The design comprises two intersecting elliptical towers located on a sculpted three-storey podium. The architectural form and client's requirement for floor efficiencies of 80% resulted in significant challenges for the structural design team. This paper discusses the structural framing adopted, wind-tunnel studies undertaken – including building acceleration, lateral movements and column-shortening effects – and mitigation measures introduced. It also describes the design of the tower's spire, which features tuned mass dampers.

Introduction

The Dubai Multi Commodity Centre's Almas Tower is a 360m high slender office tower located in the Jumeirah Lake Towers development in Dubai, United Arab Emirates (UAE) (Figure 1). The building consists of five basements, two podium levels, 60 storeys of offices and three mechanical floors. It has a total floor area of approximately 85,000m².

A typical tower floor plan is in the form of two diagonally offset ellipses, with a floor plate approximately 64m long and 42m wide (Figure 2). The floor plan from level 53 to level 64 consists of only one of the two ellipses. An 81m slender spire peaks at 360m, forming the highest point in the development.

The building was completed in September 2008 and according to the Council for Tall Buildings and Urban Habitat (CTBUH), was the world's second tallest building completed that year.

Structural system

The following constraints had a significant impact on the structural design of the tower and were considered at concept stage design

- the office floors to have an efficiencyof not less than 80%
- flexible column-free and wall-free office space
- final sellable area to be within ±2.5% of the area sold by the client to ultimate office owners
- each office floor to be capable of supporting a 2.5t safe placed anywhere within an office space.

The principal structural framing consists essentially of a tube-in-tube system. This is made up of a reinforced concrete peripheral frame and a central core wall, which are connected to each other by central spine beams on each floor and outrigger walls at service floor levels.

A parametric study of the effectiveness of different arrangements of the external frame, belt walls and outrigger walls was carried out and the findings are shown in Table 1.

The peripheral frame consists of 1000mm deep, 500mm wide beams supporting precast units which span onto peripheral columns. The columns are at a maximum spacing of 5m and form part of the lateral load resisting system. The columns are designed compositely in the lower half of the building to keep the column sizes small compared to what would be needed for a reinforced concrete column alone (Figure 3).

A typical floor slab consists of 320mm thick hollow-core precast panels with 80mm thick structural topping. It ties the external frame to the central reinforced concrete core walls or central spine beam. The floor is also designed to act as a diaphragm, transferring lateral wind and

Table 1 - Parametric analysis to study the effectiveness of the structural system

Parameters	Core wall	Core wall + peripheral frame	Core wall + peripheral frame + belt walls + outriggers
Natural period: s	14.6	12.2	9.6
50y wind sway: mm	1785	1258	771

Figure 1 - The 360m high Almas Tower in Dubai, UAE was completed in 2008

seismic forces to the central core and external frame. The precast slab option was chosen because of programme benefits: they are comparatively lightweight and provide uninterrupted space for services.

The plant floors at levels 42, 121, 212 and 279m above ground are 450mm thick solid reinforced concrete slabs, with the roof to the plant floors being a 400mm thick solid reinforced concrete slab (except the top plant floor) to provide an acoustic barrier to the floor immediately above.

The building was designed to British standards, while UBC-97[7] was used for seismic load assessment in accordance with local authority requirements. The concrete grades range from 45 to 70MPa cube strength with a reinforcement grade 460 (f_y = 460 MPa) and structural steelwork S355 (f_y = 355 MPa).

Finite-element modelling

A three-dimensional finite-element model of the tower and podium was generated in Etabs[5], which included the raft slab on spring supports to simulate the piles – although the raft weight was not considered for the purpose of assessing the seismic base shear. An allowance was made in the section properties for cracking under ultimate limit state according to UBC-97 and it was assumed that all loads would be transferred to the ground through the piles.

The spring stiffness for the piles was based on the pile working load capacity and the theoretical settlement of the pile under that load, which was taken from the geotechnical assessment. The effect of the podium on the lateral movement was considered by modelling lateral springs at various levels based on the stiffness of the podium structure.

The belt walls and outrigger walls include large service openings to allow for air intake and discharge as well as to allow for ductwork and piping routing (Figure 4).

The outrigger walls, if constructed along with the floors, would have transferred significant dead loads from the peripheral columns onto the core, in addition to which the outriggers would have attracted forces due to differential axial shortening between the core and the peripheral frame. To

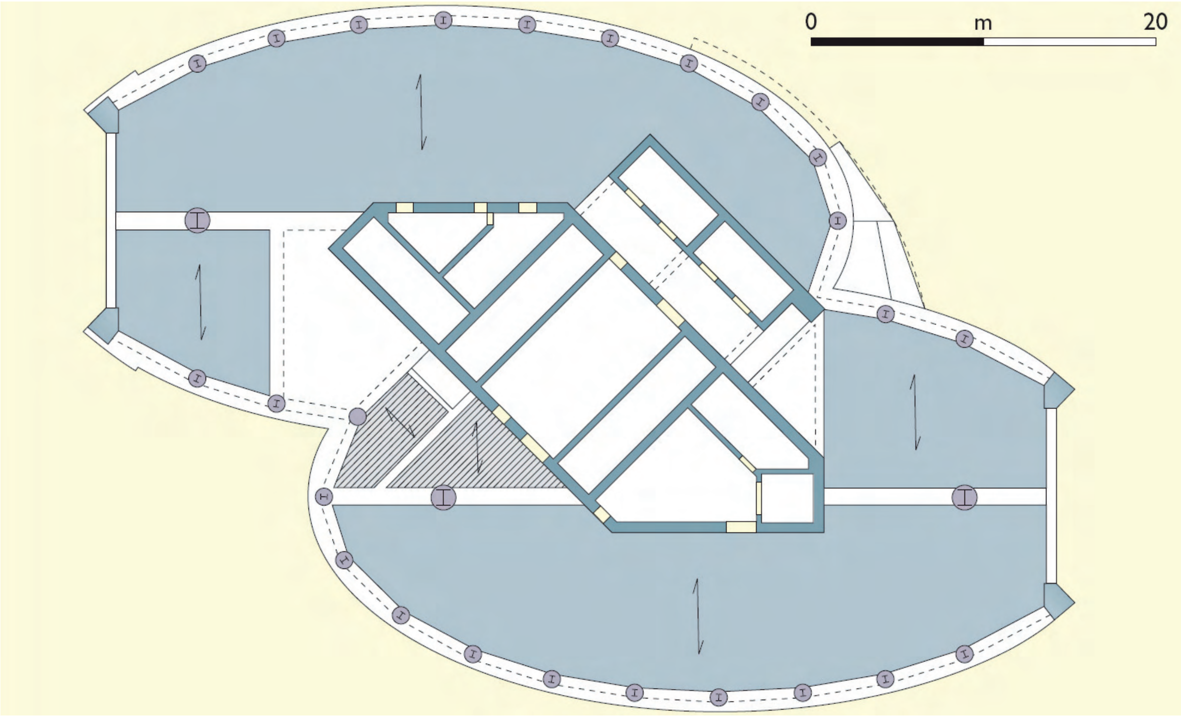

Figure 2 - Typical structural floor plan up to level +232m showing hollow core slab supported on external beams to core walls and internal beams

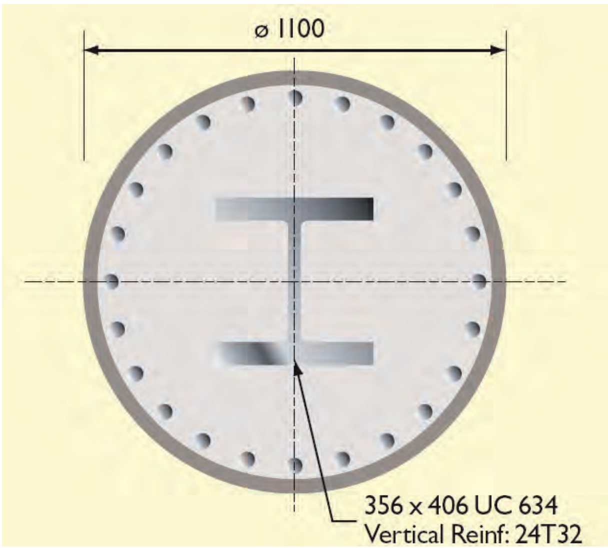

Figure 3 - Cross section through a typical composite column (dimensions in mm)

overcome this, the outrigger walls were disconnected from the floor slab above and on one vertical side until all of the floor slabs were cast, which significantly reduced the load transfer due to dead load and minimised differential shortening between the core and the frame (Figure 5).

Wind engineering

Wind tunnel testing was carried out by Rowan Williams Davies & Irwin Inc.[9] using a high-frequency force-balance model (Figure 6) with wind loads based on a 3s gust wind speed of 37.7m/s for open terrain at 10m height, in accordance with measurements at Dubai International airport between 1983 and 1997. The proximity model was based on a 575m radius.

The model was placed on a turntable and was rotated at 15° intervals to determine wind loads for 24 directions. Structural properties such as mass, mass distribution, mode shapes and frequencies were obtained from the structural analysis model and input to assess overall structural loads, building acceleration and cladding pressures. A damping value of 2% was assumed for the calculations. The tests provided overall structural loads using 24 load combinations taking into account directional effects for each sector.

The expected deflections under 50 year winds were greater than H/500. As a result, soft joints are provided between the blockwork walls and the structure – in accordance with BS 8110-2[2] - to allow for racking movement between adjacent storeys under wind loads.

The expected building accelerations at the top floor for a 10 year return period were 18.7mg, which is within the commonly accepted threshold of 23.4mg (Table 2) as per ISO criteria. A sensitivity check for a 1.5% damping resulted in an acceleration of 21.6mg – or an increase by a factor of √ (2/1.5) – which was still within acceptable limits.

The overall cladding pressure results gave a maximum value of 4.5MPa in certain local areas.

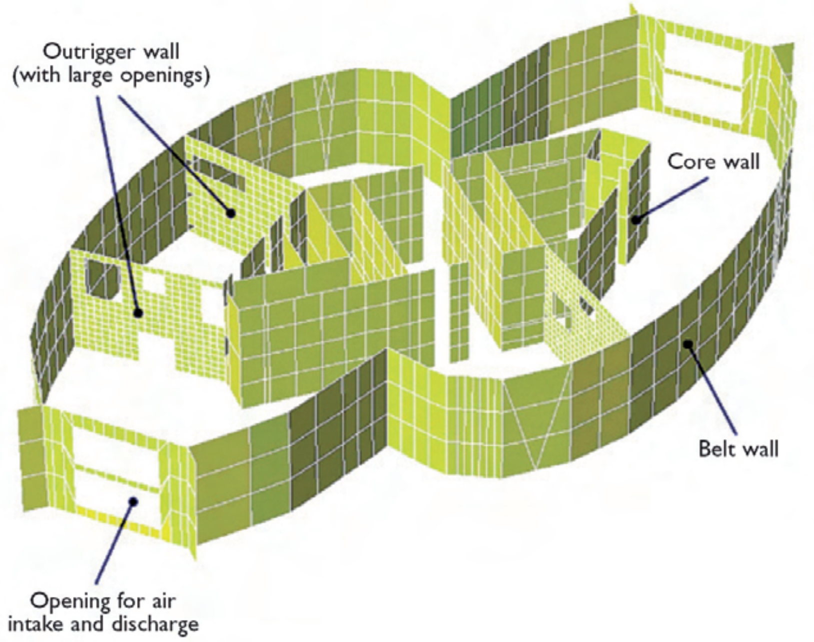

Figure 4 - Extract of three-dimensional finite-element model of a plant floor showing core walls, outriggers with openings and external belt walls

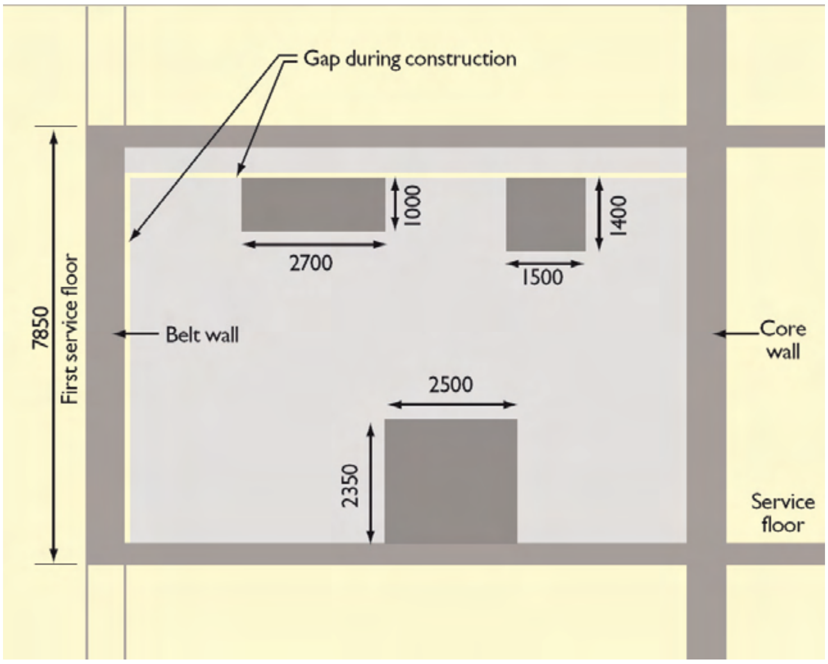

Figure 5 - Elevation of a typical outrigger wall with service openings – gaps in the wall adjacent to slab and belt wall were provided to disengage outrigger during construction to minimise differential shortening effects (dimensions in mm)

Seismic design

Seismic loads used were based on UBC-97 zone 2A in accordance with local authority requirements. A response spectrum analysis based on UBC-97 was carried out with appropriate scale factors used to obtain member forces and associated drifts.

Section modifiers as per UBC-97 were applied to the design, that is, 0.7 for uncracked walls and columns, 0.35 for cracked walls and 0.35 for beams. Ductile detailing for the coupling beams using diagonal reinforcement was specified according to UBC-97, although this is not required for zone 2A.

Foundation system

The foundation for the tower is a 3m thick piled raft supported on 1200mm diameter friction piles, which are approximately 40m long. To mitigate the effect of the heat of hydration, 50% of the raft cement was replaced with ground granulated blastfurnace slag (ggbs). Appropriate concrete cover was provided for the foundation and perimeter retaining walls to achieve the intended building design life.

The columns and walls in the podium area are supported by slabs spanning between pile caps and, to reduce the slab thickness, tension piles are designed to resist uplift in the podium basement caused by the high water table.

Vertical asymmetry

The tower has an inbuilt vertical asymmetry due to one part of the tower extending 12 floors above the other while connected to one core throughout the height of the building. It was realised early on in the design that there could be lateral movement in the building that would be in excess of sways in a conventional, symmetrically loaded building.

Building movement monitoring was included in the specification to allow the structural designers to compare actual movements with those estimated. This required survey points at each floor, which were monitored by laser surveying instruments (Leica TPS700) for lateral drift against a fixed benchmark located at ground level outside the building. Further

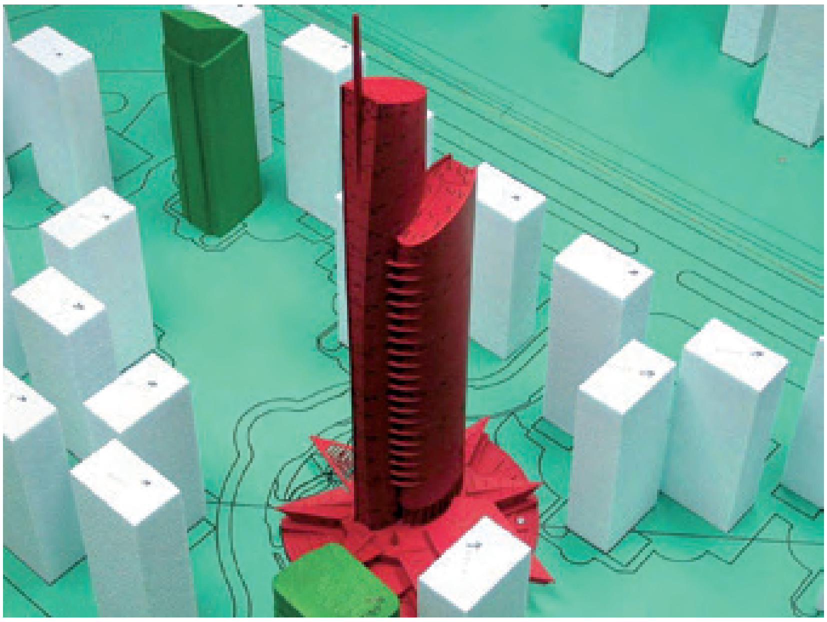

Figure 6 - Scaled model of Almas Tower used in wind tunnel studies

Table 2 - Predicted building acceleration

Return Period: y	Peak total acceleration: milli-g	ISO criteria: milli-g
1	12.1	14.0
5	16.7	19.5
10	18.7	23.4*
* The criteria for a 10 year return period is not provided in ISO and has been extrapolated		

points were located at the core and periphery of each floor to monitor movement due to axial shortening.

The results from the three-dimensional analysis model indicated that the horizontal gravity load sway would be of the order of 225mm (short term) at the uppermost floor level. This value is an overestimate as the model assumes the structure is built and then all loads are applied instantaneously – a phenomenon known as 'switch-on gravity' – which is obviously not realistic and required further investigation. Furthermore, the analysis model does not take into account the fact that:

- all floors are cast horizontally at the level shown on the design drawings, thereby reducing the differential shortening calculated in the analysis model
- vertical elements are built plumb with reference to a fixed benchmark at ground level, thereby reducing the sway calculated in the analysis model
- effects of time-dependent factors such as creep, shrinkage and age of concrete
- modulus of elasticity achieved for concrete is usually 30% higher than codified values
- compressive strength of concrete achieved is normally 10% higher than specified values.

Long-term dead load sway

Whereas a long-term dead load sway of H/1000 would be deemed acceptable, further assessments were carried out to get a better estimate of the anticipated gravity load sway.

The following procedure was undertaken to estimate dead load sway of the building.

Sway analysis was carried out using a full model of the building, with construction sequence analysis performed from levels +236m to +279m. The instantaneous dead load sway was 150mm at +236m and 225mm at the uppermost floor (+279m).

The effect of time-dependent creep was allowed for by a reduction in the effective elastic modulus. Calculations were carried out based on the principle of area moment to work out a multiplier on lateral movement. This resulted in a net increase of 110% on the calculated value of lateral movement due to load from level +236m and above.

In applying a creep coefficient of 1.1, long-term deflections were found to be 472mm (H/590) at level +279m and 315mm (H/750) at level +236m.

As a result of the calculated long-term sway values based on codified material properties, the core walls in the taller portion of the building were thickened.

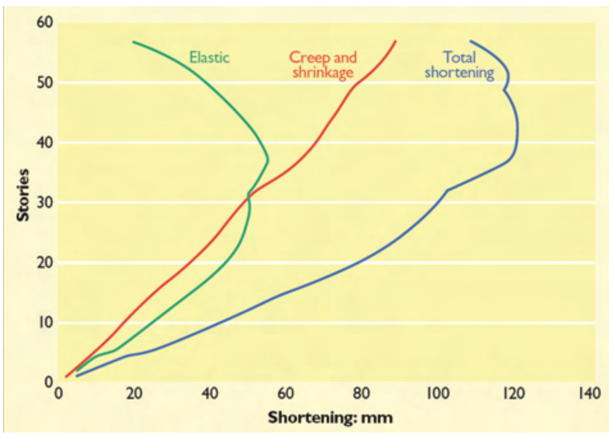

Figure 7 - Long-term settlement curve for elastic shortening, creep and shrinkage as well as the total shortening of a typical column

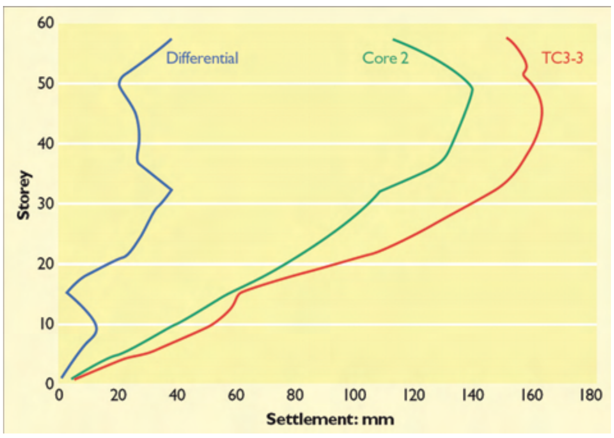

Figure 8 - Settlement of a typical tower column (TC-3) and the core wall as well as the differential settlement between the two typical column and core

STRUCTURES

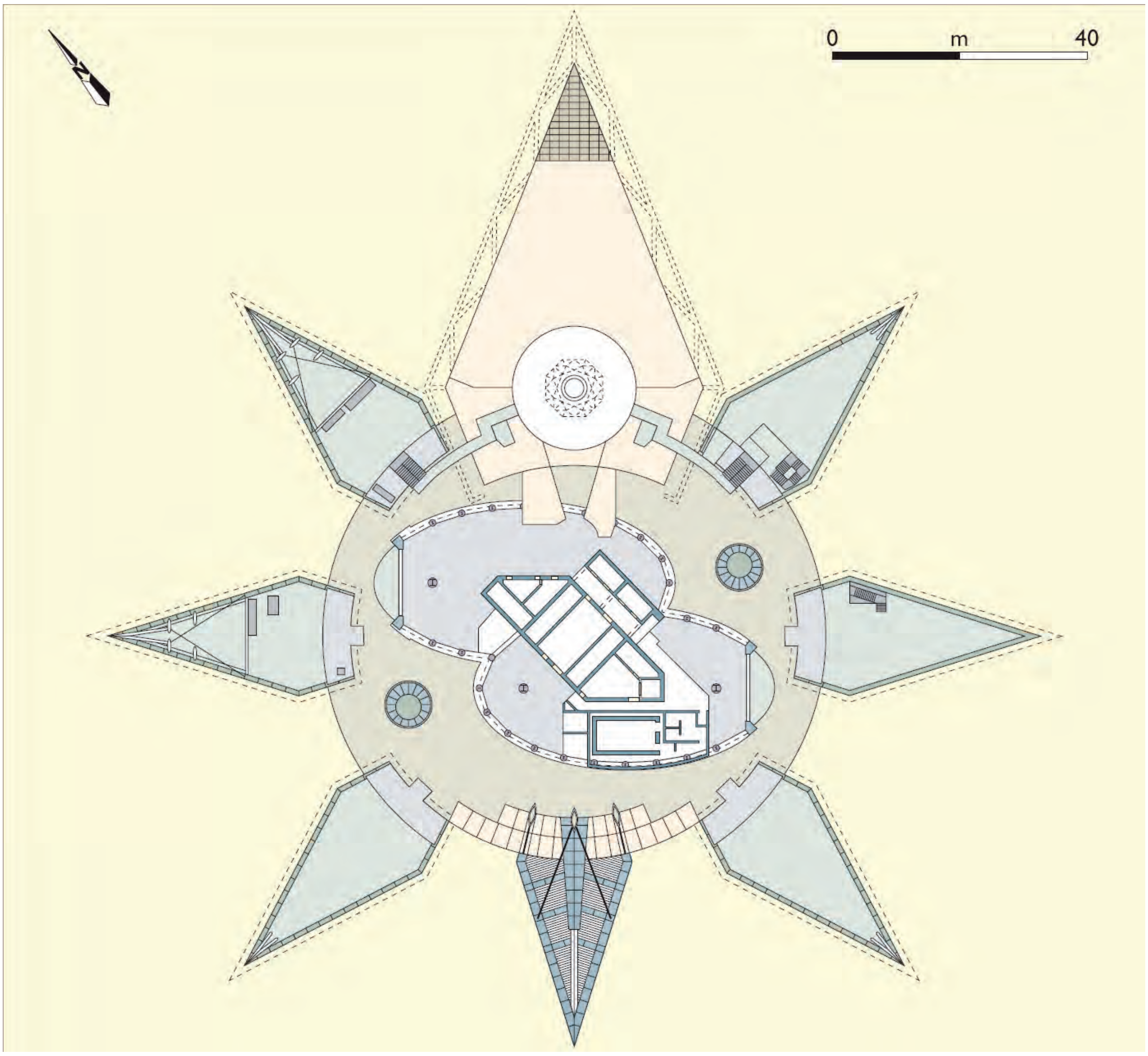

Figure 9 - Plan view of podium – the largest of eight triangular-shaped petals radiating from the central core accommodates a diamond exchange centre

The actual material test results from site demonstrated that the values of modulus of elasticity used were about 35% higher than the codified values and the compressive strength of concrete was about 10% higher than design strength. Considering these factors, the long-term deflection at completion of the last floor was estimated as 189mm (H/1470) at level +279m and 126mm (H/1900) at level +236m. These values are well within the acceptable range of H/1000.

Although the analysis indicated that in theory the sway would be within acceptable limits, the contractor monitored the verticality of the building during construction and made adjustments to account for this sway by casting floors level to the ground benchmark. Adjustments were relatively small for the lower symmetrical part and relatively large for the upper asymmetrical part of the building.

Lateral sway recorded on site immediately upon completion of the last floor was 55mm (H/5070) at level +279m and 53mm (H/4450) at level +236m.

The procedure demonstrated that with simple compensation techniques on site, lateral dead load sway can be controlled to a large extent. Other factors such as stiffening effects of internal block wall partitions and façade elements might have also contributed to the reduction in sway.

Long term axial deformation and differential shortening

As with all tall buildings, it was necessary to estimate the long-term axial deformation (see Figure 7) and the differential shortening between the core and the columns (see Figure 8). This deformation has an impact on the design of connecting elements and also requires adjustments while casting the floors to ensure that the floors are horizontal.

Two approaches were considered – one using the Eurocode 2[3] and another using the American Concrete Institute 209[1] model.

The Eurocode method does not take into account the relaxation of creep due to the presence of reinforcement, whereas the ACI model does. This was a significant

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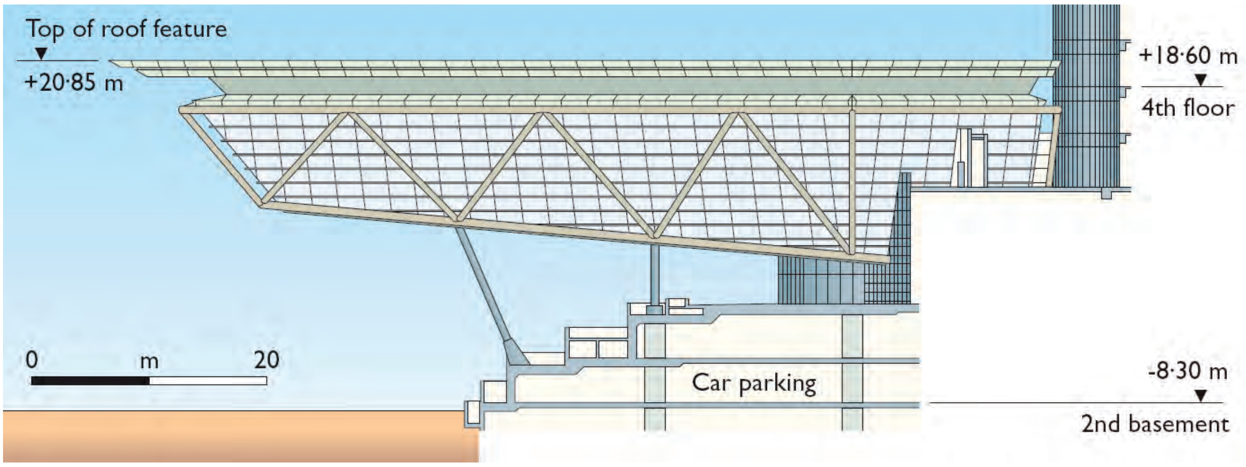


Figure 10 - Elevation of diamond exchange centre overhanging cascading podium landscape

factor in this building considering that the columns were composite and contained significant amounts of reinforcement and that the walls were heavily reinforced at lower levels.

It was therefore decided to use the ACI method for the assessment. A computer program was developed based on the procedure outlined in the ACI as well as that presented by the Portland Cement Association[6].

The program assessed the long-term axial shortening of columns and walls by considering elastic shortening, creep, shrinkage and by allowing for the fact that floors would be cast levelled to the position indicated on the drawings. The calculation was carried out for the core walls and columns and the difference was allowed for in the construction of the floors between the cores and the columns. A typical difference between the core walls and one column is shown in Figure 8.

Figure 11 - Temporary device to suppress vortex shedding on the spire during construction of the spire and installation of its tuned mass dampers

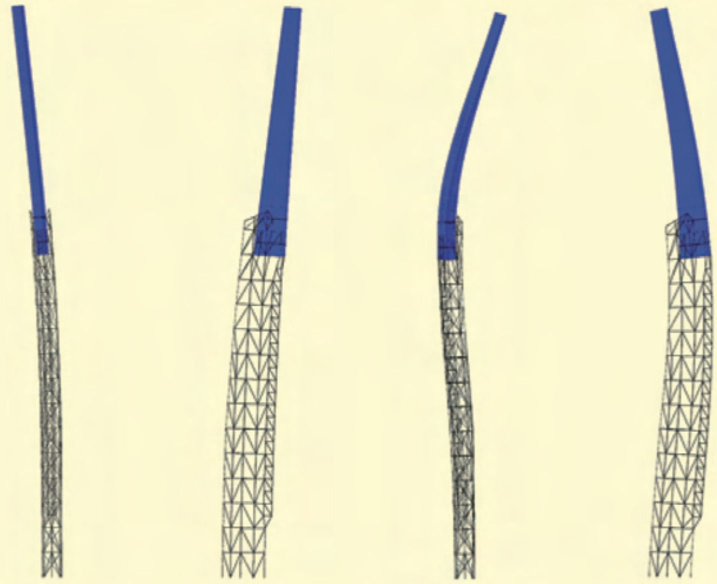
Cladding

The building façade consists of a unitised cladding and curtain walling system, which is manufactured incorporating aluminium, glass and insulation as a complete module.

The vertical spacing was based on a floor-to-floor height of 4m while the horizontal spacing was based on a combination of the structural grid and the widths of offices to achieve a maximum number of vision bays.

A 10mm gap was provided between each panel for thermal and seismic movement as well as the long-term movement of the concrete frame due to creep and shrinkage. The full depth of the mullions, including the glazing or cladding is 150mm, which left a tolerance of 25mm either way in the 200mm structural depth allowance for cladding fixing.

Table 3 - Dynamic performance of spire

Parameters	Mode 1 (minor axis)	Mode 1 (major axis)	Mode 2 (minor axis)	Mode 2 (major axis)
Frequency predicted: Hz	0·64	0·70	2·47	2·87
Frequency measured: Hz	0·51	0·70	2·20	2·70
Mode shape				
Required damping	3%	3%	2%	2%
Achieved damping	4%	4%	>5%	>5%

Podium

The design of the podium was inspired by the inherent angular geometry of a diamond. The podium comprises an array of eight triangular glass petals that radiate from the central core (Figure 9). The diamond exchange is accommodated within the north-eastern petal that juts out over a terraced water feature stepping down to a lake. The three-storey podium accommodates a variety of retail spaces and food courts, a business centre, a health club and the diamond exchange centre.

Each triangular retail petal consists of a steel framed structure with composite metal deck floor slabs. The diamond exchange centre comprises profiled metal-deck slabs supported on a grid of steel beams. These are in turn supported by an exposed steel truss and steel columns, which are straight or raking (Figure 10). The steel frames are stabilised by portal action and by the central core of the building to which they are connected.

The ground floor slab is formed by a 140m diameter stepped floor, which radiates from the central tower to the perimeter. Close to the tower, the podium is at ground level, gradually stepping to basement 1 level towards tower entrance and basement 2 towards the rear side.

The ground floor slab is a single unit of 750mm thick folded concrete plate. All the basement columns outside the tower terminate at ground floor slab level. These columns form a regular grid of 17.5m x 8m to incorporate the client's requirement of a large column free basement parking area. The folded slab supports the landscape loading and planted columns from the retail units and diamond exchange centre.

A three-dimensional finite-element model generated using the Robot package[8] was used for analysis and design of the slab. No expansion joints were provided – the slab is exposed externally and therefore designed for stresses due to seasonal temperature variations of ±20°C. All podium columns as well as perimeter retaining walls were designed for additional lateral loads due to thermal movements.

Spire structure

The top of the Almas Tower features an 81m tall spire, which forms the tip of the building reaching to a height of 360m. The base of the spire is connected to the tower through a reinforced concrete upstand wall over a length of 21m, such that the free-standing length of the spire in one direction is approximately 60m.

The spire has two distinct sections with a step transition approximately two thirds of the way up. The lower portion is roughly elliptical in shape and is constructed from a triangulated steel frame with aluminium cladding. The major and minor axis of the ellipse is 7.4 and 3.4m at the base respectively. The 23m high upper portion of the spire is of sheet steel construction. The major and minor axis of the upper portion is 4.3 and 1.5m at its base respectively.

The spire structure was initially analysed and designed for code-calculated static wind forces. However, due to the slender nature of the spire, it was realised early on in the design that without additional damping, the spire would vibrate excessively causing unacceptable fatigue stresses that could lead to structural failure.

A dynamic analysis was carried out to assess the susceptibility of the spire to a number of wind-induced excitations, such as galloping, flutter, wind turbulence and vortex shedding in accordance with provisions in Eurocode 1[4]. It was found that the critical wind speed for vortex shedding was below those expected on site and hence did have the potential to excite the spire. Significant deflection amplitudes and fatigue stresses were estimated for the first two modes in each direction (major and minor) based on the codified intrinsic level of damping of 0.5%.

The calculations were re-run with increasing levels of damping to establish the minimum damping values required in each mode and direction to produce an acceptable fatigue life.

The properties of tuned mass dampers necessary to provide the required level of damping were then determined. Four 2t dampers located close to the top of the spire were installed. Table 3 shows the mode shape, frequency and required and achieved damping values. The measured frequencies showed good agreement with the predicted frequencies. The provided damping of approximately 4% in the first mode and more than 5% in the second mode was confirmed by site measurements when the dampers were fine tuned.

The design was then verified upon completion of the dynamic study with dynamic wind forces corresponding to proposed damping. The Robot software was used to perform the static finite element analysis and for member design of the structure and an Etabs model was run in parallel to verify the modal properties of the spire.

The top part of the spire needed to be installed in stages because of its weight. The dampers were locked, that is inactive, until the damper supplier was allowed access to unlock and tune them. Unpredictable winds during the installation could have led to an extended installation period until the dampers could be activated, which meant that excessive vibration could occur in that time. Therefore a vortex shedding suppression device (see Figure 11) was attached around the spire until the dampers were activated. This device caused wind turbulence in this area rather than alternate vortex shedding and hence prevented excessive movement of the spire.

Conclusion

The design of Almas Tower evolved from meeting the client's high expectations and stringent requirements. The solution adopted by the design team proved to be most effective in producing a floor usage of the highest efficiency.

Detailed studies were carried out to address issues arising from the sway due to vertical asymmetry of mass. This was overcome by using simple compensation techniques during construction, thereby demonstrating that a practical approach can be successful in addressing such issues.

Vortex shedding suppression devices based on simple principles were used as temporary measures during the construction stage to prevent excessive wind induced movement of the spire.

Acknowledgements

The key members of the project team were: client - Dubai Multi Commodity Centre, architect, building services engineer, fire, life and safety, acoustics, structure - WS Atkins Dubai, dynamic analysis of spire – WS Atkins Epsom, wind tunnel specialist – RWDI, third party structural reviewer – Halcrow Yolles, project management – Faithful & Gould, contractor joint venture – Taisei Corporation and Arabian Contracting Company, tuned mass damper supplier – Gerb.

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References

1. ACI Committee 209 (1997) Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures. American Concrete Institute, Farmington Hills, MI.
2. BSI (British Standards Institution) (1985) BS 8110 Part 2 – Structural Use of Concrete. Code of practice for special circumstances. BSI, Milton Keynes.
3. BSI (2004) BS EN 1992-1-1 Design of Concrete Structures – General Rules for Buildings. BSI, Milton Keynes.
4. BSI (2005) BS EN 1991-1-4 Action on Structures – General Actions – Wind Actions. BSI, Milton Keynes.
5. CSI (Computers and Structures Inc) (2005) ETABS nonlinear version 8.5, extended 3D analysis of building systems. CSI, CA, USA.
6. Fintel M, Ghosh SK and Lyengar H (1987) Column Shortening In Tall Structures – Predictions and Compensation. Portland Cement Association, Stokie IL, USA.
7. ICBO (International Conference of Building Officials) (1997) Uniform Building Code, UBC-97. Whittier, CA.
8. Robobat (2005) Robot Millenium Software Package. Robobat, USA.
9. RWDI (Rowan Williams Davies & Irwin Inc.) (2005) Wind-induced structural responses, DMCC Almas Tower, Dubai, UAE. RWDI, Ontario, Canada.

Preliminary steel concrete composite bridge design charts for Eurocodes

Rachel Jones

Senior Engineer

Highways & Transportation
Atkins

David A Smith

Regional Head of Bridge Engineering

Highways & Transportation
Atkins

Chris Dolling

Technical Development Manager

The British Constructional Steelwork Association

Abstract

The switch to Eurocodes from April 2010 required the development and updating of many existing design tools. For many years Corus, and British Steel before them, have published preliminary design charts for steel-concrete composite highway bridges as part of their suite of design guidance for bridge engineers. These charts were originally developed using BS 5400 and the Highways Agency's Design Manual for Roads and Bridges (DMRB). This paper describes the development of a new set of charts based on the structural Eurocodes. The new charts take advantage of benefits in efficiency permitted by the Eurocodes and also extend the scope of the original charts. The process adopted to generate the data for the charts is described and the key differences between the BS 5400 design approach and the Eurocode approach are discussed.

Project background

The structural Eurocode program started in 1975 with the aim of removing technical barriers to trade in the European Union. The Eurocode parts and UK National Annexes required for bridge design have now all been published. Projects tendered after March 2010 under the EU Public Procurement Directive are required to use the Structural Eurocodes. A consequence of the change is that a large proportion of guidance and software currently available needs to be revised. This provides an opportunity to improve on existing design tools and enable more efficient designs. As part of the transition to the Eurocodes, Corus, the British Constructional Steelwork Association (BCSA) and the Steel Construction Institute (SCI) have revised the guidance they publish for bridge designers.

This guidance includes a set of charts that can be used to establish the plate girder sizes for steel-concrete composite bridges[7]. These can be used at the preliminary design stage to establish approximate steel quantities and to obtain initial sizing for design iteration. The original charts allow the user to obtain top and bottom flange areas and the web thickness for steel-concrete composite bridges designed to BS 5400 Part 5[1] using highways loading to the DMRB standard BD37/01[8].

This paper describes the commission to revise these charts for design to the Eurocodes. The new charts are intended to complement the new versions of the SCI composite highway bridge design guides and it was ensured that the design practice used for the charts aligned with that given in the SCI publications. The updating of the charts provided an opportunity to extend their scope and accuracy and enhance their presentation with an electronic format which allows instant interpolation.

Scope of the design charts

The new charts allow for the preliminary design of multi girder bridges with any number of main beams (Figure 1) and ladder deck bridges with two main beams and regularly spaced cross girders (Figure 2).

The scope of the new charts was established so that they cover most standard UK highway bridges. Table 1 summarises the scope of the charts.

Both simply-supported and continuous spans are included in the charts. For continuous spans a separate girder section is given at the support and at midspan. A difference from the original charts, for multi girder bridges is that separate designs have been established for the outer girder supporting the parapet outstand and for the inner girders. The designer can select from two different highway live loading types. Load Model 1 in BS EN 1991[3] roughly corresponds with the HA only loading from BD37/01 and Load Model 3 is similar to HA and HB loading combined.

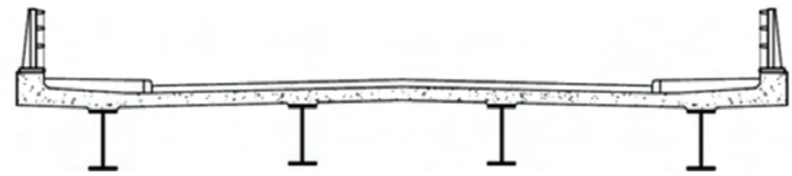

Figure 1 - Multi girder bridge - typical cross section

Figure 2 - Ladder deck bridge - typical cross section

The charts give both elastic (non-compact) and plastic (compact) designs for all situations. For each individual bridge either an elastic or plastic design will be more efficient, this choice is left to the designer's discretion. An advantage of including both designs for all situations is that it allows interpolation between discrete values on the charts, which would be prevented by having a discontinuity at the point where either the elastic or plastic design becomes more efficient.

Generating the charts - analysis

Grillage models

The original charts were derived using line beam analysis. To more accurately include the benefits of transverse distribution, the new charts are based on the results of a series of grillage analyses. The data for the charts was generated from a large number of grillage models. Figure 3 shows a typical model. Setting these up manually would have been extremely time consuming and so the process was automated.

The grillages were generated using the finite element package LUSAS[9] which allows the use of visual basic scripting. A script was written which extracted the key dimensions and section properties of each grillage from a defining Excel spreadsheet and used these dimensions to create an appropriate grillage. This is worked in a similar manner to LUSAS's built in grillage wizard. The bespoke script then applied the loading, ran the analysis of the model and extracted the analysis results back into a spreadsheet. This process allowed a large number of similar grillages to be set up and analysed in turn with minimal user intervention.

There were a number of assumptions about typical designs that needed to be made for all the grillage models. Typical dimensions for the slab concrete and reinforcement, surfacing and parapet edge beam were selected with reference to the SCI design examples. For the ladder deck bridges appropriate sizing for the crossbeam was established for different cross girder and main girder spacing ranges and these typical values were used in the grillage models. The details of these assumptions are provided with the final charts and summarised in Table 2.

For the multi girder bridges the charts cover bridges with any number of multi girders. The results for the charts were extracted for one of the outer girders and the adjacent girder (see Figure 4). The grillage models were all set up with six main girders. This provided sufficient deck width for the effects of the parapet edge beam on the remote inner girders to be small.

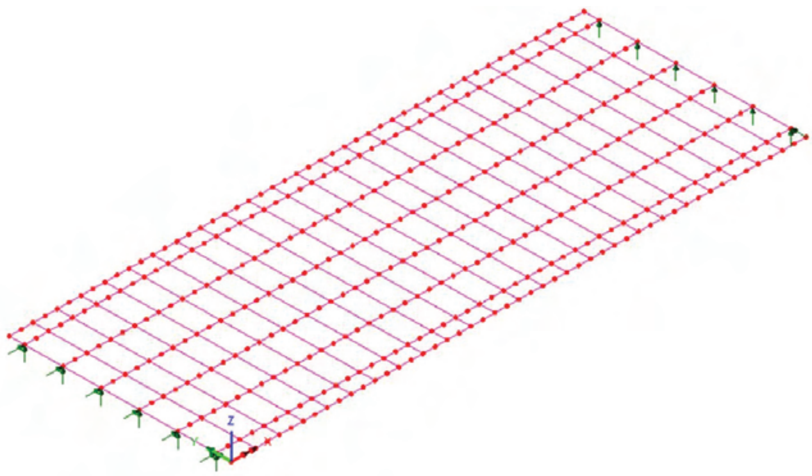

Figure 3 - Typical grillage model

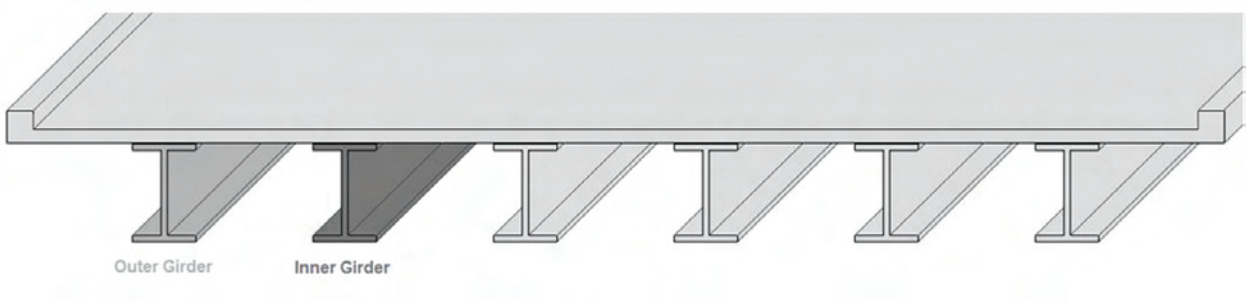

Figure 4 - Girders used for generating charts

Table 1 - Scope of charts

	Multi Girder	Ladder deck
Main girder spacing	2.5m – 4m	5m – 20m
Span	15m – 60m	15m – 60m
Span to Depth Ratio	20 – 30	20 – 30
Cross girder spacing	N/A	3m – 4m

Live loading

Live loading was applied in accordance with BS EN 1990[2] and BS EN 1991-2[3] and the corresponding UK National Annexes. It was assumed that traffic loading would control the design and so this was factored as the leading variable action with other variable actions factored as combination effects in accordance with BS EN 1990. Combination factors and partial factors for each action were taken from Annex A2 of BS EN 1990.

The two live loading cases provided in the charts are Load Model 1 and Load Model 3 (see Figure 5). Load Model 1 includes a uniformly distributed load (UDL) across the whole carriageway and tandem systems (TS) of different magnitudes applied to three lanes. The UK National Annex values calibrate the UDL to correspond approximately to BD37/01 HA loading.

Load Model 3 includes, in addition to the UDL and tandem systems, a Special Vehicle (SV) and so is similar to HA with HB loading in BD 37/01. There is a variety of special vehicles detailed in the UK National Annex to EN 1991-2. SV196 (shown in Figure 6) was used for generating the charts as this most closely corresponds to the magnitude of HB loading to BD37/01 used for trunk roads.

At the time the charts were produced a suitable commercial automatic load generation tool for Eurocode live loading was not yet available. The most adverse live loading arrangement is most efficiently established using influence surfaces. However, this method would have required a complex automation process to be developed. Instead, the live loading was generated by applying loading at all possible positions and enveloping the results to obtain the worst effects at specific locations. This increased the computational time and file sizes, but was considerably simpler to automate. If the UK National Annex is followed the magnitude of the Eurocode UDL is independent of loaded length and constant for all lanes and so this simplified approach was easier to adopt than it would have been with BD37/01 loading. The Eurocode does give scope for different UDLs in different lanes but this was avoided in the calibration of the UK National Annex.

Continuous spans

In order to obtain designs for hogging and sagging regions of continuous spans a basic span layout needed to be assumed. The arrangement used was a three span structure with end spans 70% the length of the central span. The section design at the pier support location was based on the load effects over the pier and the sagging spans section design was based on the load effects at midspan of the central span and the splice location between the two section types (see Figure 7). This arrangement gave reliable results for continuous bridges where the span

Table 2 - Key assumptions for producing charts

Key assumptions used to produce charts	
Slab/surfacing	Deck slab: 250mm average thickness.
	Longitudinal deck reinforcement: B20s @ 150mm centres top and bottom.
	Deck slab; C40/50 concrete
	No haunches on deck slab
	Parapet edge beam; 500mm x 500mm
	Cantilever; 1600mm wide
	120mm thick surfacing
	Deck slab cast in one stage
Steelwork	Steel grade S355
	Minimum top flange width 350mm
	Transverse stiffeners; provided at lesser of 8m centres or 1/3 span length
	Torsional bracing; provided at transverse stiffeners locations
	Ladder decks cross girder dimensions based on assumed approximate sizes
Live loading	Footway: 2m wide
	Special Vehicle SV196 used in Load Model 3
	Traffic loading is always the leading effect
General	Single span designs are based on a single girder size throughout its length.
	Continuous span designs are based on 3 span models with side spans 70% the length of the central span.
	For continuous bridges splices located 0.2 x main span, either side of pier.
	The steelwork is unpropped during construction. The steel and deck act compositely for all superimposed loads.
	For lateral torsional buckling during the casting of the deck slab. It is assumed that the bracing at 8 m centres provides full torsional restraint.

lengths are all roughly similar. In the grillages cracked section properties were used for a length of 0.15 x span either side of the internal supports. For the continuous spans the secondary effects of temperature and shrinkage were determined by the application of relaxation moments to the grillage models.

Generating the charts – section design

The visual basic script file generated the grillage models, applied the loading, analysed the model and extracted the moments and shear forces to a spreadsheet. The plate girder sizes were then obtained using a design spreadsheet for each section. The various design checks:

- ULS shear;
- ULS moment;
- ULS shear moment interaction;
- SLS stresses;
- lateral torsional buckling.

were programmed into the spreadsheets, so that a usage factor was returned for each check. The sections sizes were then obtained using a bespoke macro. The plate girder dimensions were gradually incremented from a minimum size, checking the usage factors at each stage until an optimised section was arrived at. The top flange was kept to a minimum practical width of 350 mm to allow sufficient space for the shear connectors to the concrete slab.

Section classification

BS 5400 divided plate girder sections into compact and non-compact classes. Compact sections can accommodate plastic compressive strains without local buckling whereas non-compact maximum moments of resistance are limited by first yield. The original plate girder design charts to BS 5400 were based on non-compact section designs. The new charts provide both compact and non-compact designs, although the terminology and treatment of section type in the Eurocodes is different. Sections are divided into four classes. Class 1 and 2 sections can reach their full plastic resistances and are equivalent to compact sections. Class 3 and 4 sections buckle before they

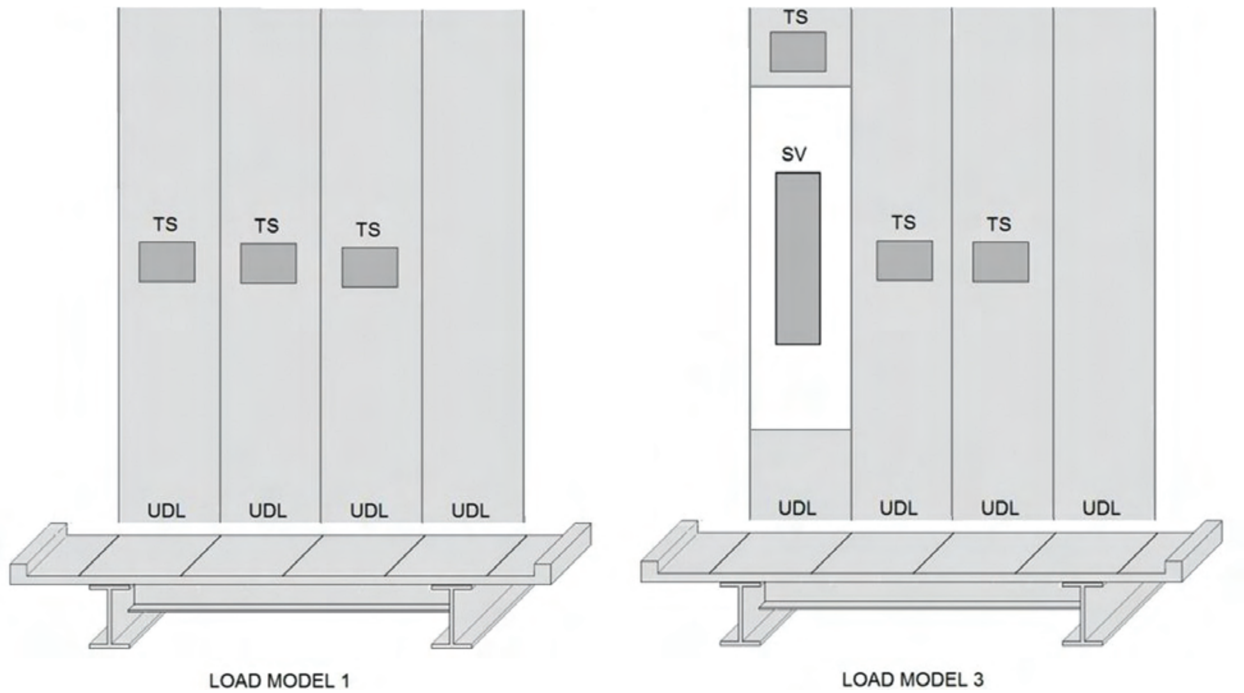

Figure 5 - Arrangement of traffic live loading

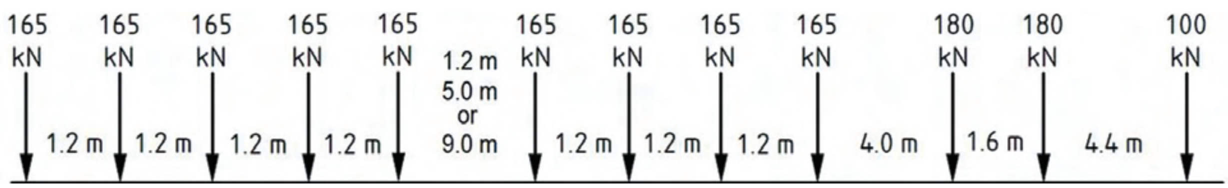

Figure 6 - Details of Special Vehicle 196

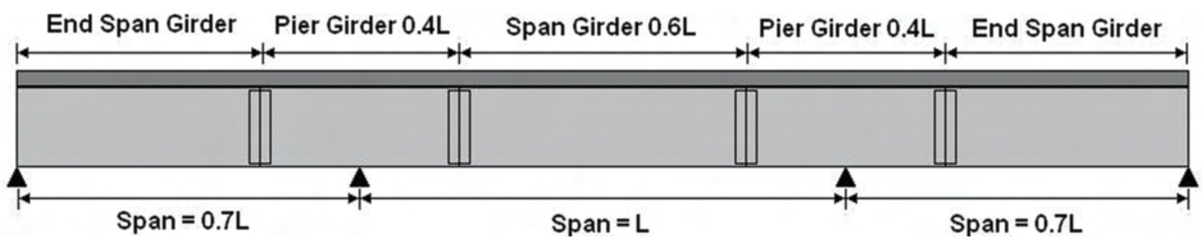

Figure 7 - Continuous span arrangement

reach their plastic resistances and their section design is limited to the material yield strength. The difference between Class 3 and 4 sections is that Class 3 sections reach their full elastic resistances, whilst Class 4 sections are so slender they buckle before their elastic resistances are reached.

BS 5400 dealt with Class 4 sections by using a reduced plate thickness to obtain a reduced resistance. The Eurocodes use a reduced width or depth (introducing hypothetical holes into the cross section) to obtain the reduced strength.

To obtain plastic designs for the charts the web was kept in Class 1 or 2 by maintaining a minimum web thickness. The flanges were kept within the Class 2 limits by keeping a constant outstand ratio. For the elastic designs the flanges were kept within the Class 3 limits for simplicity.

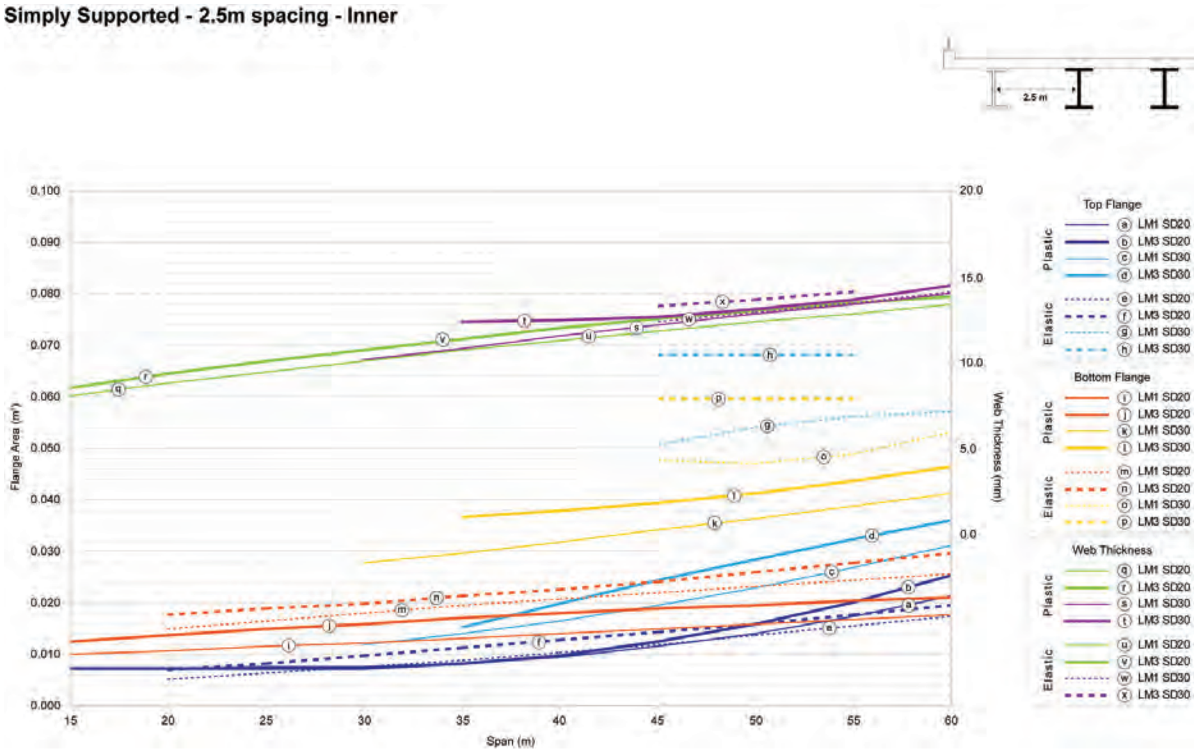

Figure 8 - Typical chart for plate girder sizing

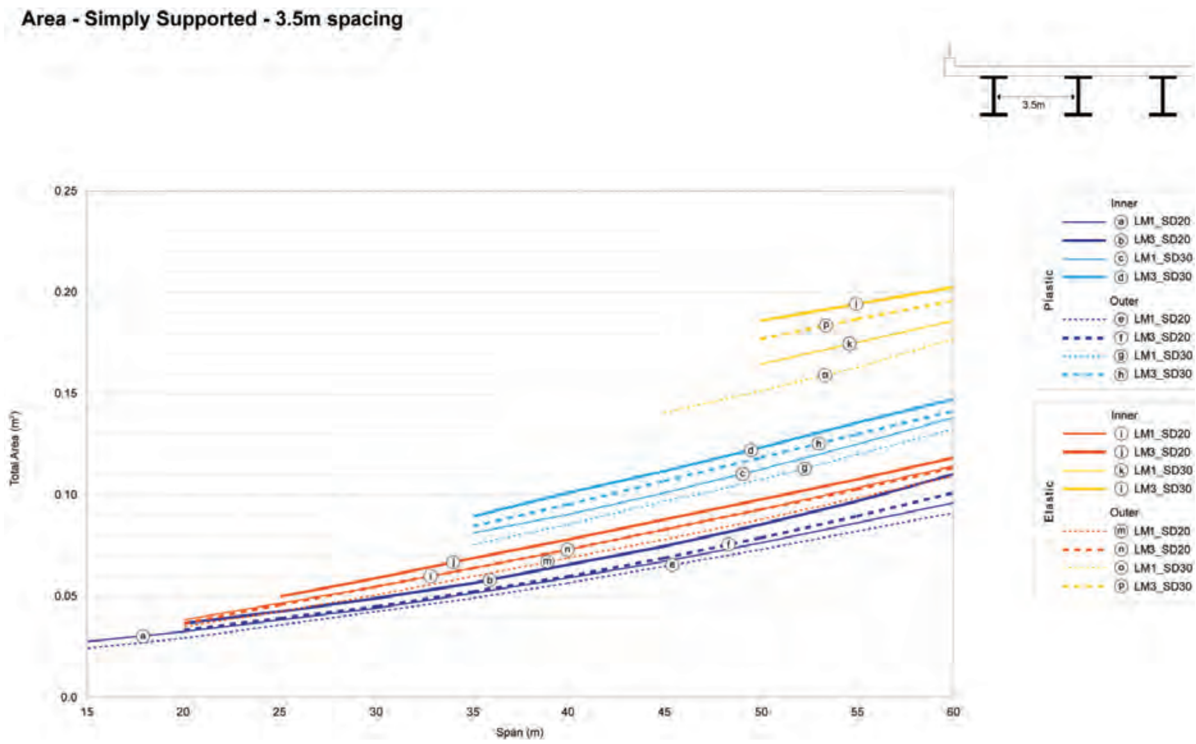

Figure 9 - Typical chart for total steel girder area

STRUCTURES

Class 4 webs were included where they give a smaller total steel area. Although the data for the charts were produced with an assumed flange width and thickness, the final charts only present a flange area and it is left to the designer to ensure the plate dimensions they choose for the flanges are within the section class limits.

Shear lag

The effective widths of the concrete flanges were reduced for shear lag to determine section properties for the grillage models and to calculate the section resistances. BS EN 1994-2[5] gives an effective width that varies linearly along the length of a bridge span. To simplify the modelling this linear variation was ignored and uniform effective widths were used for pier girder sections and span girder sections. In individual bridge designs shell elements can be used to model the deck slab eliminating the need to make reductions in section properties. This method requires more post processing to extract the moments and shears at a section and as the process was automated the simpler shear lag reduction approach was used.

ULS shear check

For stocky webs the shear resistance is independent of the web panel length. For more slender webs, where shear buckling can occur the shear resistance is dependent on the spacing of transverse stiffeners. To produce the girder designs an assumption on the spacing of transverse stiffeners was required. Based on typical standard practice in the UK, it was assumed that they would either be provided at one third points of the span, or at 8m intervals, whichever was smaller. The Eurocodes allow an increase in shear resistance based on the resistance of the flanges as well as the web. This was taken into account when determining the girder size, although it is generally a small benefit.

ULS moment check

The ULS moment resistance was calculated based on plastic section properties for the Class 1 and 2 sections and elastic section properties with any reduction required for local buckling for Class 3 and 4 sections.

BCSA

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Eurocode Preliminary Steel Composite Highway Bridge Design

Example 1 02/08/2009

INPUTS

Multi Girder or Ladder Deck?	Multi Girder
Simply Supported or Continuous?	Continuous
Span (15 - 60 m)	60
Span to Depth Ratio (20 - 30)	20.0
Girder Depth (mm)	3000.0
Multi Girder Spacing (2.5 - 4 m)	2.5
Ladder Deck Girder Spacing (5 - 20 m)	
Cross Girder Spacing (3 - 4 m)	
Load Model 1 or Load Model 3?	LM1

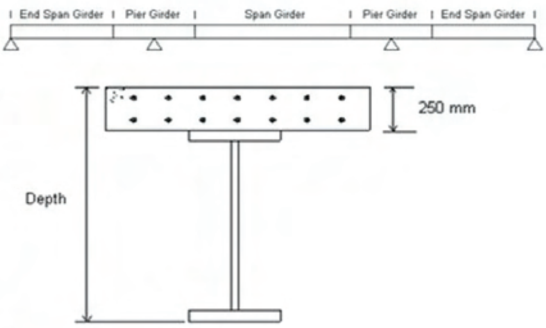


Figure 10 - Inputs for preliminary design spreadsheet

Elastic

Span Girder

Inner Girder

Top Flange Area =	5241	mm²
Web Thickness =	10.4	mm
Bottom Flange Area =	8018	mm²
Total Area =	**41879**	**mm²**

	Width	Depth
Top Flange	300	18
Web	10.4	2705
Bottom Flange	300	27
Total Area =		41652

Outer Girder

Top Flange Area =	6016	mm²
Web Thickness =	11.2	mm
Bottom Flange Area =	9749	mm²
Total Area =	**46482**	**mm²**

	Width	Depth
Top Flange	350	18
Web	11.2	2699
Bottom Flange	300	33
Total Area =		46348

Pier Girder

Inner Girder

Top Flange Area =	5835	mm²
Web Thickness =	13.1	mm
Bottom Flange Area =	23770	mm²
Total Area =	**65498**	**mm²**

	Width	Depth
Top Flange	500	12
Web	13.1	2698
Bottom Flange	600	40
Total Area =		65214

Outer Girder

Top Flange Area =	12788	mm²
Web Thickness =	14.1	mm
Bottom Flange Area =	35919	mm²
Total Area =	**87452**	**mm²**

	Width	Depth
Top Flange	400	32
Web	14.1	2670
Bottom Flange	750	48
Total Area =		86418

Figure 11 - Elastic design outputs for preliminary design spreadsheet

ULS shear-moment interaction

The approach for considering shear-moment interaction in the Eurocodes is different and less conservative than the equivalent method prescribed in BS 5400. For elastic section design the Eurocodes allow shear-moment interaction resistance checks to be based on the plastic moment resistance of the section, whilst in BS 5400 the interaction is based on the elastic resistance. This less conservative approach has been shown to be adequate in other studies[10]. In this area the Eurocodes give a more efficient elastic girder design which helps to give more economic girder sizes in the charts.

SLS stress checks

The section sizes determined based on ULS checks were checked for SLS stress limits in the plate girder and slab. The primary stresses due to temperature, creep and shrinkage were added to stresses from the global effects determined from the grillage models.

Lateral torsional buckling

The Eurocode rules governing lateral torsional buckling (LTB) are based on first principles and are less prescriptive than the rules set out in BS 5400. The Eurocode approach encourages designers to use finite element models to determine elastic critical buckling resistance rather than empirical rules. The girder sizing established for the charts needed to be checked for LTB as this can often control plate sizes if excessive bracing is to be avoided. Due to the large number of cases investigated the most rigorous approach of using shell Finite Element (FE) models was not practical. Additionally, such an approach would also require extra parameters related to bracing spacing and stiffness to be assumed and the improved accuracy would be of limited general benefit. Instead, the empirical rules of BS 5400 were used to check the sections. This approach is described in the Published Document PD 6695-2[6] which accompanies BS EN 1993. It was assumed the bracing would be co-located with the transverse stiffeners and be fully rigid. This method provided a feasible, but not overly conservative, design. It has been shown[11] that the BS 5400 rules give more conservative results than can be obtained from using an FE model and EN 1993[4]. In the detailed design stage the bracing spacing could be increased or less rigid bracing provided and the plate sizes given by the charts could still be valid.

Iteration

The section properties entered into the grillage models clearly have an influence on the design action effects extracted from them. The design process is thus iterative and the development of the preliminary design charts needed to account for this to obtain sufficient accuracy. As a starting point, a first set of grillages was created using section properties based on the original charts to BS 5400. The section designs obtained with the load effects from the first set of grillages were then used to create a second set of grillages and the sections re-designed using the updated forces from the subsequent models. No further iteration was deemed necessary within the accuracy of the preliminary design charts, although further economy may be possible for individual cases.

Checking

A number of checks were carried out to verify the sizing obtained for the charts. For a number of specific bridge layouts an independent team carried out a check of the dimensions obtained. This involved setting up separate models and carrying out independent section resistance checks. At a higher level the curves produced were checked against the existing curves to ensure the results were similar.

Presentation of the charts

The original BS 5400 charts consisted of a set of basic charts, from which the results needed to be multiplied by factors, obtained from another chart, to allow for girder spacing. As the new charts take into account transverse distribution more accurately, a similar ‘girder spacing factor’ would be dependent on both girder spacing and the span. Without losing accuracy, or increasing the complexity of use, it was not possible to recreate this approach for the new charts. The charts are thus presented as a larger number of separate charts for each girder spacing. It is intended that interpolation between charts be undertaken for the design of non-standard girder spacings.

Charts giving the required flange areas and web dimensions (Figure 8) are provided alongside charts giving the total section area (Figure 9) for each case considered. The total area charts can be used first to establish total steel quantities and whether the elastic or plastic design is more efficient. Following this, the other charts provide more detail on the girder sizing.

In addition to the standard charts a software design tool was developed. Created as a simple spreadsheet, the designer inputs the arrangement of their bridge in the input cells as shown in Figure 10. The elastic and plastic designs are then provided in the format shown in Figure 11. The spreadsheet generates the design by automatically interpolating between the data that are presented in the charts.

Using the charts

Total steel areas

For a given girder spacing, girder type, load model, span to depth ratio and span the total girder areas for an elastic and plastic design can be determined. Based on these the designer then chooses whether to use an elastic or plastic section. It should be noted that EN1994–2 clause 6.2.1.3 (2) limits the plastic bending resistance in a span girder to $0.9M_{pl,Rd}$ when elastic and plastic sections are mixed if the ratio of lengths of the spans adjacent to that support is less than 0.6. If this applies the size of the span girder would need to be increased slightly, as this has not been accounted for in the charts.

If span to depth ratios or girder spacings are required which do not match the discrete values in the charts the elastic design or plastic design areas can be obtained by interpolating between charts. (The spreadsheet does this automatically.)

The span to depth ratio given in the charts and spreadsheet tool is based on the total depth of the girder and slab, see Figure 12. For some of the shorter span arrangements the higher span to depth ratio of 30 did not give a viable design as the area

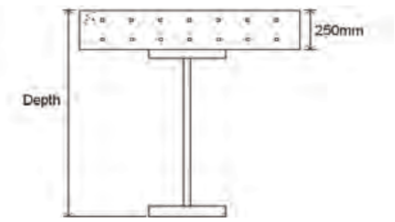


Figure 12 - Depth used for span to depth ratio

of steel required was very high. In these cases the charts have been curtailed and the uneconomically high steel areas are not shown.

Plate girder sizes

The individual flange areas and web thickness can be obtained from the plate size charts. The web depth is not given directly as it is a function of the span to depth ratio and span. When selecting the flange width and depth based on areas from the charts, the limits on flange outstands for elastic and plastic section given in Table 5.2 of EN 1993-1-1 should be taken into account. (The spreadsheet indicates compliance with this automatically.)

Continuous spans

Pier girder and internal span girder charts are provided for continuous spans. For end span girders (see Figure 7), suitable plate sizes can be obtained by looking up values for a span 25% greater than the actual end span and using the continuous span girder charts.

Skew, Curved and Integral Bridges

The charts are based on the grillage models of bridge decks with no skew or curvature and integral bridge effects have not been considered. The literature distributed with the charts gives some discussion on how these differences can be allowed for. It is important to note that little conservatism other than that required by the codes has been built into the charts, beyond general smoothing of the design curves. It has been left up to the chart user to add this at their discretion for cases outside the standard assumptions. In the case of bridge with a minor skew or curvature the charts could be used for initial sizing. Where these factors are more significant the charts still provide an order of magnitude or “sanity” check on any sizing determined by the designer.

Conclusions

The structural Eurocodes for steel-concrete composite bridge design differ from BS 5400 in a number of ways. In general these differences are based on advances in engineering understanding and adopting European best practice to allow more efficient designs to be prepared.
In order to take benefit from these advances, and to assist in the change to Eurocodes, a new set of preliminary steel-concrete composite bridge design charts has been developed. By automating the processes required it was possible to carry out more refined analyses to obtain more realistic values for the charts. The results have been presented in a traditional chart format and have also been used to develop a spreadsheet design tool that gives preliminary sizing directly.

Acknowledgements

The authors gratefully acknowledge funding for this project provided by Corus and the British Steel Construction Association.

References

1. BSI (British Standards Institution) (1978) BS 5400 : Steel, Concrete and Composite Bridges, Part 5 – Code of Practice for Design of Composite Bridges. BSI, London.
2. BSI (British Standards Institution) (2002) BS EN 1990: Eurocode - Basis of structural design. BSI, London.
3. BSI (British Standards Institution) (2003) BS EN 1991-2: Eurocode 1- Actions on Structures. Traffic loads on bridges. BSI, London.
4. BSI (British Standards Institution) (2005) BS EN 1993-1-1: Eurocode 3- Design of steel structures. General rules and rules for buildings. BSI, London.
5. BSI (British Standards Institution) (2005) BS EN 1994-2: Eurocode 4- Design of composite steel and concrete structures. General rules and rules for bridges. BSI, London.
6. BSI (British Standards Institution) (2008) PD 6695-2 Recommendations for the design of bridges to BS EN 1993. BSI, London
7. Corus Construction and Industrial (2005) Composite Steel Highway Bridges
8. Highways Agency, (2001) BD37/01 Loads for Highway Bridges, Design Manual for Roads and Bridges, Highways Agency, London
9. LUSAS Finite Element Analysis Software (2007) Finite Element Analysis Ltd, Kingston-upon-Thames. LUSAS Bridge Version 14.3
10. Presta, F et al.: ‘The Numerical validation of simplified theories for design rules of transversely stiffened plate girders’, The Structural Engineer, Nov. 2008
11. Hendy CR and Jones RP (2009) Lateral buckling of plate girders with flexible restraints. Proceedings of the Institution of Civil Engineers, Bridge Engineering 162(1): 25-33

Fatigue analysis of stay cable mono-strands under bending load

Jan Winkler

Structural Engineer

Atkins Europe

Jesper Schaarup

Regional Head of Bridge Engineering

Atkins Europe

Abstract

Large-amplitude vibrations of bridge cables have been frequently reported in the last decade. Despite extensive research on the mechanisms of dynamic excitation, it remains unclear what effect these vibrations have on the internal stresses and the fatigue life of the cable.

Currently, existing test procedures do not simulate actual field conditions, in which the cable is subjected to bending due to wind loading, parametric excitation or other forms of dynamic loading. Although fatigue spectra for the axial loading of high-strength steel cables and mono-strands exist, little work has been undertaken on determining the fatigue resistance of cables under bending. As such, the fatigue resistance of most stayed structures subject to cable vibrations has never properly been evaluated. Due to the absence of published bending fatigue spectra, Technical University of Denmark, in collaboration with DYWIDAG Systems International and Atkins, conducted a preliminary experimental campaign to determine the fatigue resistance of high-strength steel mono-strands under bending. As an extension of the current work, further PhD-research is proposed with Atkins as an industrial partner.

Large amplitude cable vibrations

The search for improvements in efficiency and aesthetics of structures has led engineers, architects, and planners to continually push the limits in structural design with ever longer, taller and lighter structures. For many of these structures, high strength steel cable is the preferred tensile load bearing structural element, as it is by far the cheapest per unit tensile force. It follows that cable stays support many telecommunication masts, stadia, bridges and offshore platforms. In many cases, these stays are exposed to wind, waves or water currents that can generate large amplitude vibrations, both in the stays and in the supported structures.

For several decades, there were no significant reports of cable vibrations on stayed structures. This was most likely due to the still relatively short cable lengths and the small number of cable-stayed structures in existence. In the mid-80s, though, reports of worrying vibrations started to emerge. Super-tall telecommunication masts were unable to send and receive signals due to stay vibrations, long cable-stayed bridges were vibrating and roof structures of sport arenas were being reassessed in wind tunnels.

The most worrisome of these were the large amplitude vibrations observed on cable-stayed bridges, as consequence of which immediate structural damage was observed.

Figure 1 shows the 6m amplitude cable vibrations that led to anchorage damage of the Second Severn Bridge (recorded in 1993). Although, vibration suppression mechanisms were devised, numerous troubling cable vibrations on long-span bridges were still reported[1, 2].

Figure 1- Large amplitude stay cable vibrations

Structural application of spiral strands

Presently, spiral strands and locked coil strands represent two main types of structural cables used in cable-net and cable-stayed structures. Since spiral strands are the most versatile structural cables exhibiting a high breaking strength and a good strength to weight ratio, this paper is focussed on those cables.

Spiral strands are employed for a variety of structural purposes, including prestressing of concrete, stays for guyed masts, cable-net structures, bridging applications such as stays for cable-stayed bridges and external post-tensioning.

Figure 2 - Common configurations of spiral strands

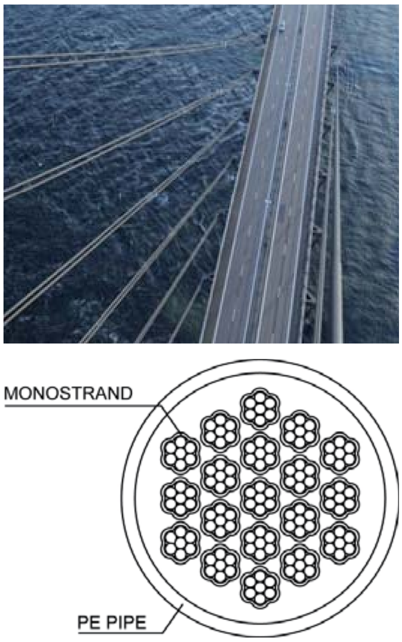


Figure 3 - Stay cables of the Øresund Bridge and an example of parallel strand stay cable

Spiral strands consist of large diameter galvanised round wires helically spun together (Figure 2). The wires are stranded in one or more layers, mainly in opposite directions, to form a closed wire system.

Contemporary strand stay cables generally consist of a predetermined number of parallel arranged strands enclosed in an UV resistant HDPE stay pipe of circular cross-section. Most commonly used in structures are seven-wire (1x7) and nineteen-wire configurations (1x19).

The individual strands have a diameter of 15.7mm and are of low relaxation grade, with nominal cross-sectional area of 150mm^2 and tensile strength (f_{GUTS}) of 1860MPa. Additionally, strands are waxed and individually sheathed with a continuous and wear resistant coating providing each strand with a triple protection arrangement.

Safety and durability of bridge cables

Cables with very small structural damping and flexibility are particularly susceptible to different types of vibrations. Variation in cable force for some structures where a cable is relatively short may be low enough to ignore during design.

However, for structures like cable-stayed or suspension bridges where a cable is a main structural element, load variations are very relevant. The development in the techniques for constructing large structures with exterior prestressing, like a cable-stayed bridge, demanded that particular attention be directed toward the safety and durability of the cables.

The most important factor in ensuring the durability and performance of a cable-stayed structure is the cable itself. Since cable-stayed and suspension bridges depend upon high-strength steel cables (Figure 3) as the major structural element and problems with cable vibrations on these bridges were widely reported[3-9], this paper concentrates on this type of structure.

Vibrations and fatigue

Several cable excitation mechanisms have been identified and significant research towards comprehending these has been undertaken. Excitation mechanisms are currently broadly grouped into two forms: those attributable to the wind and those which are the consequence of some other form of end loading, i.e. structurally induced. Of the wind induced mechanisms, those involving rain and/or ice seem to generate the largest cable vibration amplitudes[10].

Although wind-rain induced stay cable vibrations are normally not dangerous for the stay stability, they can considerably reduce its durability. Left unprotected, the vibrations in the highly tensioned cables can cause the fatigue of the tensile elements and breakages of the secondary elements; this may reduce the public confidence in the bridge. Therefore, cable vibrations and fatigue of stay cables are two phenomena inextricably bound together, that should be jointly investigated. Especially in cable stayed bridges, high fatigue resistant stay cables are of great importance[11, 12].

The fatigue phenomenon is well established in different engineering fields like welded joints, while in the case of cables made of steel wires many important aspects remain to be clarified. Moreover, after review of state-of-the-art in cable fatigue testing and review of experimental investigation on bending fatigue the following problems were recognised.

State-of-the-art in cable fatigue testing - evaluation deficiencies

The existing fatigue test procedures for stay cables are outlined in fib[13], PTI[14], SETRA[15] recommendations and Eurocode 3[16].

The most applied qualification tests are those outlined by PTI and fib, neither of which requires testing for bending. A set up with the principle of shimming (Figure 4) to provide flexural effects according to PTI and fib recommendations is considered to be sufficiently representative of the real conditions in a cable-stayed structure. As a consequence of this, high-strength steel cable bending fatigue spectra have not yet been developed. Thus, the calculation of the fatigue lifetime of bridge stay cables is currently only possible for axial variations in stresses.

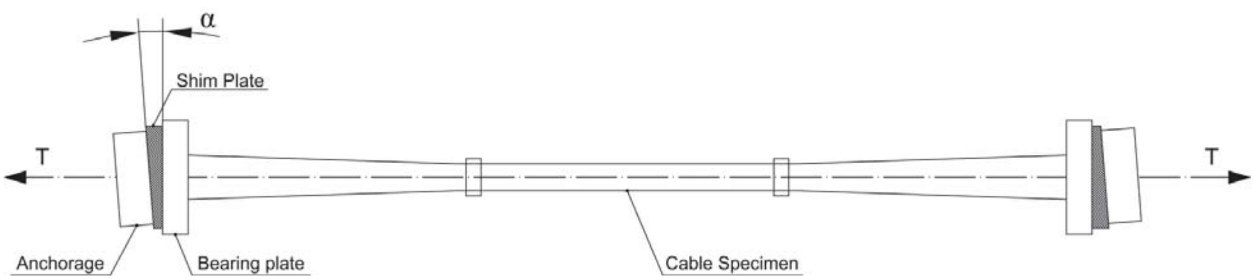


Figure 4 - Principle of shimming to provide flexural effects

Lack of experimental investigation on bending fatigue performance of cable mono-strands

To date there have been no reports of stay cable fatigue failure, however, large cable vibrations caused structural damage of bridge deck anchorages and gave rise to experimental investigations of cable stay fatigue behaviour under bending[6,9,17,18].

Moreover, in the case of combined road and rail bridges, bending fatigue is of general concern since bending at the anchorage can occur due to large variations of the live load, combined highway and rail traffic[18].

So far, experimental investigation on bending fatigue performance of cables was has been made on parallel wire stay cables[18], grouted parallel mono-strand stay cables [6,9] and 19-wire strands[17] subjected to bending load. However, to date no study has been made specifically on the fatigue performance of a stay-cable mono-strands under bending load. Furthermore, in most cases research teams were not able to measure bending stresses at the anchorage corresponding to the applied angular deviations.

It should be noted that the tendency in the choice of structural cable system is currently towards parallel mono-strands (no risk of twisting) rather than multi-layer strands. Therefore, experimental investigation of bending fatigue response of non-grouted mono-strands is of particular relevance.

Inaccurate assessment of service life of the cable

An important consideration in the design of cable structures is the fatigue resistance of cables under cyclic loading. Monitoring outcomes from bridges show small, medium and large vibrations of the stay cables. However, to date there is no accurate procedure to estimate the lifetime of cables undergoing cyclic bending.

Currently, the fatigue assessment undertaken is based on the following three approaches:

- Evaluation of fatigue life from available S-N curves for axial fatigue tests[20]
- Evaluation of fatigue life based on a wire fretting model developed by Raoof[19]
- Evaluation of fatigue life based on the existing Eurocode 3, Part 1-11[16].

It was reported[5,6] that these approaches and fatigue models do not give sufficiently accurate results and that the methods seeking to evaluate fatigue strength should be developed further to be consistent with observed fatigue failures for cables subjected to bending.

Furthermore, there were a number of bridge projects where design engineers encountered major problems with the estimation of service life of the cable[7-9].

Unfortunately, although broad knowledge of the excitation mechanisms exists, best practice engineering for the design of bridge stay cable is still based on the assumption that cables are not subjected to bending stresses. Furthermore, very little research has been conducted on the fatigue resistance of stay cables under bending.

Research project

Due to the absence of published bending fatigue spectra, the Technical University of Denmark, in collaboration with DYWIDAG Systems International and Atkins, recently completed a preliminary experimental campaign to determine the fatigue resistance of high-strength steel mono-strands under bending. A series of bending fatigue tests were conducted, with the goal of developing a basic bending fatigue spectrum[21].

To address the aforementioned deficiencies and realistically evaluate the fatigue lifetime of a cable, the assembly should be tested under cyclic flexural load reversals at a fixed axial load.

Research methodology

Most cable-supported bridges are typically supported by parallel mono-strands, comprised of seven wires each (Figure 5). Cable manufacturers produce cable assemblies that possess different forms of solutions for diminishing bending stresses in the anchorage. Therefore, it would be difficult to generalise axial and bending fatigue tests, based on the specific assembly of one manufacturer. A more rational approach to the problem would be to test the fatigue resistance of individual mono-strands, as these are often supplied by the same wire manufacturers to the cable manufacturers.

Test set up - design and construction

Since recent recommendations[15] regarding adjustments of fatigue test set ups are intended to develop the capacity to conduct complex fatigue testing with bending

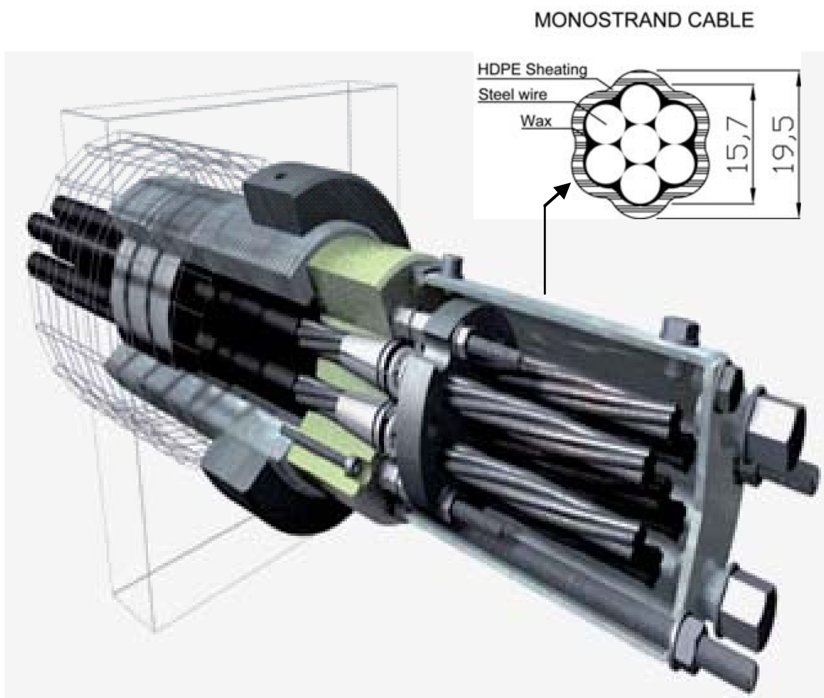


Figure 5 - Parallel mono-strand stay cable anchorage[35]

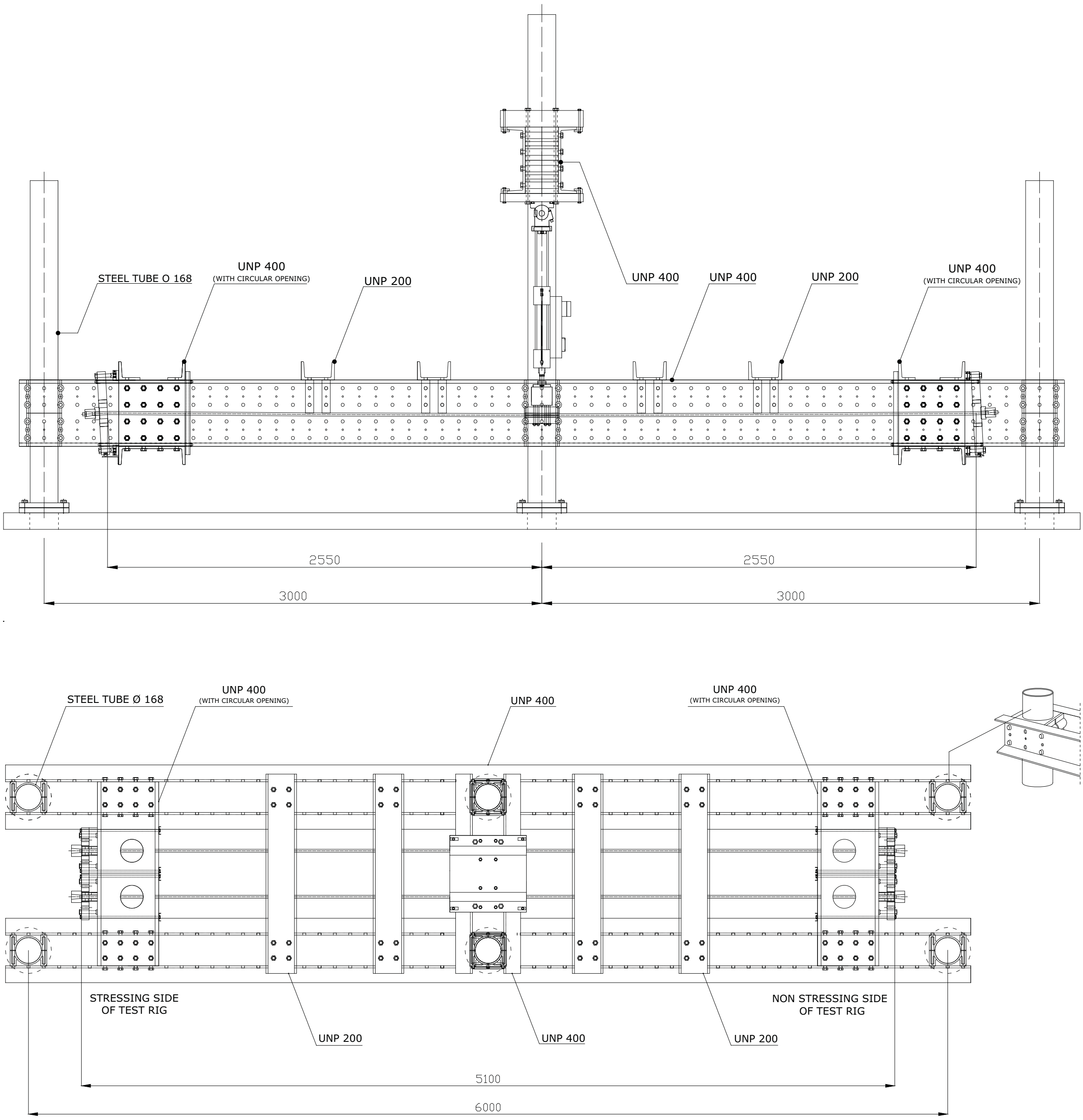


Figure 6 - Geometry of reaction frame

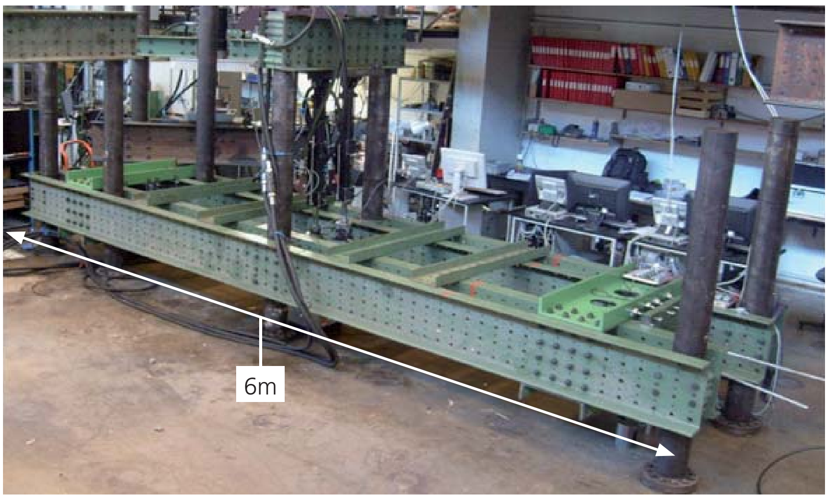


Figure 7 - Test rig for bending fatigue tests at the Technical University of Denmark (Byg DTU)

forces transmitted by an additional dynamic hydraulic actuator perpendicular to the cable, the test rig at the Technical University of Denmark (DTU Byg Laboratory) was designed accordingly (Figure 6).

The set up with principle of shimming allowed by fib and PTI, Figure 4, specifications was not considered to be representative of the real conditions in a cable-stayed structure where the cable is subjected to bending, due to wind loading, parametric excitation or other forms of dynamic loading.

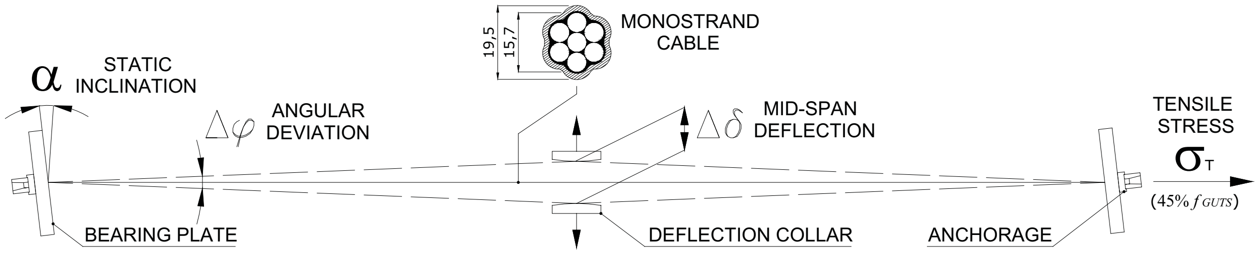


Figure 8 - Simplified model of the cable configuration

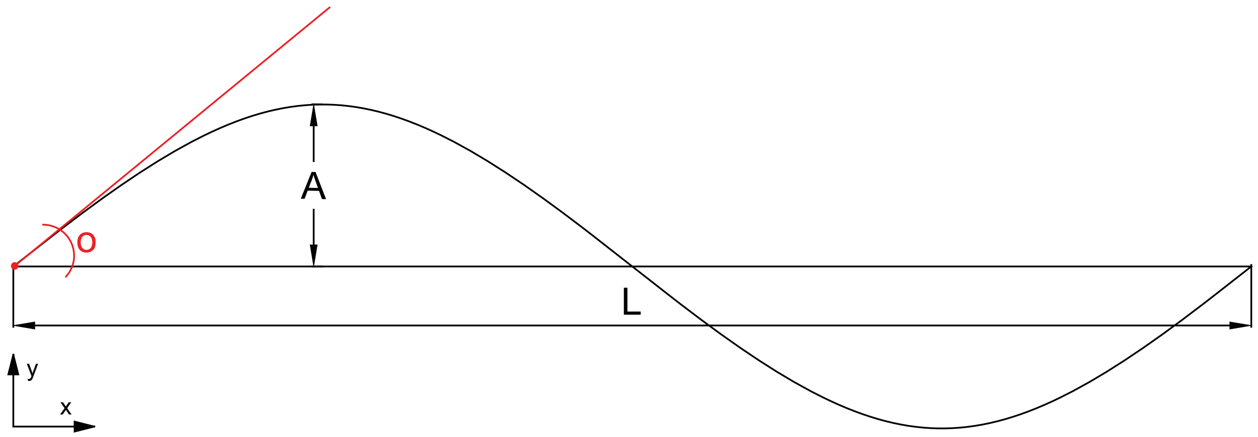


Figure 9 - Calculation of angular deviations

The goal was to build a test rig for testing post-tensioned high-strength steel mono-strands, Figure 7, that will simulate flexural effects, in which 5.1m long mono-strand cables can be tested under axial and bending load at a fixed axial load level (45% of ultimate tensile strength).

The test rig allowed for both static inclination at the anchorage α (simulating installation tolerances) and dynamic variation in deflection angle of the strand (simulating vibrations). The angular deviation Δφ at the anchorage was obtained by sinusoidally varying the mid-span deflection Δδ of the cable (Figure 8). With this test set up, maximum bending stresses were introduced near the anchorages and the performance of the cables under bending fatigue was evaluated.

Determination of angular deviation ranges for bending fatigue testing

Based on the information collected from the available literature [4,9,22,23-31], a database comprising different vibration events noted was created. Gathered records were used to determine realistic ranges of angular deviations relevant for bending fatigue tests. The database is complemented with early stage monitoring outcome of the Øresund Bridge conducted by DTU researchers[19].

Cable vibrations result in the rotation of the cable at the anchorage. The angular deviation may be determined by means of the amplitude's magnitude, mode of vibration and cable length. By representing the vibrations in terms of cable rotation, data from different bridges and events are comparable.

The motion of vibrating cables can be described by the function:

$$f(x) = A \cdot \sin\left(\frac{i\pi x}{L}\right)$$

where: i - mode of vibration, L - length of the cable, A - amplitude

The angular deviation is calculated as a tangent line to a curve describing a vibrating cable at its termination, i.e. at the cable's anchorage (Figure 9).

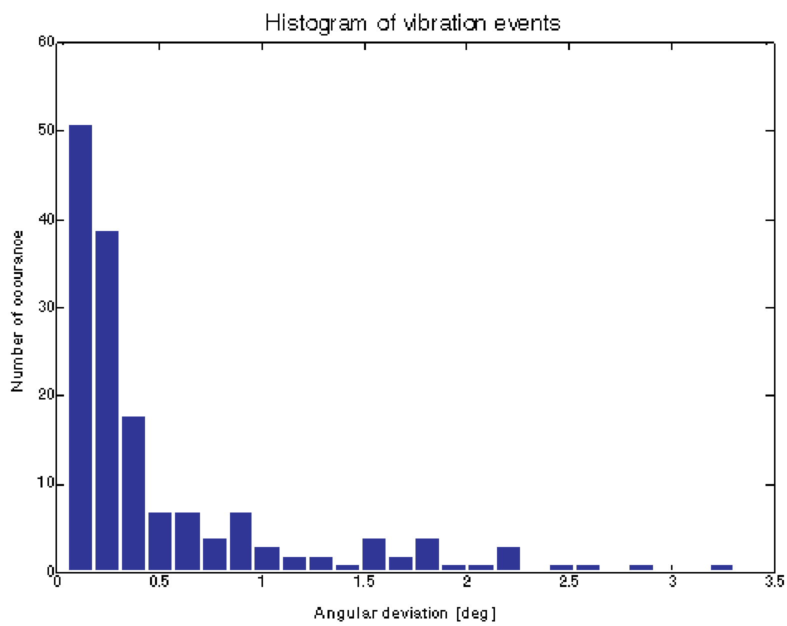


Figure 10 - Histogram of cable angular deviations

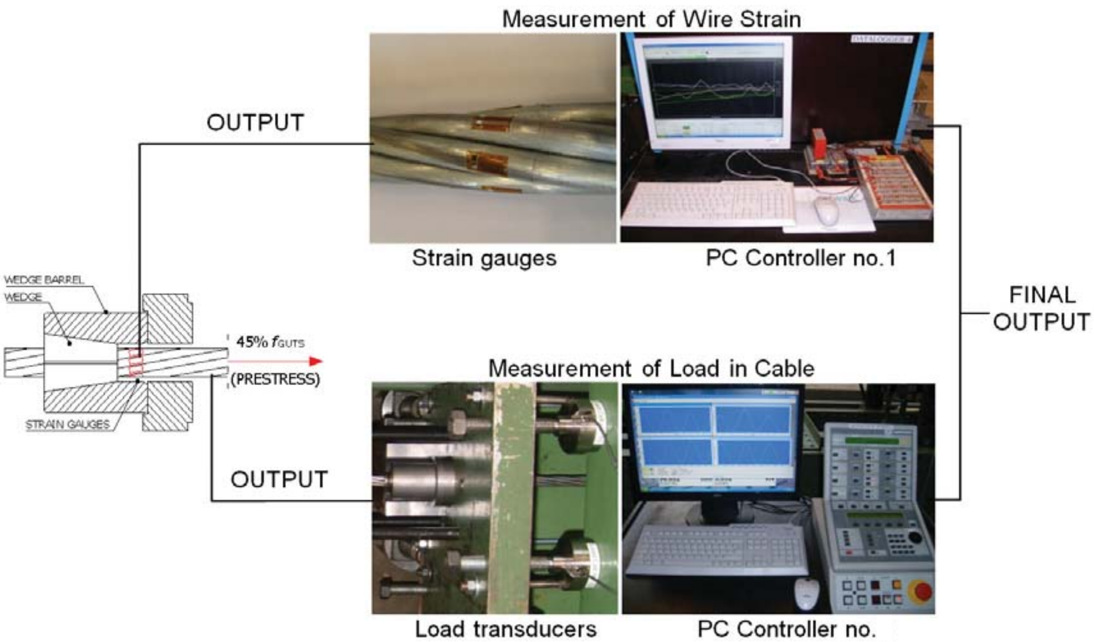


Figure 11 - Simplified schematic diagram of data processing

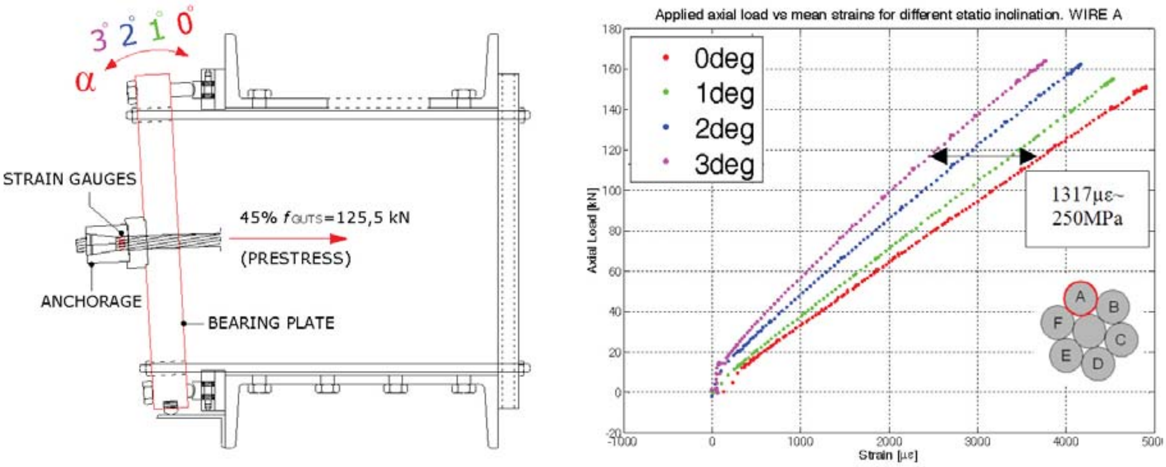


Figure 12 - The axial strains due to stressing operation and increase in strain depending on static inclination for wire A

$$f'(x) = -A \cdot \cos\left(\frac{i\pi x}{L}\right) \cdot \frac{i\pi}{L}$$
$$\Theta = f'(x=0) = -A \cdot \frac{i\pi}{L}$$

Figure 10 shows a histogram created from gathered records covered in the database. The histogram of angular deviations illustrates the possible range of vibrations.

The following four ranges of angular deviations were selected for bending fatigue testing purposes: 0.5°; 1.0°; 1.5°; 2.0°.

Static test - Axial and bending stresses

The analysis of cables under tension and bending loads is a complex problem. Hence a static test was conducted with the primary objective to determine the strain and stresses experienced by a strand when the mono-strand cable is subjected to axial and bending load. The only direct method to measure the strand response was to attach strain gauges to the surface of the strand. Since the bending stresses in the cable occur locally, it was attempted to place the gauges as close to the fixation point (wedge) as possible, without harming the gauges (Figure 11).

The actual stressing force in the strand was controlled and measured by load transducers placed behind bearing plate, while wire strains were measured by strain gauges recorded by data logger (Figure 11). Merging both outputs allowed the plotting of applied axial and bending load versus individual wire strains.

Axial stresses - Strains due to prestress and influence of static inclination on initial stresses

Measurements were recorded for stressing operations at four different inclination of the bearing plate which allow analysing the influence of static inclination α on axial stresses. It is important to see the behaviour of the strand in terms of single wires while stressing the cable. After stressing operation, each single wire remained at different axial strain level, thus

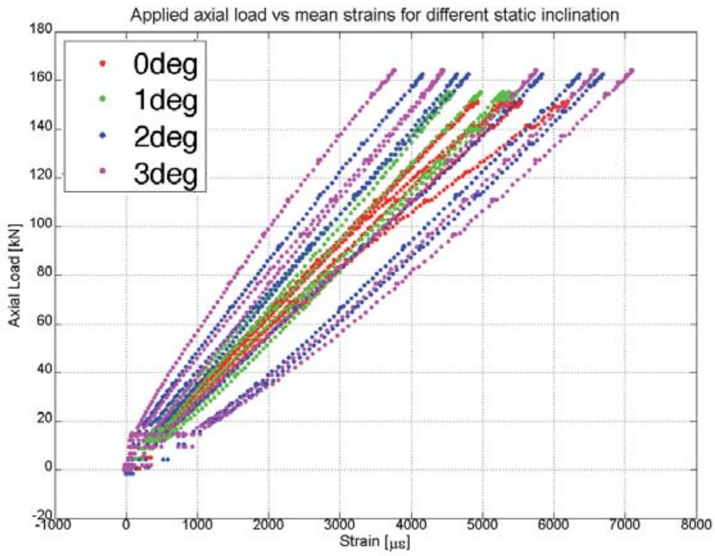


Figure 13 - Dispersion of strains depending on static inclination

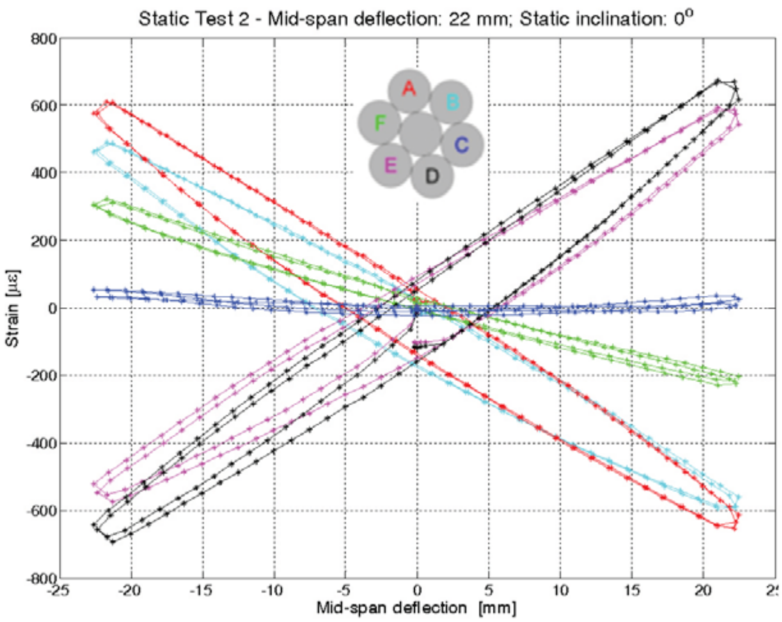


Figure 14 - Increase in wire strains with mid span deflection

wires were exposed to different total strains. The graph in Figure 12 shows the stress-strain relation of the outer wire A having the highest strains.

It can be noted that there is a significant difference in stresses on the outer wires depending on the degree of the static inclination. Variation in stresses is caused by local bending due to the inclination of the bearing plate. Figure 12 shows the readout of strains taken under axial load of 125kN (45% f_{GUTS}). It can be seen that static inclination of the bearing plate of 3° caused increase in the total stresses of 250MPa for wire A. It was thus decided to use static inclination of 3° for the dynamics tests, as it simulates the installation tolerances most explicitly.

Figure 13 shows the relationship of the applied axial load and single wire strains during four different stressing operations (when bearing plate was not inclined, inclined to 1°, 2°, 3°).

At increasing static inclination of the bearing plate, the individual wire strains were more dispersed. Inclination of the bearing plate introduced additional local bending stresses in the axially stressed cable, resulting in large differences in single wire's stress level. Since strain gauges are placed close to anchorage zone, they capture additional stresses caused by inclination.

Bending stresses - strains due to applied mid-span deflection

Figure 14 shows the variation of strain with mid-span deflection for a strand with a pre-stress force of 125kN.

Strains shown on Figure 14 represent the change in strain due to bending. Initial strains from pre-stressing are not included. The strain gauges attached to the extreme wires experienced the highest absolute strains. Gauge on wires A and D were furthest from the centre of the cross section in the bending plane.

Influence of local bending stress on overall stress in strand

Figure 15 illustrates the contribution of bending stresses due to applied mid-span deflection on overall stress in strand. Growing ranges of bending stresses correspond to increase in angular deviation. It should be noted that when the cable strand is deflected at mid-span to 89mm (angular deviation of 2°), wire D is reaching yielding point. It can be seen that combined axial and bending stresses for angular deviations of 0.5°; 1.0°; 1.5° did not exceed linear region. Therefore strains for further purposes can be multiplied by modulus of elasticity to obtain relevant stress ranges. Strains due to the highest angular deviation of 2.0° are close to yielding point, hence corresponding stresses were read out from the stress-strain curve from the tensile test provided by the manufacturer.

Dynamic test

After completion of the static test, the cable was further tested in reversed cyclic flexural loading at a fixed axial load level (45% of f_{GUTS}). The central deviation was introduced by the actuator placed at mid-span and the load was applied sinusoidally. Depending on the test program, the actuator was imposing four different deformations at mid-span corresponding to the previously determined angular deviations of 0.5°; 1°; 1.5°; 2°. In addition, all dynamic tests were performed at the static inclination of the bearing plate of 3°. The test stopped automatically after initial rupture of one of the seven wires.

Variation of load in strand and variation in wire strains

Figure 16 shows the variation of axial force in the cable tested at angular deviation of 2°. The graph shows cycles at three stages during the whole test.

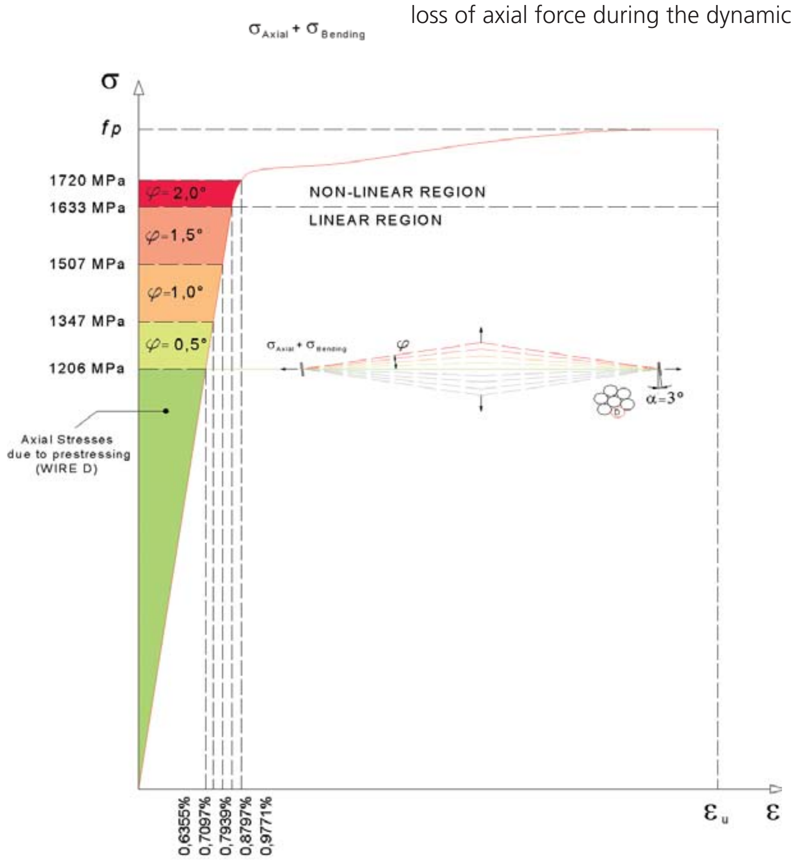


Figure 15 - Contribution of bending stresses on overall stress in strand

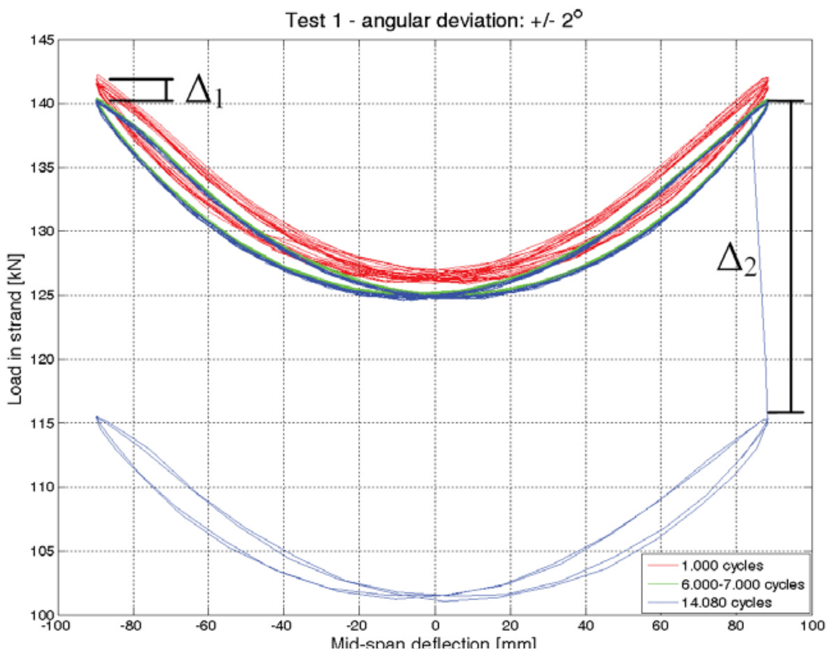


Figure 16 - Variation in axial force of cable

Throughout the dynamic tests conducted, loss of prestress force in range of Δ_1=2kN was observed. The displacement gauges were installed on the wedge assembly to check whether the loss of axial force might be related to further seating of the wedge. The results from measurements have shown that the loss of axial force during the dynamic test does not correspond to slip of the wedge. Therefore the reduction of the prestress force over time might be due to the relaxation of the steel. The drop Δ_2=24kN was caused by single wire rupture and corresponds to 19% of the total tensile force in strand.

The overrated drop of force (14% was expected) might be related to cracks that were observed in the vicinity of the ruptured wire.

Strain gauges were measuring strain throughout the test. The graph in Figure 17 shows fluctuation of the wire strains due to bending. Wire A and D experienced the highest strains, similarly to static test results.

Note that Figure 17 shows only strains variations due to flexural loading, axial stresses are excluded.

Moment of wire failure

Figure 18 shows the moment of the first wire breakage. During breakage moment, sudden change in strain was observed. The strains of wires A, B decreased and D increased to the same amount of strains, while wires F and C almost kept the same level. Wire E experienced a significant drop in strains and amplitude of the test after rupture decreased by ~ 1000 με, while the rest of the wires kept the same amplitudes. Measurements were taken on the non-stressing side and wire failure occurred at the stressing side, hence it was difficult to verify which of the measured wire strains correspond to the fractured wire. However, the drop of force in wire E caused further seating of adjacent piece of the wedge assembly. It may confirm an assumption that it was wire E which broke.

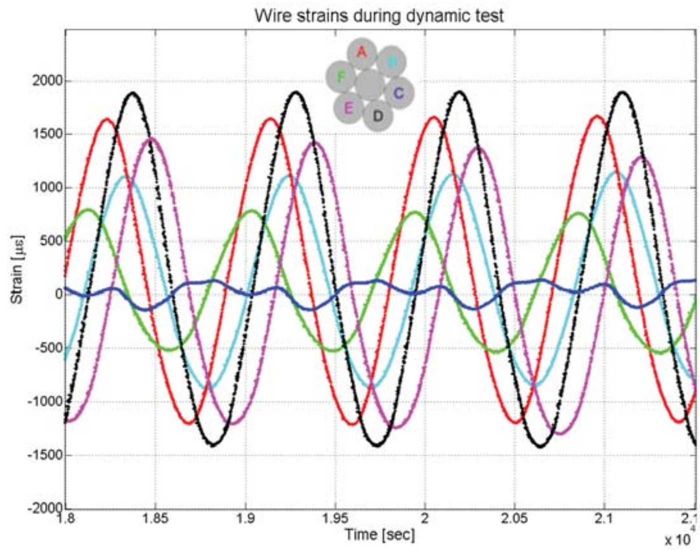


Figure 17 - Variation in wire strains during dynamic test

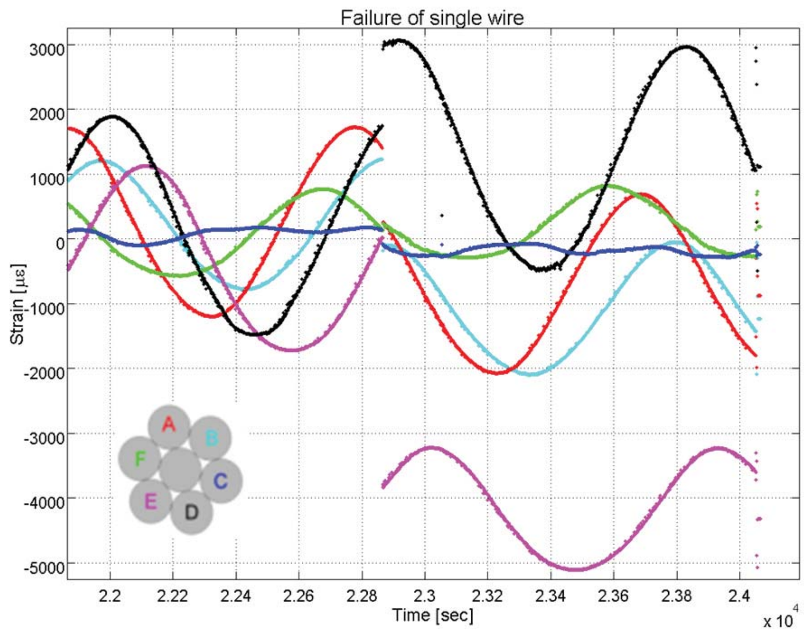


Figure 18 - Moment of wire failure

Inspection of the fractured sections of specimens

Wire breaks from all fatigue tests were concentrated within the anchorage. No bending fatigue damage was reported along the free length of the strand and in the vicinity of the clamping region, hence all results are considered to be representative of conditions in the field.

Rupture of wires occurred in regions where the strand experienced a sudden change in strain. Therefore, of the 8 cables tested, the wire breaks occurred at the location of the last tooth of the wedge (Figure 19).

Failure surface

The shape of the failure surface of the fractured wire in the strand is similar in all inspected specimens. Evidence of fatigue crack growth suggests that rupture process was due to applied flexural load reversals (Figure 20). Fatigue of wires was greatly influenced by the local bending stresses and partially by the loading history and the mean stress. The reduction of the amplitude of the mid-span displacements was found to be the factor that has the largest influence on increasing the fatigue life of tested strand specimens.

As expected in a bending fatigue test, the majority of the breaks occurred at the outer fibres of the strand cross-section where the bending stress ranges are the highest.

Figure 19 - Location of fatigue fracture

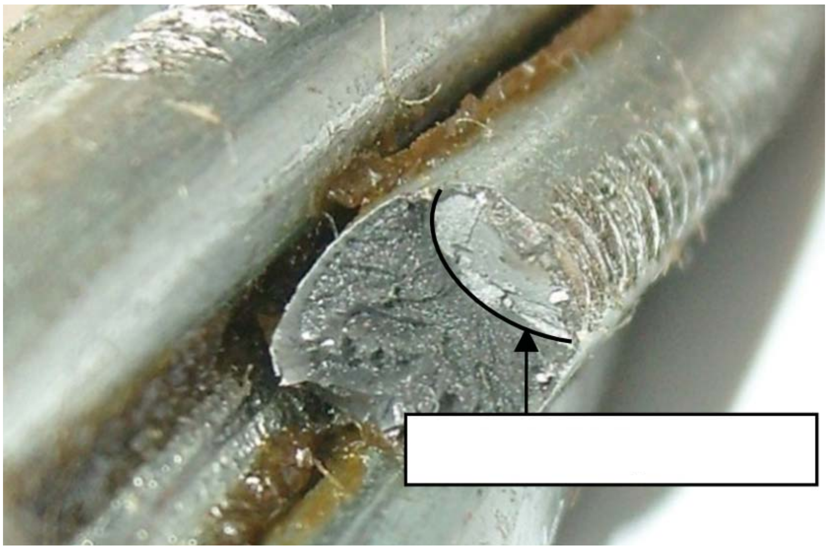


Figure 20 - Failure surface of the fractured specimen

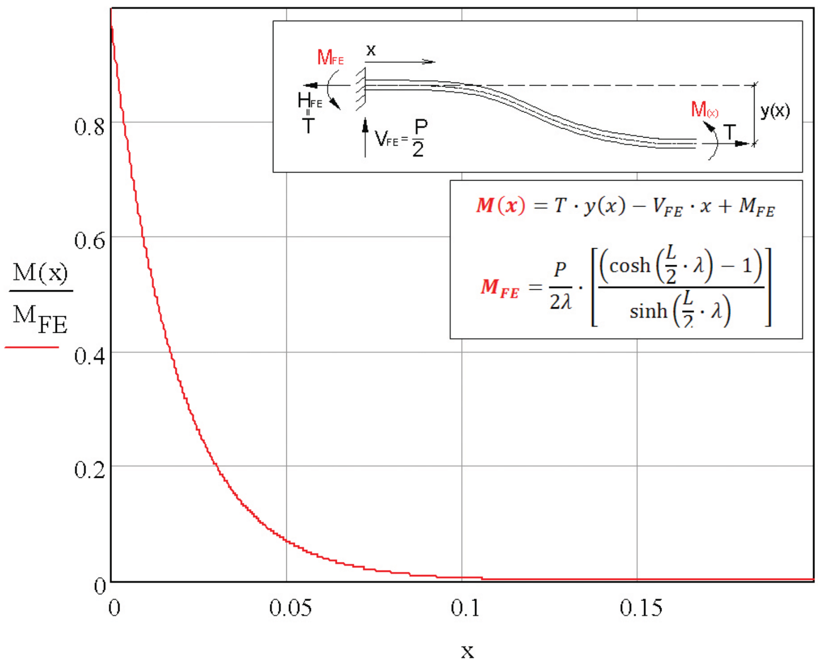


Figure 21 - Distribution of normalised moment

Local bending stresses in cables

Observed fatigue failure suggests that fatigue cracks were initiated by wedges. This was an indication that these are areas with localised high curvatures and therefore high bending stresses which have led to the fatigue fractures.

The graph in Figure 21 reflects the local nature of bending stresses of a cable. The moment distribution along the length of the cable M(x) was normalised by the moment at fixed end M_{FE} and plotted in Figure 21.

The bending moment and hence the bending stresses decrease exponentially with a distance.

Preliminary bending fatigue spectrum - design curve for cable strand

One of the main objectives of the research project was to obtain a preliminary bending fatigue spectrum for mono-strand cable. Table 1 summarises testing parameters and the total number of wire breaks for each specimen.

The S-N curve describes the relation between a given stress amplitude and the number of cycles until a specimen fails. To obtain the curve each specimen was tensioned to 45% of ultimate strength, exposed to a bending stress range ($\Delta\sigma_{2\varphi}$), and the number of cycles until the specimen break (N) was observed.

Table 1 -Testing parameters and number of cycles to 1st wire breakage

Testing Parameters						**1st Wire Break**		
Specimen	Angular deviation φ	Mid-span deflection	Bending stress range $\Delta\sigma_{2\varphi}$	Static inclination α	Frequency	Cycles	Active End	Dead End
#1	2,0°	± 89 mm	1013 MPa	3,0°	1,5 Hz	14081		x
#2	2,0°	± 89 mm	1013 MPa	3,0°	1,5 Hz	10733	x	
#3	2,0°	± 89 mm	1013 MPa	3,0°	1,5 Hz	14948		x
#4	1,5°	± 67 mm	761 MPa	3,0°	2,0 Hz	25177	x	
#5	1,5°	± 67 mm	761 MPa	3,0°	2,0 Hz	14350	x	x
#6	1,5°	± 67 mm	761 MPa	3,0°	2,0 Hz	19872		x
#7	1,0°	± 45 mm	530 MPa	3,0°	1,5 Hz	46855	x	
#8	0,5°	± 22 mm	280 MPa	3,0°	1,0 Hz	120047		x

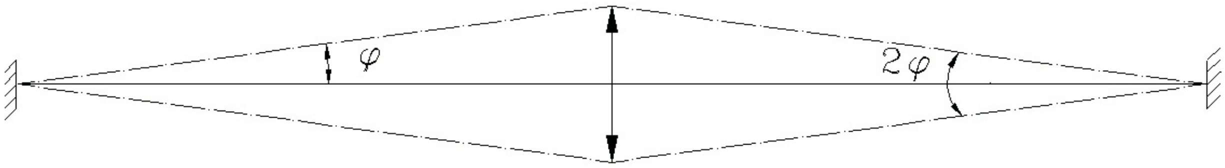


Figure 22 - Bending stress range (2φ) and applied angular deviation (φ)

Table 2 - Bending stress ranges for S-N curve purposes

Mid-span deflection (Angular deviations)		σ_φ [MPa]	$\Delta\sigma_{2\varphi}$ [MPa]
22mm (0.5°)	max	142	280
	min	-138	
45mm (1°)	max	301	530
	min	-229	
67mm (1.5°)	max	464	761
	min	-297	
89mm (2°)	max	650	1013
	min	-363	

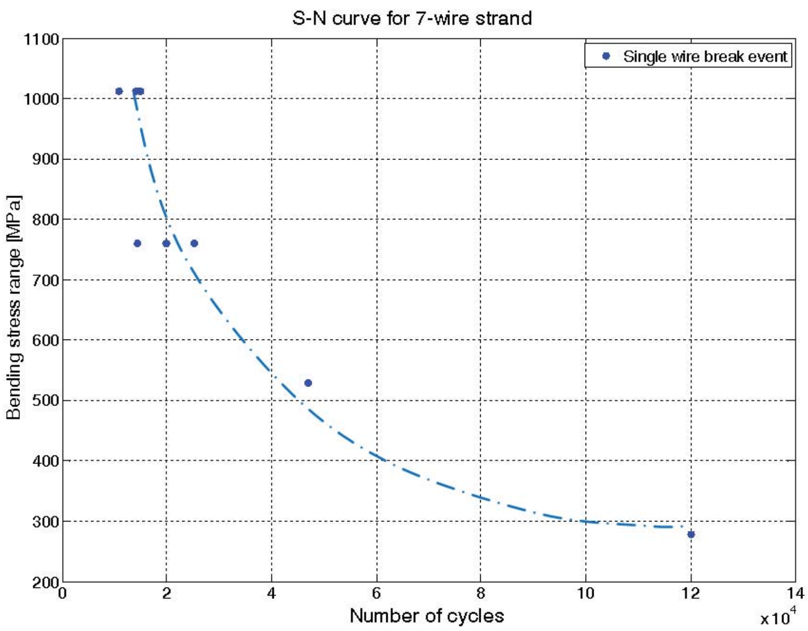


Figure 23 - Logarithmic function formed by fatigue data

The bending stress range ($\Delta\sigma_{2\varphi}$) is the difference between the maximum and minimum stress in the cycle (Figure 22) while the number of cycles to failure (N) is known as the endurance life or fatigue life.

The bending stress range ($\Delta\sigma_{2\varphi}$) was determined from results obtained after static tests and are shown in Table 2.

Determination of preliminary bending fatigue model for mono-strand cable

It can be seen in Figure 23 that data obtained from fatigue tests form a logarithmic function. This function represents a curve with the number of cycles to failure increasing with a decrease in cyclic stress amplitude, approaching the endurance limit.

Note that the stress range ($\Delta\sigma_{2\varphi}$), presented in the graph refers to stresses only due to bending (peak to peak amplitude). It should be noted that all test specimens were first tensioned to approximately 45% of ultimate strength and had a constant mean level stress of around 830MPa.

The expression linking N and $\Delta\sigma_r$ can be plotted on a logarithmic scale as a straight line and is referred to as an S-N curve using the following equation:

$$log\ N = A - m\ \log\ \Delta\sigma_r$$

Results of the eight bending fatigue tests performed (Figure 24) were used to develop the following preliminary fatigue life model of a strand undergoing cyclic flexural loading:

$$log\ N = 9{,}74 - 1{,}88\ \log\ \Delta\sigma_r$$

Constants 'A' and 'm' are derived from the linear function fit to the logarithms of data by linear regression. The method used to fit a predictive fatigue model to an observed bending fatigue data was least-squares fit.

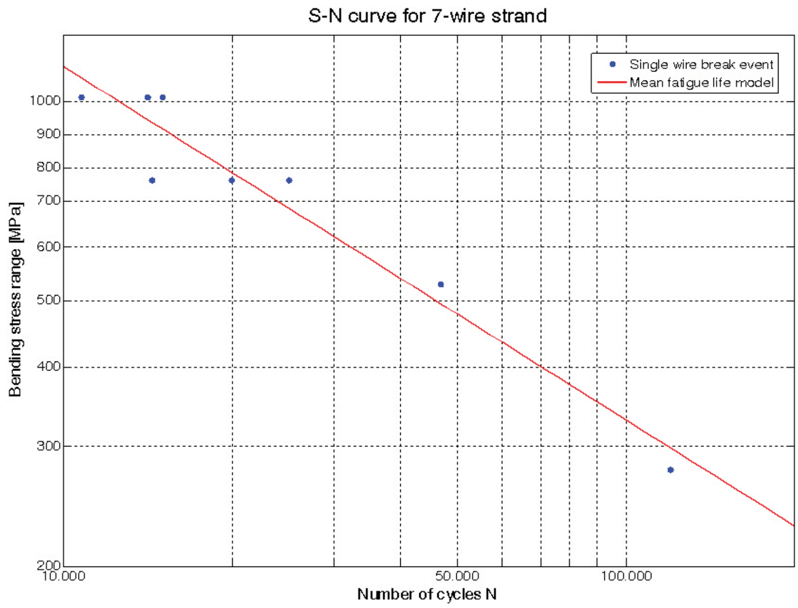


Figure 24 - Preliminary bending fatigue spectrum for mono-strand

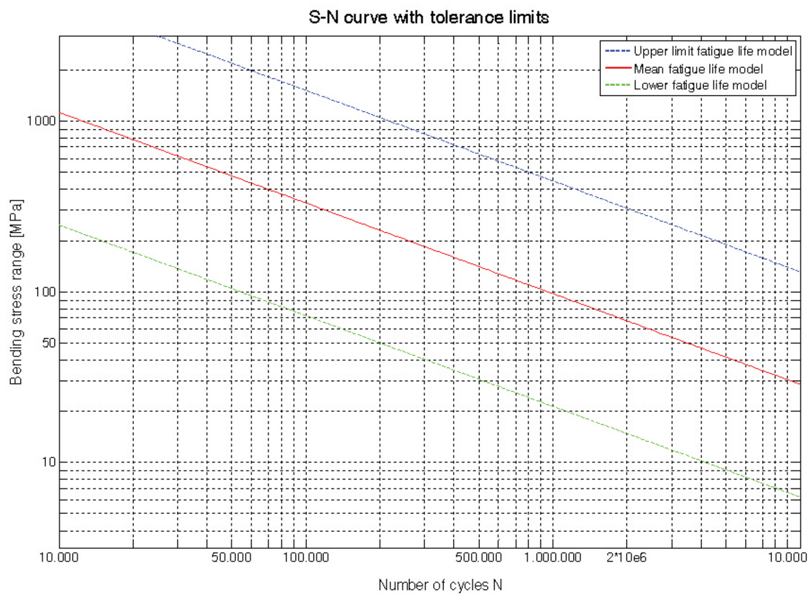


Figure 25 - Mean fatigue model with tolerance limits

Lower and upper limit models

Since more tests are needed to eliminate uncertainties resulting from lack of data at the low stress range, a mean line of model with tolerance limits (Figure 25) is proposed.

The lower tolerance model will provide a design engineer with a tool to minimise the chance of fatigue failure while the upper limit model will allow for less conservative design. The tolerance limits are a function of both the anticipated standard deviation and the number of data points obtained from the tests.

The lower limit regression model (green line) should be used for the purpose of design as it best fits to the requirement of properly conservative design guides [20].

Comparison of results with axial fatigue data

Researchers from the University of Texas at Austin after summary of published axial fatigue tests on single seven-wire strand in air[20] developed the following lower bound relationship:

$$log\ N = 11 - 3{,}5 \log \Delta\sigma_r \quad [\text{ksi}]$$

Note that the above equation is valid in American Metric System, where unit of stress is KSI. Adequate equation in SI unit system would have a form:

$$log\ N = 14 - 3{,}5 \log \Delta\sigma_r \quad [\text{MPa}]$$

Figure 26 and Figure 27 shows a graph of the axial and the bending fatigue model, respectively.

It can be seen that axial fatigue spectrum differs from that obtained after bending fatigue tests. The results from bending fatigue tests indicate that the derived models and the proposed range are more conservative and should be used for design purposes rather than lower bound relationship resulting from axial fatigue tests.

Application of scientific outcome - fatigue analysis of the Øresund Bridge stay cables

The following considerations are examples of how the obtained bending fatigue spectrum could be utilised and should not be treated as a decisive analysis.

The monitoring system of the Øresund Bridge involves the four longest cables on the mid-span of the bridge. These four monitored cables (Figure 28) range from a 261m to 192m in length, all of them have the same diameter of 250mm. Vibration data were used to characterise the motion of the stay cable and estimate the number of wind-rain induced and other form of vibration cycles that cable experienced during the six month period[22].

Integration of acceleration histories and Fourier Transform

Figure 29 depicts a 20-second excerpt of the acceleration history of CABLE 1 during a vibration event.

The Fourier transformation was used to obtain a compact representation of a signal. A frequency representation of the signal acquired for one event is displayed in Figure 30.

As a result of Fast Fourier Transform (FFT), the signal in time-domain was decomposed into a sum of sinusoids with different frequencies and amplitudes.

The frequency-domain graph (Figure 30) shows how much of the signal lies within each given frequency band over a range of frequencies. Frequencies corresponding to the ten highest peaks in the FFT were recorded which correspond to the frequencies of the modes that dominate the displacement response.

It was decided to use the first ten modes to describe the motion of the cable since most of the energy present in the signal lies in these modes[32].

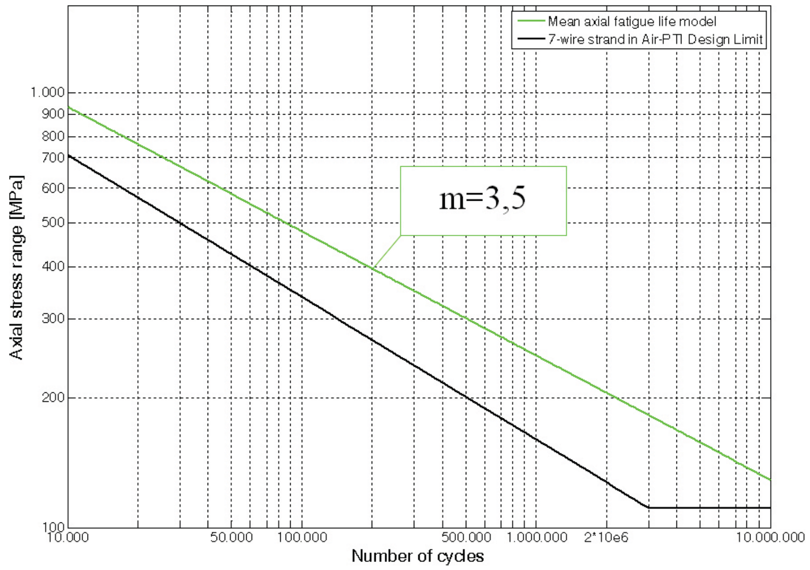


Figure 26 - Axial fatigue model

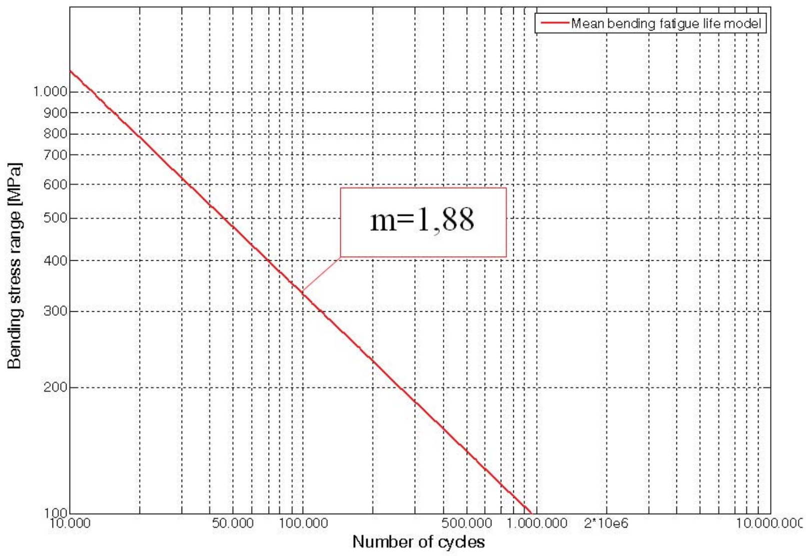


Figure 27 - Preliminary bending fatigue model

Characterisation of motion - vibrations in both planes

The acceleration data were numerically integrated twice to obtain the in-plane and out-of-plane (lateral) modal displacements.

The total displacement of cable including contribution from ten modes can be seen in Figure 31.

Subsequently, total angular deviation of cable in anchorage location (Figure 32) was calculated using equation:

$$\varnothing_{tot} = f'(x=0) = -A_{tot} \cdot \frac{i\pi}{L}$$

Where A_{tot} is the total displacement of cable in location of accelerometer (i) is mode number and L is cable length.

Rainflow analysis

The rainflow cycle counting algorithm is commonly employed for fatigue life assessment of structures under non-constant amplitude loading. The algorithm extract cycles from the load history obtained from monitoring[33, 34].

Relevant rainflow counting output for CABLE 2 is illustrated in Figure 33. During the static test strains are measured corresponding to angular deviations and therefore the results obtained from rainflow analysis can be related to stress near the anchorage.

Estimation of service life of stay cable

The estimated total number of cycles that the cable has experienced can be compared with the obtained preliminary bending fatigue spectrum (Figure 34).

Subsequently, the results from strand fatigue tests are used for the preliminary estimation of the remaining service life of the stay cables. At this point of the analysis, individual static test for the actual cable anchorages of the Øresund

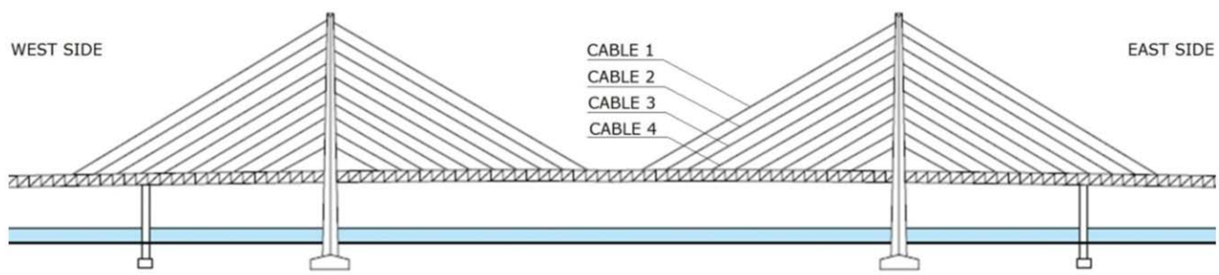


Figure 28 - Cables of the Øresund Bridge instrumented with accelerometers

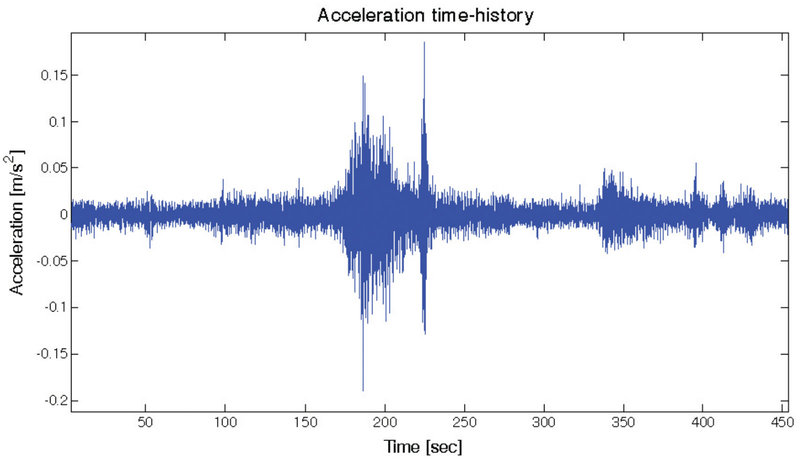


Figure 29 - Acceleration-time Record for CABLE 1[22]

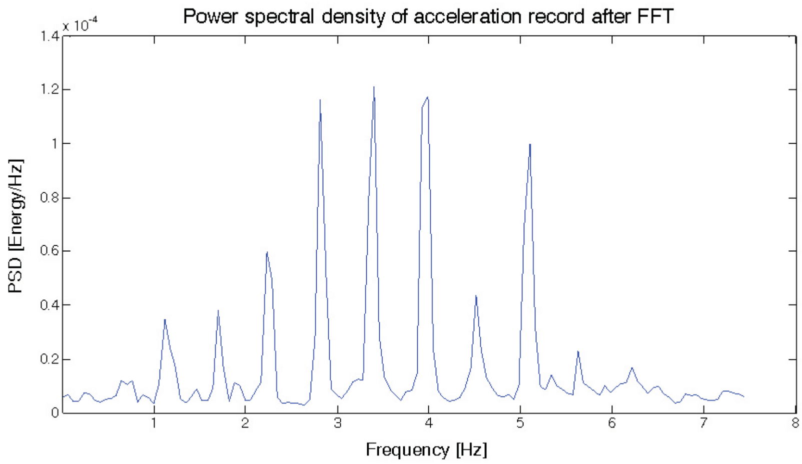


Figure 30 - Frequency representation of the signal[22]

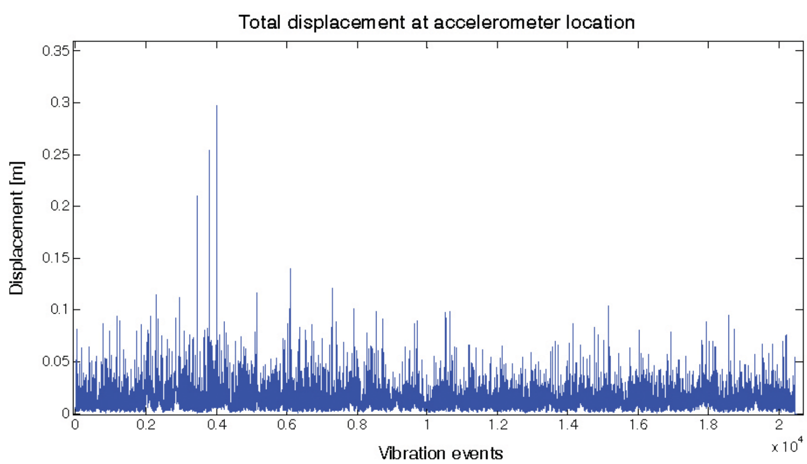


Figure 31 - Total modal displacements[22]

Bridge should be performed to find relevant stress ranges.

However, for simplification purposes the data from proposed S-N curve (cable attached to a fixed end without a guide) were taken to calculate stresses corresponding to angular deviations smaller than 0.5°.

Subsequently, the Palmgren-Miner formula may be used to estimate the expected fatigue life:

$$\frac{n \cdot f_1}{N_1} + \frac{n \cdot f_2}{N_2} + \cdots + \frac{n \cdot f_n}{N_n} \leq 1$$

Where N_n is the number of cycles to single wire break failure, f is the number of events in a particular stress range (angular deviation) and n is the expected endurance life of cable.

For the purpose of Fatigue Limit State (FLS) design, the lower limit regression model was used since it best fits the requirement of conservative design guides. The fatigue life of the CABLE 2 (damped) of the Øresund Bridge was calculated to be 42 years using the data from the proposed S-N curve. It should be noted that the calculated fatigue life is subjected to considerable uncertainty, as the half year measuring period is relatively short and may not be representative.

Summary

From the bending fatigue tests conducted, a preliminary S-N curve is proposed for the conservative estimation of mono-strand cable service life expectancy. The presented bending fatigue spectrum of seven-wire high-strength mono-strands is currently unavailable in the published literature. Moreover, the results provide relevant information about the bending mechanism and fatigue characteristics of mono-strand steel cables in tension and flexure and show that localised cable bending has a pronounced influence on the fatigue resistance of bridge cables under dynamic excitations.

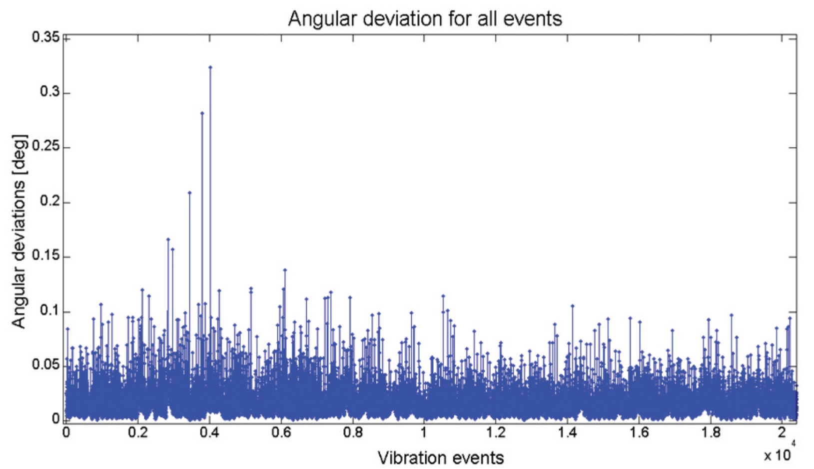


Figure 32 -Total angular deviation[22]

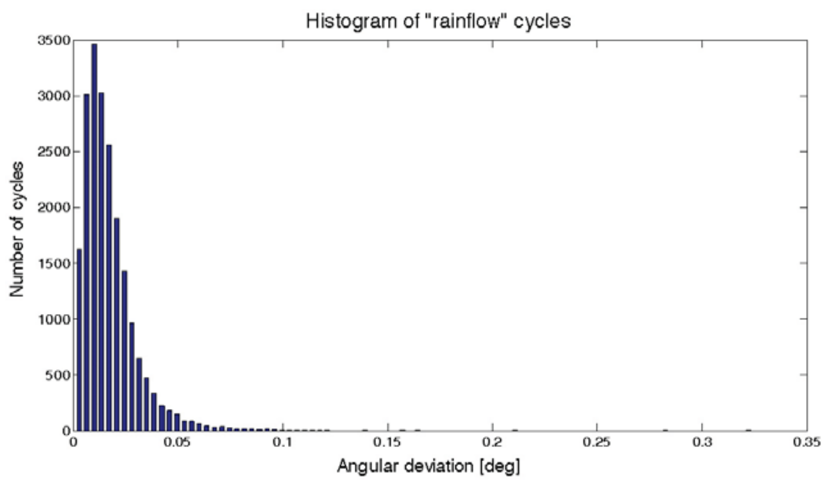


Figure 33 - Rainflow counting algorithm output

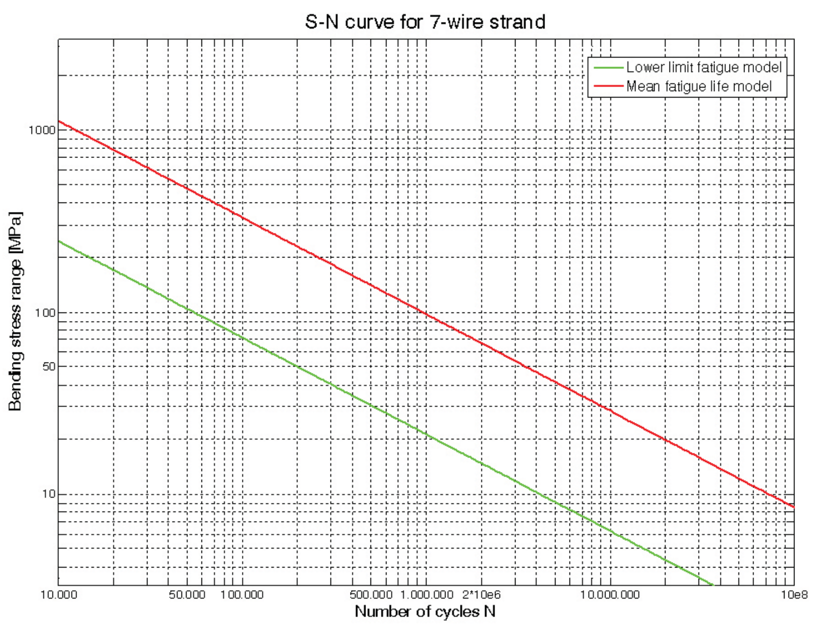


Figure 34 -Design curve for estimation of cable stay service life

Future work - Industrial PhD

The mono-strand forms the basis for parallel mono-strand stays. But even so, knowledge of the bending fatigue resistance of mono-strands does not directly lead knowledge of the bending fatigue resistance of a bridge stay. The mechanisms of stress generation in full stays are much more complex. Appropriate theoretical modelling and experimental validation are needed.

The objective of future PhD research is, first, the extension of mono-strand bending fatigue tests undertaken at DTU Byg, to enhance the reliability and improve the resolution of the existing spectrum. Secondly, a relationship between the fatigue resistance of a mono-strand and a full parallel mono-strand stay cable would be made, through mathematical modelling and full-scale static testing. The proposed PhD research should lead to a simplified model that can be used for the assessment of stay cable fatigue.

With this research, it is hoped that one of the most basic oversights in the lifetime assessment of cable-supported structures, namely the bending fatigue resistance of parallel mono-strand cables, will be addressed.

References

1. Kumarasena, S., Jones, N., Irwin, P., Taylor, P., "Wind-Induced vibrations of stay cables", US Federal Highway Administration Report, FHWA-HRT-05-083, August 2007
2. Elsa de Sá Caetano, Cable Vibrations in Cable-Stayed Bridges, SED 9, International Association of Bridge and Structural Engineering, 2007
3. Savor, Z., Radic, J., Hrelja, G., "Cable vibrations at Dubrovnik bridge", Bridge Structures, Vol. 2, No. 2, June 2006, 97–106
4. Takano, H., Ogasawara, M., Ito, N., Shimosato, T., Takeda, K., Murakami, T., "Vibrational damper for cables of the Tsurumi Tsubasa Bridge", Journal of Wind Engineering and Industrial Aerodynamics 69 71 (1997) 807-818
5. Zhou, H. J., Xu, Y. L., "Wind–rain-induced vibration and control of stay cables in a cable-stayed bridge", Structural Control and Health Monitoring, 2007; 14:1013–1033
6. Wood, S.L., Frank, K.H, "Experimental investigation of bending fatigue resp onse of grouted stay cables", Journal of Bridge Engineering. Submitted August 1, 2008; accepted May 13, 2009.
7. Jensen, J.L., Bitsch, N., Laursen, E., "Fatigue Risk Assessment of Hangers on Great Belt Bridge"
8. Bitsch, N., Laursen, E., Andersen, J.E., "Analysis and Mitigation of Large Amplitude Cable Vibrations at the Great Belt East Bridge "
9. Dowd, J., Poser, M., Wood, S.L., Frank, K.H, Williamson, E.B.,"Bending Fatigue Tests on Stays Cables", Journal of Bridge Engineering, November/December 2001, 639-644
10. Gjelstrup, H., Georgakis, C.T., Larsen,A., "A Preliminary Investigation of the Hanger Vibrations on the Great Belt East Bridge", 7th International Conference on Cable Dynamics, 10-12 December, 2007, Vienna
11. Gimsing, N., "Cable Supported Bridges, Concept & Design", 2nd edition, 1997
12. Mondorf, P., "Concrete Bridges", Polyteknisk Forlag, ISBN 10 87-502-0962-0, 2006
13. Fédération internationale du béton (fib). "Bulletin 30 Acceptance of stay cable systems using prestressing steel", 2005
14. Post Tensioning Institute (PTI). "PTI Guide Specification. Recommendations for stay cable Design, Testing and installation", 2007
15. SETRA Cable Stays Recommendations of French Interministerial Commission on Prestressing. "Haubans - Recommandations de la CIP.", 2002
16. Comité européen de Normalisation. "prEN 1993-1-11: Design of structures with tension components", 2006.
17. Gourmelon, J.P., "Stay cable fatigue. Organisation and main results of the research programme conducted by the LCPC", Laboratoire Central des Ponts et Chaussées, 2003
18. Miki, C., Endo, T., Okukawa, A., "Full-Size Fatigue Test of Bridge Cables", Length Effect on Fatigue of Wires and Strands, International Association for Bridge and Structural Engineering, Report, Vol. 66, pp. 167-178, 1992
19. Raoof, M., "Axial Fatigue of Multilayered Strands", Journal of Engineering Mechanics, Vol. 116, No. 10,October, 1990
20. Paulson, C., Frank, K. H., Breen, J.E., "A Fatigue Study of Prestressing Strand" , Ferguson Structural, Engineering Laboratory Research Report, 1983
21. Winkler, J., Kotas, A., "Fatigue Analysis of Steel Cables under Bending Load". Master's thesis, DTU Byg, November 2010, Kgs. Lyngby
22. Acampora, A., "Understanding of cable vibrations and the associated flow-field through the full-scale monitoring of vibrations and wind", PhD Thesis (in preparation)
23. Wind Induced Vibration of Stay Cables, Missouri Department of Transportation Research, Development and Technology, 2005
24. Caetano, E., Cunha, A., Gattulli, V., Lepidi, M., "Cable–deck dynamic interactions at the International Guadiana Bridge: On-site measurements and finite element modelling", Journal of Structural Control and Health Monitoring, 2008
25. Cheng, S., Larose, G.L., Savage, M.G., Tanaka, H., Irwin, P.A., "Experimental study on the wind-induced vibration of a dry inclined cable-Part I: Phenomena", Journal of Wind Engineering and Industrial Aerodynamics, 2008
26. Cunha, A., Caetano, E., Delgado, R., "Dynamic Tests on Large Cable-Stayed Bridge An Efficient Approach"
27. Main, J.A., Jones, N.P., "Full-scale measurements of stay cable vibration", Department of Civil Engineering, Johns Hopkins University, Baltimore, USA
28. Matsumoto, M., "Inclined cable Aerodynamics of Cable-Stayed Bridges", 7th European Conference on Structural Dynamics, Southampton, 2008
29. Matsumoto, M., Shirato, H., Yagi, T., Goto, M., Sakai, S., Ohya, J., "Field observation of the full-scale wind-induced cable vibration", Journal of Wind Engineering and Industrial Aerodynamics, 2003
30. Seidel, C., Dinkler, D., "Rain–wind induced vibrations – phenomenology, mechanicalmodelling and numerical analysis", Computers and Structures, 2006
31. Takano, H., Ogasawara, M., Ito, N., Shimosato, T., Takeda, K., Murakami, T., "Vibrational damper for cables of the Tsurumi Tsubasa Bridge", Journal of Wind Engineering and Industrial Aerodynamics, 1997
32. Chopra, A.K., "Dynamics of Structures", 3rd edition, 2006
33. Nieslony, A., Rainflow Counting Algorithm for MATLAB, Department of Mechanics and Machine Design, Technical University of Opole
34. ASTM E 1049-85 (Reapproved 1997), Standard practices for cycle counting in fatigue analysis, in: Annual Book of ASTM Standards, Vol. 03.01, Philadelphia 1999, pp. 710-718
35. Freyssinet H1000 Stay Cable System, (Freyssinet brochure)

Tom Singleton
Associate Vice President
Integrated Water Resources
Atkins North America

Abstract

The Sustainable Water Resource Management Plan for the City of Winter Haven, Florida, would restore as much as possible the networking of water that existed before the watershed was altered. It is an investment in the capacity of the natural landscape to provide multiple water resource benefits. In contrast, man-made structural solutions such as channels, ditches, reservoirs, and pipes are generally implemented primarily for singular benefits. In the long term, using the natural landscape to provide these multiple benefits will result in a less costly, more efficient water supply system for people and the environment.

The story of Winter Haven and the Peace Creek watershed not only provides compelling reasons why the protection and restoration of water resources are essential to a community's future, but the City of Winter Haven's ongoing efforts also serve as a good example of how other communities can work to understand and manage their water resources.

Animated and pdf versions of the Plan can be obtained at:

http://northamerica.atkinsglobal.com/WHSP

Introduction

The Winter Haven area has had a number of severe weather extremes in the past 10 years, including the lowest recorded lake levels in history in 2001, 3 hurricanes that caused extensive flooding in 2004 and one of the worst 3-year droughts on record from 2007 to 2010. Even before these extreme events took place, the goal of restoring the Peace Creek watershed and storing more water in the Polk County area had been discussed for at least the past 20 years. Today, the following conclusions are clear:

- Past efforts to manage water by draining, piping, and covering recharge areas in the Peace Creek watershed and regional aquifer drawdowns have had, and will continue to have, negative effects on Winter Haven's water resources, including lakes
- At best, today's regulations keep things the same but do not provide for restoration. At worst, they allow the further gradual degradation of water resources. The current rules for wetland mitigation and stormwater management encourage further degradation of the Peace Creek watershed
- The Peace Creek watershed will experience significant new growth in the coming decades that could further harm the networking of water in the watershed—or help it, if that growth is planned correctly
- Managing the area's water supply for both people and natural systems such as lakes are equally important
- Using the natural infrastructure, which provides many hydrologic benefits at a much lower cost, is far preferable to engineered, structural solutions that are expensive to build and maintain, and that generally address single issues
- Today's body of knowledge, gained through scientific studies, staff experience, and historical perspective, represents the best chance of addressing the watershed's water resource issues over the long term
- Managing land and managing water are inseparable activities and responsibilities
- As part of the much larger Peace River Basin, the City of Winter Haven can actively work to influence not only the health of its local water resources but also to help preserve the water resources of the entire region.

An unsustainable approach

The hydrologic alteration of the watershed that began in the early 1900s has reduced the watershed’s wetland areas by over 30%, or nearly 9,900 acres. As a result, at least 27 billion gallons of water are no longer stored in the landscape. This is no small quantity—it is enough to supply half of the water needs of the Winter Haven area for nearly 7 years.

The alterations to the watershed's hydrology have improved flood protection near drainage ditches such as the Wahneta Farms and Peace Creek Canals and made more land available for agriculture, homes, and businesses. However, the economic, environmental, and social costs of these alterations to the Winter Haven community, the Peace Creek watershed, and the larger Peace River Basin are far greater than their benefits. Many of these costs cannot be quantified and will more than likely be paid for with public funds in the future.

The most significant cost to the City of Winter Haven is the loss of its ability to provide adequate water resources in the future for both people and the environment (including lakes, wetlands, and aquifers). This loss affects the City's long-term economic viability. Major environmental costs include declining lake levels, decreased lake water quality, surficial aquifer drawdowns, navigable waterways that are dry part of the year, and less habitat for fish and wildlife. All of these result in a diminished quality of life and decreased sustainability.

For the larger region, the impacts include the environmental degradation resulting from reduced baseflow to the Peace River alternating with large pulses of nutrient-laden water during heavy storms, as well as regional aquifer declines that reduce the available supply of water for all uses.

In addition, the City (and the entire region) faces challenges such as changing rainfall patterns, prolonged drought, and continued economic and population growth. New development could result in further impacts if current regulations and approaches are followed. In the same way, increased water supply demands outside the watershed could make matters worse by lowering the regional aquifer further.

The current approach to managing water resources is not sustainable. If future residents want the same or a better quality of life, alternatives have to be considered as to how best manage water and land. The clear choice is to actively pursue partnerships between governmental agencies, citizens, and land development interests to restore and improve water resources, while moving forward with comprehensively planned and strategically located economic growth. If this choice is not made today, future generations will pay a high economic, social, and environmental price.

The Sustainable Water Resource Management Plan

The vision for the Peace Creek watershed consists of an interconnected hydrologic network of lakes, canals, wetlands, aquifers, open spaces, and parks that will help meet long-term water resource needs, including supply (water quantity), treatment (water quality), flood protection and the preservation of natural resources. The restored hydrologic network, combined with existing lakes and other waterbodies, resembles a "sapphire necklace" of water (Figure 1). Infiltration and treatment areas in the headwaters of the watershed would carry water into the surficial and Floridan aquifers. From there, it would move into a number of different storage and conveyance areas in the middle and lower reaches.

The Plan effectively balances the water needs of both people and the environment by using the capacity of the natural landscape as efficiently as possible, including preserving land for water storage. In other words, the system would be allowed to do naturally what it does best. In the long term, using natural infrastructure such as aquifers, lakes, and wetlands to provide services will result in a less costly, more efficient water storage and delivery system for all future generations. Ultimately, what is good for the lakes and the environment (storage, treatment, and recharge) is good for the community and economic growth (supply and flood protection).

The Sustainable Water Resource Management Plan is a cornerstone of the City's initiative to work towards a sustainable community. It represents the City's commitment to its current and future citizens to provide a desirable, enjoyable, beautiful and safe place to live. It preserves and

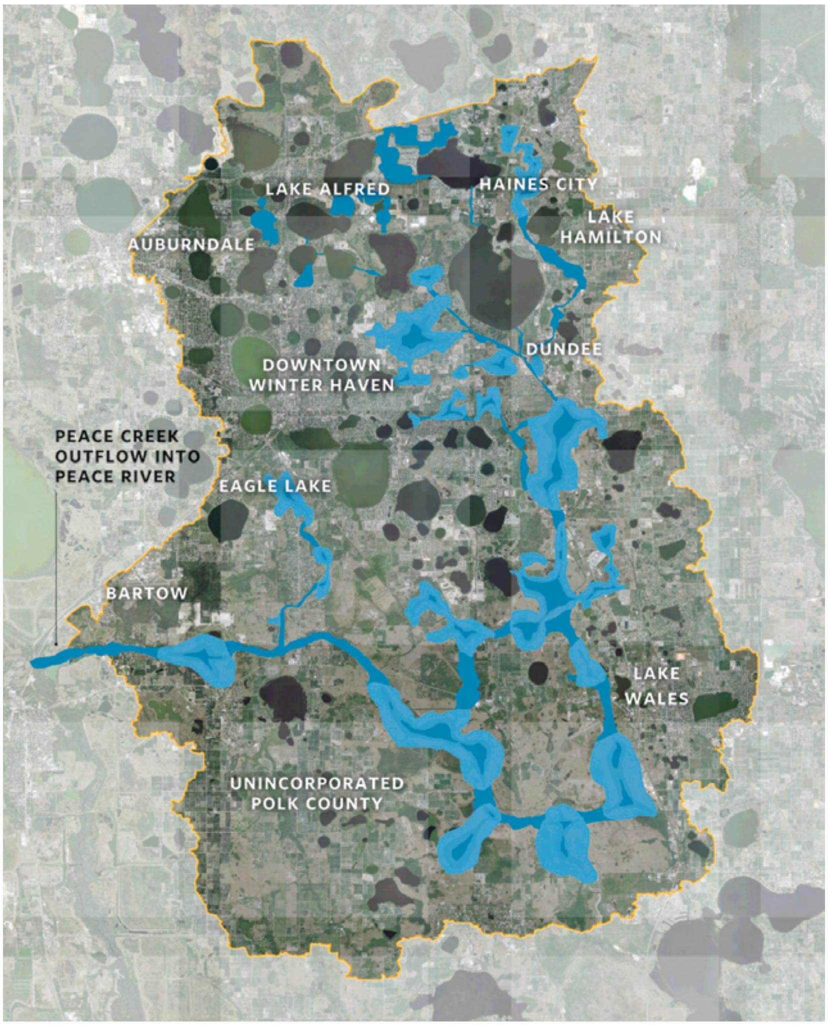


Figure 1 - The area's restored hydrologic network resembles a "necklace" of water, known in Winter Haven as the "sapphire necklace." Source: Atkins

enhances the innate ability of the watershed's unique landscape to provide many different water resource benefits, such as improved water quality, water supply, natural systems and flood protection. Incorporating restoration projects into a series of interconnected nature parks will provide trails, vistas, opportunities for wildlife observation, scenic beauty and water resource amenities that will attract growth and economic development. These amenities will help to integrate the Plan into the economic and cultural fabric of the City and make it sustainable.

Other benefits include a strengthened local economy; greater social, cultural and recreational opportunities; and a more aesthetically beautiful City. Together, all of these will enhance the community's quality of life and preserve it for future generations.

Goals of the Sustainable Water Resource Management Plan

- Meet the long-term water resource needs of people and natural systems
- Manage all sources of water—including floodwaters, stormwater reuse water, groundwater, and rainfall—as a finite, interconnected resource
- Establish a local leadership role in managing water resources and not rely on others to make decisions for the watershed
- Ensure that the unique characteristics of the Peace Creek watershed are considered in local and regional decisions
- Use existing experience, information and science to make sound decisions for today and the future
- Direct today's actions for a defined future result—that of water resource sustainability
- Preserve and restore the natural infrastructure as much as possible and use it to provide multiple hydrologic functions and benefits
- Ensure that all approaches are aligned with goals for economic growth; key considerations include providing opportunities for nature-based tourism, recreation, open space, water amenities and urban development
- Mitigate any impacts to water supply, water quality, hydrology and natural systems within the Peace Creek watershed
- Integrate water storage and treatment areas into the community using nature parks
- Manage land to improve water resources in the future and not allow their gradual degradation as in the past.

Benefits of the Sustainable Water Resource Management Plan

- **Water quality benefits** - Improves water quality in the Peace Creek watershed's lakes, rivers, and wetlands, and restores water quality in impaired waters as part of the state's Total Maximum Daily Load (TMDL) Program
- **Environmental benefits** - Restores; enhances water levels in the lakes and the wetlands surrounding those lakes; creates and protects quality habitat for fish, wildlife, and plants native to the community; and helps to restore natural rainfall and climate patterns in the area
- **Water supply benefits** - Recharges aquifers where all public water supply and most other supplies originate; provides more water for natural systems, lakes, and aquifers in the Peace Creek watershed; contributes to the maintenance of Minimum Flows and Levels (MFLs) in the Peace River; and is an important component of the Southwest Florida Water Management District's Recovery Strategy for the larger regional aquifer
- **Flood protection benefits** - Increases the capacity of the landscape to treat and store the water from small rainfall events that is currently being discharged from the watershed in the name of flood protection and, through increased storage and conveyance, provides much-needed flood protection during large storms
- **Economic benefits** - Restores and protects lakes, which are the reason that many people move to the area; expands the effective amount of waterfront to attract future development; creates economic opportunities for mitigation banking, water storage, and stewardship for landowners; facilitates economic growth by establishing a viable watershed approach to mitigation planning and stormwater permitting for future development; and saves money over the long term by using the natural infrastructure to provide valuable water resource services. Paying for these services now by restoring and preserving the natural infrastructure will provide more benefits and will be less costly than implementing and maintaining structural solutions in the future
- **Social, cultural, and recreational benefits** - Provides an integrated system of parks, trails, and other recreational areas (such as greenways and blueways), improves the area's aesthetic beauty, provides a more enjoyable and safe place to live with increased property values, strengthens the community's identity, creates a collaborative water resource management framework for many of the east Polk County cities and towns, provides the basis for making future land use decisions and improves quality of life.

The natural hydrology of the Peace Creek watershed

Florida's natural landscapes before European settlement were organized to efficiently store both surface water (lakes, rivers, and streams) and groundwater (aquifers), and to allow it to move slowly through the landscape. Figure 2 shows a typical natural landscape in Florida. In the headwaters of the system, lakes and isolated wetlands provide water quality treatment and infiltration into both the shallow and deeper underground aquifers. In some places, the aquifer literally seeps out of the landscape into wetlands, lakes and streams, creating streams that appear and disappear. These form disconnected strands and sloughs in the middle reaches of the watershed. The wetlands, along with

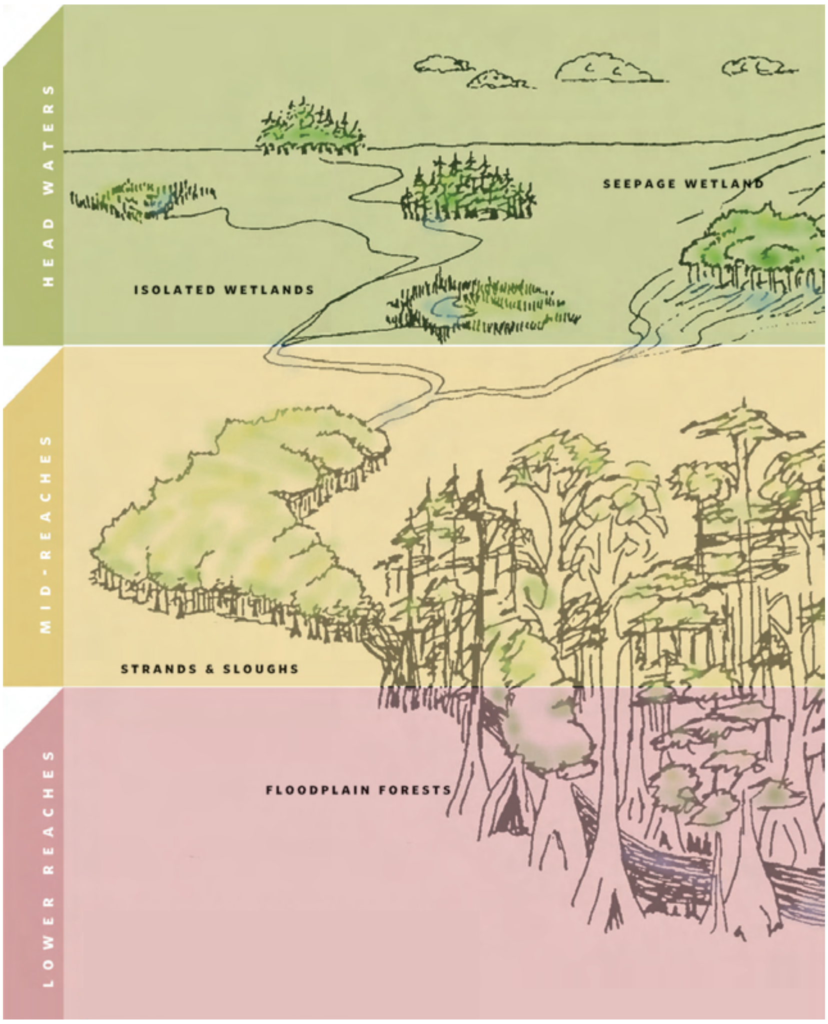


Figure 2 - A typical natural landscape in Florida, showing how water is retained and moves through the landscape. This graphic is also broadly applicable to the Peace Creek watershed. Source: Mark T. Brown for Atkins

the broad floodplain forests in the lower reaches, provide important surface water storage and conveyance during large storm events by allowing water to spread over the landscape.

The movement of surface water and groundwater in the natural Peace Creek watershed is similarly organized. In the headwaters/ Ridge area, where the elevation is highest (the area outlined in red at top left in Figure 3), ancient sand dune deposits contain sandy soils with very high infiltration rates. Over time, percolating rainwater dissolved areas of the underlying limestone and the surface subsided and formed lakes. These lakes are well connected to the aquifer system and rise and fall with aquifer levels.

There are very few, if any, surface streams in the sandy headwaters/ Ridge area of the watershed. Instead, rainwater infiltrated into the ground and then slowly moved through the surficial and Floridan aquifers to recharge the area's many lakes.

Groundwater that accumulated under the Ridge seeped out to form lakes and wetlands in the middle and lower reaches. This area of the watershed, the Polk Uplands, encompasses about two-thirds of the watershed (the term "Polk Uplands" is a misnomer of sorts, since this is actually the lowest part of the watershed).

During heavy rains, the lakes overflowed their banks, and the water moved across the landscape through elongated strands, sloughs and wetland areas, much like those of the Everglades in south Florida. These spread the water over a wide area and retained it in the landscape before the water flowed out of the watershed into the Peace River. During periods of drought and low rainfall, there was little flow in the Peace Creek system. However, during periods of high rainfall, excess water moved quickly through the system because it spread out over the landscape. In the lower portion of the watershed, where Peace Creek flows into the Peace River, sloughs likely connected to form forested swamp floodplains with a network of numerous small, meandering channels.

In its natural condition the Peace Creek watershed held water on the land surface in lakes, wetlands, floodplains, and streams, and held water below the surface in aquifers. The water stored in the surficial and Floridan aquifers in the upper part of the watershed kept lake levels higher during periods of low rainfall. Similarly, in the middle and lower portion of the watershed, wetlands and the surficial aquifer stored floodwaters and slowed water flow, gradually releasing the water to maintain higher flows in the Peace River during dry seasons.

The hydrologically altered Peace Creek watershed

Although today we know more about the long-term effects of altering an area's hydrology, this was not known or understood historically. In many watersheds nationwide historical drainage projects and development patterns have led to a significant loss of hydrologic function. That has led to long-term impacts to water supply and water quality, lowered lake levels, increased flooding, reduced river flows and habitat destruction. The movement and storage of water are altered for human uses such as agriculture, homes, schools, businesses, industry and roads. Urban development often occurs first in the highest, driest areas with the greatest potential for aquifer recharge. The last areas to be developed are usually those that were historically wet; often these are still prone to flooding.

The hydrologically altered Peace Creek watershed is no exception. The higher, drier areas in the headwaters/ Ridge portion of the watershed were developed first for residential and business use. These high-recharge areas historically helped maintain water levels and water quality in the lakes, streams, and aquifer systems. Historical drainage "improvements" (as opposed to

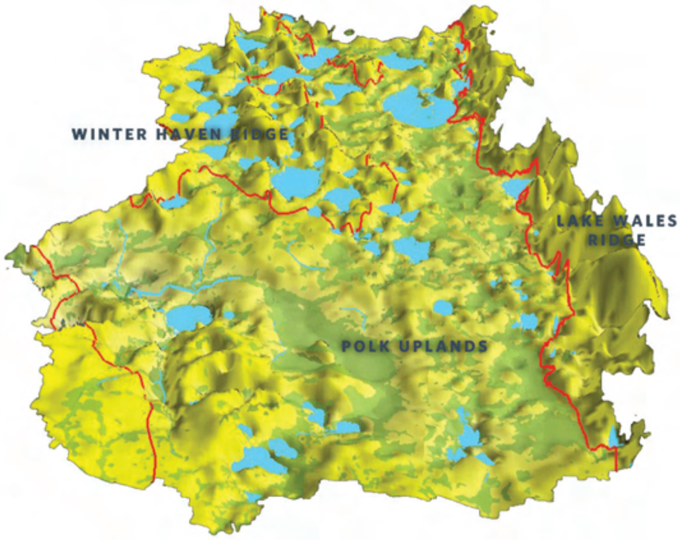


Figure 3 - A perspective view of the natural Peace Creek watershed, showing the large number of lakes in the headwaters (on the Winter Haven Ridge) (top left), and the large number of wetlands (shown in light green) in the middle and lower portions (in the Polk Uplands), located between the Winter Haven and Lake Wales Ridges. The elevations shown in this map are exaggerated by a factor of 10 to differentiate the high, dry land of the headwaters/Ridge area from the low-lying land in the middle and lower reaches/Polk Uplands area. The red lines represent different regions that in part reflect these differences in elevation. Source: Atkins

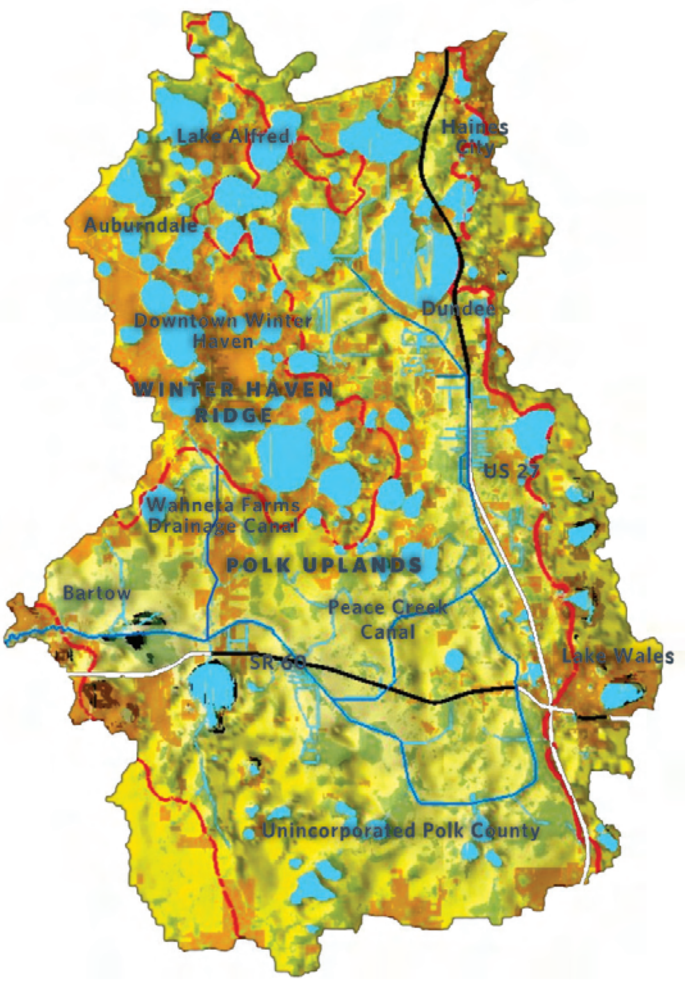


Figure 4 - A plan view of the hydrologically altered Peace Creek watershed, showing the Peace Creek and Wahneta Farms Drainage Canals (the dark blue lines at the bottom), which initially provided significant drainage improvements for human uses. As the community of Winter Haven was developed, agriculture gave way to urban development in these low-lying areas, which are still prone to flooding. Source: Atkins

present-day stormwater treatment systems) associated with the resulting development took the rainwater that once percolated underground into the surficial aquifer and instead directed it into the lakes. Regional agricultural, urban and phosphate mining withdrawals lowered Floridan aquifer levels. The resulting changes in infiltration, flow and storage have contributed to lowered lake levels and degraded water quality.

In the early 1900s, a number of canals were constructed in the watershed (Figure 4): the Peace Creek Drainage Canal was constructed to provide an outflow for the Northern Chain of Lakes and to drain the surrounding lands (Figure 5); the Wahneta Farms Drainage Canal was built to allow outflows from the Southern Chain and to drain neighboring areas; and numerous other canals were constructed to connect Winter Haven's lakes so that they were navigable. As a result, water was rapidly moved out of the watershed, and levels in a number of lakes and the surficial aquifer have dropped by approximately 10 feet over the last 90 years. In addition, some of the canals connecting the lakes go dry during drought years.

Today, many of the 50 lakes in the City are listed as impaired for nutrients, which can lead to the overgrowth of algae that reduces oxygen levels in the water and causes fish kills. The City's water supply wells are located in the headwaters/Ridge area of the watershed. Most of the water pumped out of the ground in this area is discharged from the headwaters, where it could do the most good. Five million gallons per day of treated wastewater are sent to the Peace Creek Drainage Canal in the southern part of the watershed, where the water very rapidly flows downstream to the Peace River. New development, especially in the storage/conveyance areas of the watershed, could result in the further dewatering of the watershed, since existing stormwater, wetland and floodplain regulations allow further degradation to water resources.

A plan for preserving and restoring the water resources of the Peace Creek watershed

When implemented, the Sustainable Water Resource Management Plan would increase aquifer storage throughout the watershed and restore about 6,300 acres of wetlands in the middle and lower reaches/Polk Uplands area. The principal opportunities for restoring the watershed's hydrologic function are as follows:

- Increased treatment and infiltration - The most important opportunity in the headwaters/ Ridge area, including the residential and commercial areas around downtown Winter Haven, is to restore the infiltration of stormwater to recharge the surficial aquifer. This will provide storage capacity and flow for lakes during dry seasons, and will also ensure that adequate treatment is provided before the stormwater reaches the lakes or the aquifer. Treated water from local wastewater facilities (called reuse water) also provides opportunities for greater aquifer recharge and increased lake levels in the headwaters/Ridge area
- Increased storage and conveyance—The most important opportunity in the middle and lower reaches/Polk Uplands area, which has the highest development pressure today, is to provide for additional storage of water and wider conveyance paths through the system during wet seasons. Increasing the amount of available storage will allow more water to be retained in the lakes in the headwaters/Ridge area during years with a water surplus. In the event of a storm, excess water will flow into the middle reaches, where it will be stored in the landscape (wetlands). Increased storage on the land results in higher surficial aquifer levels, which will provide more consistent outflows to Peace Creek and the Peace River during dry seasons. Finally, in the lower portion of the watershed, a wider conveyance area will allow large volumes of water to move through the system more quickly with less damage to property during flood events. At the same time, planning for this increased conveyance with future development will protect property from flood damage.

The Plan links most of the east Polk County communities in the Peace Creek watershed hydrologically, providing a basis for consistent, comprehensive land and water planning on a regional scale, as well as a means to develop common recreational amenities. Regional storage and mitigation provide benefits to individual landowners, allowing development to occur, while preserving natural systems and meeting the water resource needs of people and the environment. At the same time, flood protection for adjoining neighbors or the region is also improved.

Implementing the plan

Increasing water treatment and infiltration in the headwaters/Ridge area

To provide for the infiltration of stormwater in the headwaters/Ridge area of the Peace Creek watershed, open land is needed in an already developed urban landscape. An initial goal is to devote 5% of the watershed's current land area to implementing stormwater infiltration technologies such as rain gardens, swales and percolation systems. This percentage is based on the calculation that about 2 billion gallons a year of stormwater are no longer infiltrated because of altered hydrologic function, and 5% of the watershed's land area would be needed to infiltrate that quantity of water annually.

By providing depressions in the landscape to collect runoff before it enters storm drains, the City can capture most of the runoff so it can be infiltrated into the surficial aquifer. The best strategy is to develop as many small depressions as possible, spread throughout the landscape, to mimic infiltration patterns in the natural system.

Different approaches to capturing stormwater are required for the various land use types and locations in the watershed, including residential and downtown commercial

Figure 5 - The Peace Creek Canal has effectively drained agricultural lands during the past century. Source: Southwest Florida Water Management District, March 2006, Southern Water Use Caution Area recovery strategy

areas, and at the City's edge.

(1) Opportunities to capture stormwater in residential areas

Figure 6 shows a stormwater capture system that covers 5% of the surface area (highlighted in green) of an existing residential neighborhood in southwest Winter Haven near Lake Shipp. Stormwater can be collected in the following ways:

- Capturing the water that runs off the roofs of private homes. Private rain gardens in homeowners' yards can be incorporated into landscaping and located anywhere in the yard where a depression can be created to capture water from the roof
- Capturing the water that runs off streets and driveways. In some areas, water can be collected in long, vegetated swales that parallel the road. These swales can be located in a median if present, or within the public right-of-way along a road
- Using portions of neighborhood parks for larger retention areas. There are two different types of nature parks, or hydric parks. Some contain water all the time, and others fill with water only during heavy storms.

(2) Opportunities to capture stormwater in the commercial areas

Figure 7 shows a stormwater capture system (highlighted in green) that covers 5% of the downtown Winter Haven commercial district. Commercial areas can be one of the most challenging places to capture stormwater because of the high percentage of impervious surface and lack of available land. Besides roofs and roads, parking lots make up a

Figure 6 - Aerial photo of a residential neighborhood in southwest Winter Haven, showing a stormwater capture system that covers 5% of the surface area (highlighted in green). Source: Mark T. Brown for Atkins

large portion of the surface area. Three principal ways exist to capture stormwater in commercial areas:

- Constructing swales and rain gardens within and along the edges of parking lots. This is one of the best ways to capture stormwater in areas with little land. Some parking spaces could be lost to provide enough area for runoff infiltration
- Constructing roadside swales in the City's commercial district to capture runoff from street surfaces. Although swales are an important way to capture stormwater and provide treatment in the headwaters/Ridge area, they are particularly important in providing treatment before stormwater runs into the lakes
- Retrofitting parks and open spaces with shallow depressions to collect runoff. In higher density commercial areas, such as downtown Winter Haven, rain gardens and swales will be smaller but more numerous than in the more open residential areas. The City has a central park that provides a large, open space in the urban center. This provides some opportunities for retrofitting, and helps to prevent a heat island effect through the use of green spaces. Retrofitting is especially important in areas that are being redeveloped in the headwaters/ Ridge portion of the watershed. The use of pervious concrete in areas undergoing redevelopment would also increase the amount of stormwater captured.

(3) Opportunities to capture stormwater at the City's edge

Some of the greatest opportunities to maximize treatment and infiltration are found at the edges of cities, where development has not yet occurred, or is still occurring. Planning, design, permitting, and implementation should all reinforce each other, and should focus on the preservation of natural hydrologic function and as many existing natural features as possible. There are four major recommendations for developing areas at the outer edges of Winter Haven, as follows (Figure 8):

- Preserve pre-existing, low-lying areas that collect and infiltrate rainwater. Don't fill these in, but allow them to continue their important and essential function in the landscape, and enhance these areas where possible
- Enhance and extend the wetland edges of lakes with nature parks, or hydric parks. These wetland areas can be used to treat stormwater before it enters the lakes, providing water quality benefits. They also store floodwaters and become recreational attractions
- Preserve wet corridors in the landscape. During heavy rainfall, having a path for the water to move across the landscape without affecting permanent structures is critical to maintaining high water levels and healthy lakes during normal weather patterns. These wet corridors can be enhanced to create wetland areas that act as wildlife and recreational corridors in the landscape
- Require new developments to use low-impact stormwater management techniques, such as rain gardens and percolation ponds. Requiring a new development to manage all its runoff within the development itself ensures that it is not creating a problem that will require more land elsewhere to solve.

Figure 7 - Aerial photo of the downtown Winter Haven commercial district, showing a stormwater capture system that covers 5% of the surface area (highlighted in green). Source: Mark T. Brown for Atkins

Increasing water storage and conveyance in the middle and lower reaches/Polk Uplands area

Ninety percent of storm events in Florida generate less than an inch of stormwater runoff. This water should be retained on the landscape and allowed to infiltrate. However, in a channelized system such as Peace Creek, the water from many of these smaller storms is conveyed off the land as stormwater. With more than an inch of rainfall, only the excess should be discharged. For peak events, all of the water can be passed through the system and discharged, but only when absolutely necessary to prevent flooding.

Increased water storage can be provided by creating wetland storage areas, stormwater treatment areas, and forested wetland sloughs to help buffer the system against droughts, while at the same time providing flood protection (see Figure 9). This can be done by designing a system with significant surface water storage along the current Peace Creek Canal, and a wider conveyance corridor that facilitates high-volume flows during flood events, as shown in the cross-section view of the restored canal in Figure 9.

Wetland storage

In the restored hydrologic network, portions of the Peace Creek Canal would be strategically dechannelized in as many places as possible, and barriers (called ditch blocks) would be placed in the canal to hold back water and allow it to spread across the landscape, as shown in Figure 9. These changes would slow down the water moving through the system, and the water would be stored for a much longer time, allowing it to recharge the surficial aquifer.

The newly created wetland storage areas would hold floodwaters to reduce flooding, and then release the water slowly downstream to increase baseflow in Peace Creek and the Peace River during the driest times of the year, when water is most needed in the system. By storing more water, these areas would also reduce peak flows during heavy rains, helping to provide flood protection downstream. In addition, the wetlands would serve as valuable waterfront amenities, providing recreational opportunities and wildlife habitat.

Wetland storage areas should be designed with varying water depths that allow a wide range of wetland habitats and support diverse plant and animal life. Higher land areas with shallow water should be provided to support forested wetlands and emergent wetland marshes. Medium-depth pools provide for submerged vegetation. Deep open-water areas create refuges for wildlife when the system goes through dry cycles and the majority of the wetland dries out. The variation in water depth and hydroperiod will create a system that contains all the biological components necessary to adapt to changing conditions in the future, with little human maintenance.

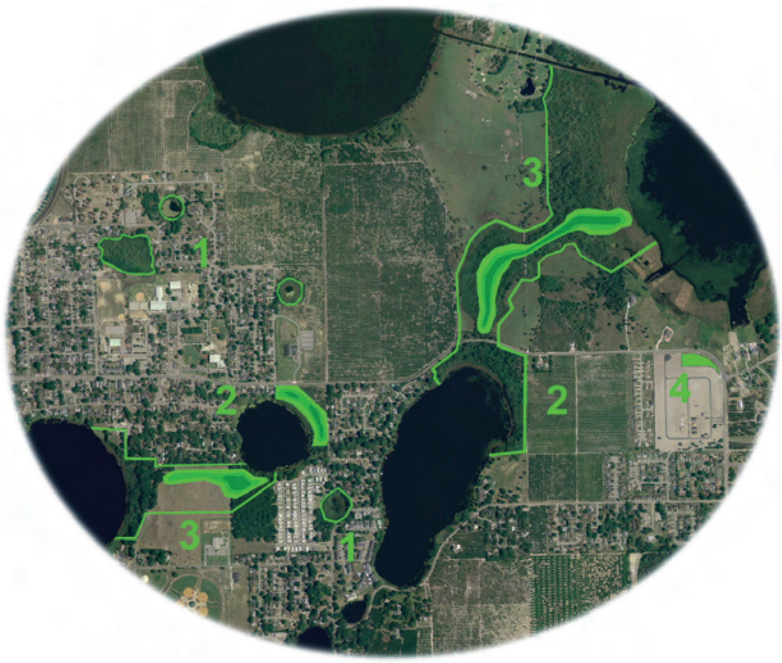


Figure 8 - Strategies for permitting development in undeveloped areas of Winter Haven include (1) preserving existing low-lying, undeveloped areas, (2) enhancing and extending wetland lake fringes, (3) preserving wet corridors, and (4) requiring low-impact development. Source: Mark T. Brown for Atkins

Stormwater treatment

Wetland areas in the restored Peace Creek watershed would treat stormwater flowing off developed and agricultural lands. These areas would naturally remove nutrients, sediments and other pollutants through biological, chemical and physical processes. The large surface area would provide high-efficiency treatment for normal amounts of rainfall and significantly increase water quality in the system (Figure 10), helping to prevent waterbodies from becoming impaired and restoring and preserving water quality in urbanizing areas. The treatment areas would provide additional groundwater storage through slow infiltration. Portions of the wetlands would be designed to provide additional stormwater treatment to address existing or future water quality impacts. These created wetlands could also provide further treatment for reuse water before it is discharged to aquifers, lakes or wetlands.

Storage and conveyance

The wetland storage and stormwater treatment areas, both of which store surface water, should be connected by forested wetland corridors, or sloughs, that can accommodate flows after heavy rains, when large quantities of water need to be moved quickly through the system (Figure 11). During periods of excessive rainfall, water would flow over the barriers and across a broad cross-section of the canal (an area broader than the canal) to maintain flood protection to neighboring properties. At the same time, these corridors should help to minimize the amount of water lost from the watershed during the dry season, when water storage is important.

SUSTAINABILITY

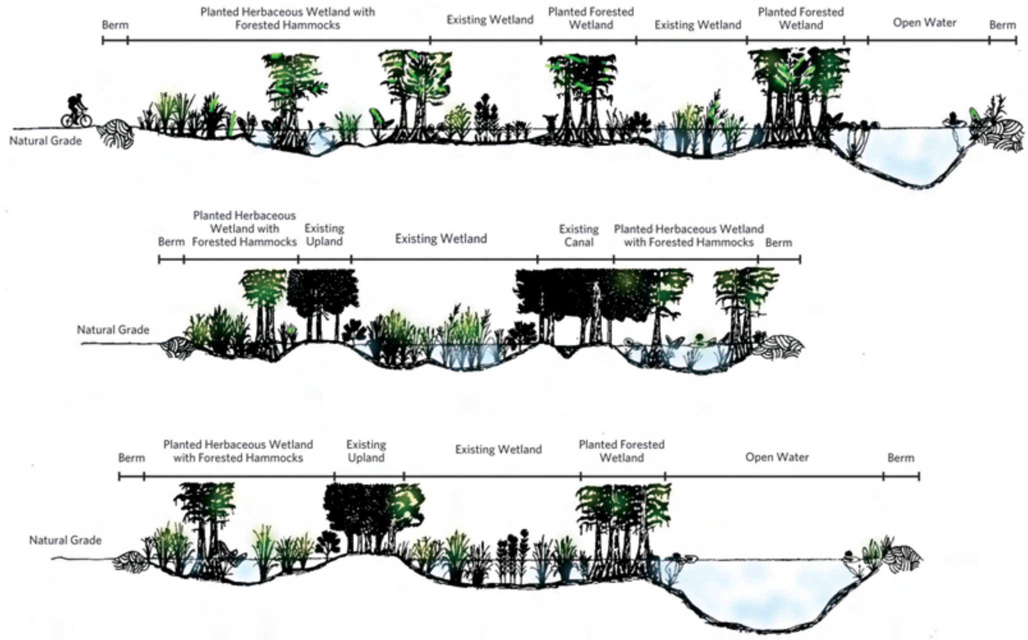


Figure 9 - Cross-section of a wetland storage area. The center of the middle graphic shows the existing canal, with wetland storage on either side. In some places, the canal will actually remain as a central feature, and in other places the channel could be expanded, as shown in the top and bottom graphics. Wetland storage areas should be designed with variations in water depth to allow a wide range of wetland habitats and support diverse plant and animal life. These variations will create a system that contains all the biological components necessary to adapt to changing conditions in the future, with little human maintenance. Flood protection to the surrounding lands will be maintained or improved by increasing the cross-section width of the discharge area. Source: Mark T. Brown for Atkins

The numerous small stream channels in a slough create friction, helping to slow the water. When water levels rise, a much wider corridor allows large amounts of flow. This wider area is kept clear of vegetation and brush by the tree canopy overhead. The wetland trees shade out vegetation below and can tolerate the system's constant variation in water levels.

The conveyance capacity in the restored system is equal to or greater than that of the existing system, including an allowance for friction loss. For example, if the existing channel was 60 feet wide by 10 feet deep (or 600 square feet), the restored system would provide the same conveyance capacity if it was 200 feet wide by 3 feet deep. The revised capacity would be validated through modeling, using the Southwest Florida Water Management District's Peace Creek model. Using this approach, the sloughs will at least maintain, if not enhance, the capacity of the system for flood protection.

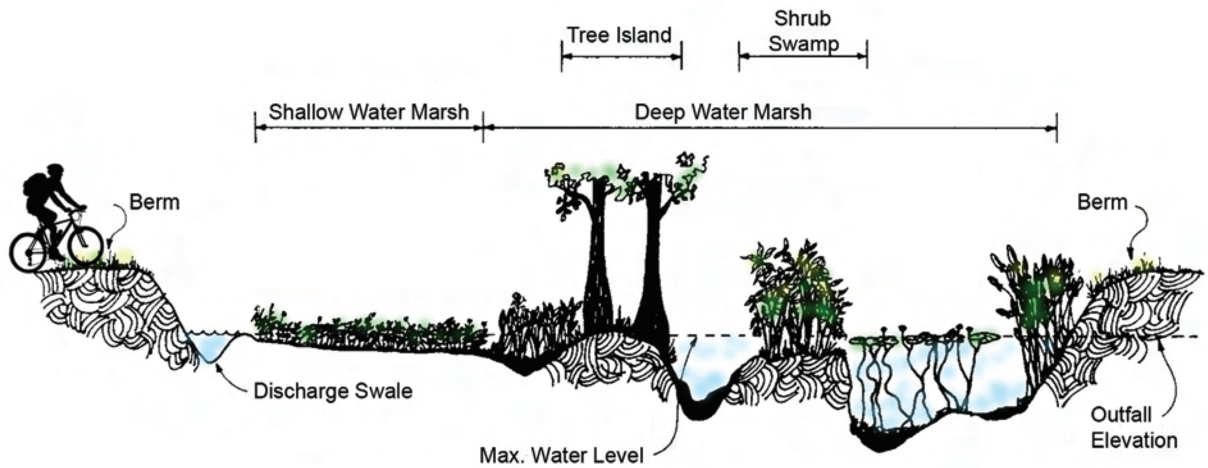


Figure 10 - Cross-section of a typical constructed stormwater treatment area. Source: Mark T. Brown for Atkins

SUSTAINABILITY

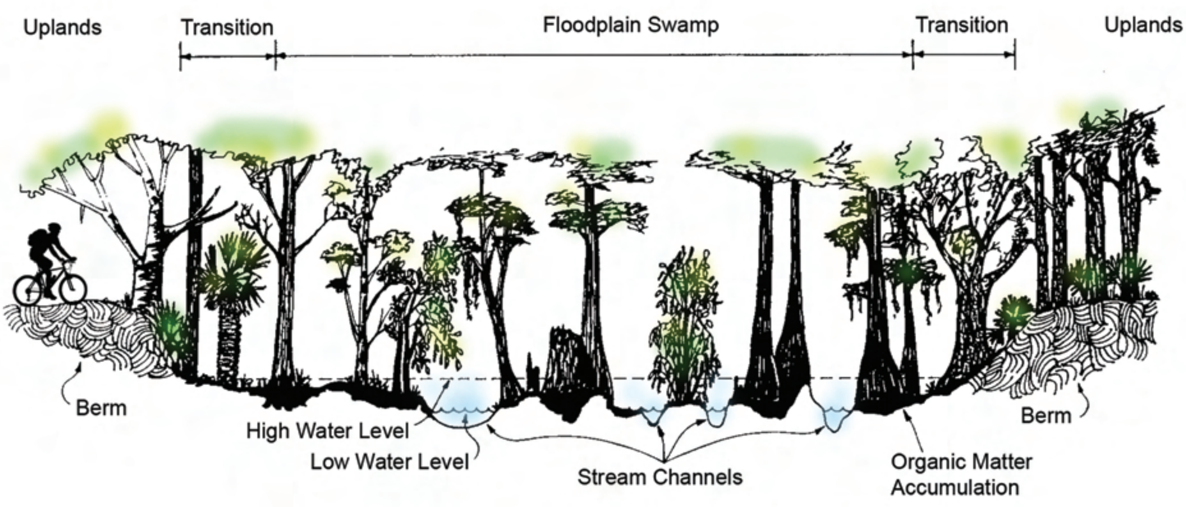


Figure 11 - Cross-section of a typical forested wetland slough with multiple stream channels that provide paths for water to flow across the landscape slowly, and wide corridors that can move floodwaters through quickly. Source: Mark T. Brown for Atkins

Priorities, strategies, and recommendations for implementing the plan

The Plan can be used to direct a wide range of activities to restore and protect water resources on a neighborhood, community, and regional scale. The recommendations and strategies discussed in the Plan are, in essence, a set of tools that can be used to fund, direct, and support the implementation of the Sustainable Water Resource Management Plan, and to measure the results of implementation. They do not require a choice between people and the environment, but support both.

The tools to be considered - which can be political, regulatory, financial, incentive based, or project related—can be used simultaneously on both local and regional scales. They include land use planning, mitigation banking (and other funding options), redirecting stormwater and reuse water, and monitoring to measure progress towards specific goals. They build on activities that are already under way in the City of Winter Haven, Polk County, and the larger region, as well as those of regional and state agencies, and provide a long-term approach to implementation. They can be carried out by the City as part of a coordinated, collaborative effort with Polk County and other communities in the region, as well as with state and regional agencies, to minimize costs and avoid duplicated effort. Opportunities for public/private partnerships as well as profit-making private initiatives are also encouraged to further the Plan's implementation.

Many of these tools are subject to additional evaluation to identify their feasibility in the Peace Creek watershed and the City of Winter Haven. Some options may be more feasible than others; some may be more cost-effective; some can be implemented relatively quickly; and others would need to be implemented over a period of years. The tools will also need further fine-tuning in terms of design and implementation. In addition, new tools may become available that are not discussed in the Plan.

Conclusion

Over the past century, the Peace Creek watershed was drained to make way for agricultural and urban development. As a result, much of its natural hydrologic function—including aquifer recharge, storage in lakes and wetlands, water quality treatment, and measured releases to the Peace River downstream—was destroyed or significantly altered. The consequence of this historical approach is that today the City of Winter Haven and the other communities in the Peace Creek watershed will not have enough water to meet the long-term needs of both people and the environment in the face of continued economic and population growth.

The Peace Creek watershed receives no water other than rainfall. What falls on the ground is all that is available for current residents, natural systems and future growth. This situation is unique to the entire Peace River Basin and the region encompassed by the Southwest Florida Water Management District's Southern Water Use Caution Area (SWUCA).

Protecting and restoring the Peace Creek watershed's hydrologic system so that water is stored naturally during the wet season for use in the dry season—and also stored during wet years for use in dry years—is critical to meeting the future water resource needs of people and the environment. Natural storage, with its many benefits, including natural systems and flood protection, is much more desirable than structural, man-made storage, which usually has only a single purpose, is expensive to construct and maintain, and often is limited in managing extreme events (for example, no matter how much money is spent, there will always be storms that exceed a structure's design capacity).

Merely holding the line on maintaining current aquifer levels is not enough: the City needs to identify specific actions to restore aquifer levels, which are critical to the future sustainability of the region.

Many of the watershed's historical wetland areas still exist, but future development plans may preclude the use of these areas in the future; thus the City needs to act now.

Through the implementation of the Sustainable Water Resource Management Plan, the City has the opportunity to serve as a model for other communities in the region by carrying out many restoration activities within the Peace Creek watershed. However, a coordinated effort is needed to solve the larger problems that are outside the City's jurisdiction but that affect all communities in the area. Restoring the regional aquifer and addressing other issues such as diminished Peace River flows must be carried out in collaboration with the other communities in the watershed, in concert with the Southwest Florida Water Management District, Florida Department of Environmental Protection, Florida Department of Transportation, Polk County, Lake Region Lakes Management District and other agencies. Working together, these entities can leverage common goals to integrate regional water resource planning with local land use decisions.

There is no project or single solution that will restore aquifer levels; it will require a comprehensive and adaptive management approach, sustained long-term effort and significant funds. To a large extent, future development will need to pay for restoration through mitigation banking and other activities. In turn, developers will require economic incentives to carry out restoration work. These incentives might include water supply for new growth, waterfront development amenities, cultural and recreational resources, expedited permitting, and mitigation banking opportunities.

Ironically, while the watershed was dewatered as a result of historical development, one of the best opportunities to preserve and restore lost hydrologic function is to partner with future development. The key lies in managing the types of development that are planned and built, where they are located, and the conditions that the City and other governmental agencies and entities place on that development. The City's long-term future is sustainable only if the watershed is developed in a way that is sensitive to the unique land and water resources in the region. This will take a bold, new approach, as well as courage and leadership. Current and future efforts to restore the Peace Creek watershed's water resources must be supported and encouraged by agencies and entities at all levels, both public and private, as well as individual landowners and residents.

Acknowledgements

The Atkins project team included the following individuals: Tom Singleton, Project Director, Cheryl Propst, Project Manager, Doug Robison, Principal, Quality Assurance/ Quality Control, Ed Cronyn, Dave Tomasko, Ph.D., Pam Latham, Ph.D., Ralph Montgomery, Ph.D., Robert Woithe, Ph.D., Robert Viertel, PG. Mark T. Brown, Ph.D., Professor in environmental Engineering Sciences and Director of the Howard T. Odum Center for wetlands, University of Florida, Gainesville, and David Pfahler, a Ph.D. candidate in the Department of Environmental engineering, University of Florida, Gainesville served as project consultants. Special acknowledgement to Mike Britt, P.E., Director, Natural Resources Division, City of Winter Haven, who had the knowledge, experience, and vision to conceive of the Plan. Special thanks to the Atkins Media Group in Miami, Florida, and to Word, Ink, Maitland, Florida, for their extraordinary contributions to the Plan.

GIS and Information Management on Crossrail C122 Bored Tunnels contract

Richard Budden

Principal Geospatial Consultant

Atkins Middle East

Abstract

The implementation of Geographical Information Systems (GIS) technology on large infrastructure projects is yet to reach the full extent of what it can achieve. Whilst being a 'no-brainer' to GIS savvy people, the technology is still widely perceived as a 'nice-to-have' or additional service and is often a victim of cuts to project budgets.

This paper demonstrates how GIS and information management principles were implemented on the Crossrail bored tunnels contract, and is an exceptional GIS success story. It demonstrates and proves the savings in time and money that can be achieved, as well as the mitigation of risks that stem from inappropriate use of data and information.

The project further demonstrates a seamless integration of CAD and other specialist software in 2D and 3D. What makes this example remarkable is the scale of the project with regard to duration, budget, number of stakeholders, amount of data and information, variety of software and data formats.

The project is still ongoing and new opportunities for further GIS work and improvements through GIS arise daily.

Introduction to Crossrail

Crossrail is the new high frequency railway for London and the South East. From 2018 Crossrail will travel from Maidenhead and Heathrow in the west to Shenfield and Abbey Wood in the east via 23km of new twin tunnels under central London. It will link Heathrow Airport, the West End, the City of London and Canary Wharf.

The Crossrail project is the largest civil engineering project in the UK and the largest single addition to the London transport network for over 50 years.

Crossrail is both an interesting and challenging geographical project.

The aim of this paper is to explain how Geographical Information Systems (GIS) technology has been used to understand geographical constraints and manage information efficiently, and the benefits this approach brought to this project.

Figure 1 shows the Crossrail alignment through London.

The overall objective of the Crossrail project is to deliver the new railway, from planning, design and construction to maintenance and running of the service.

Atkins and Arup work as a joint-venture on the bored tunnels contract, referred to as contract C122. It is one of the largest design packages of the entire Crossrail project in terms of size, staff and budget. The contract is for the design of 42km of bored tunnel beneath central London, interfacing with 8 new underground stations. The scope includes work in the following areas:

- Development of previous scheme design 3 baseline into a fully engineered design
- Ground behaviour, settlement and damage assessment
- Geotechnical services

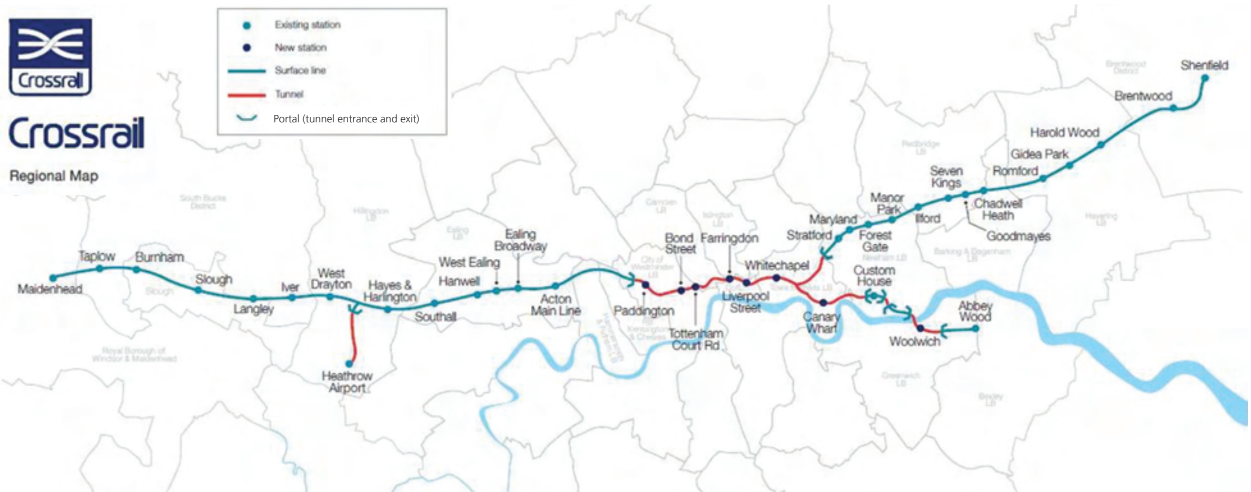


Figure 1 - Crossrail Route

- Instrumentation and monitoring
- Noise and vibration analysis
- Flood protection and drainage
- Interface management
- Alignment design
- Planning and environment
- Provision of the ITT scope document packages for potential contractors

What is hereinafter referred to as 'the project' describes the implementation and use of GIS and information management on Crossrail from the central Crossrail GIS and the C122 contract GIS perspective, rather than the overall Crossrail project itself.

Objectives of the project

The project started in May 2009 and is still ongoing as of January 2011. The construction of bored tunnels and new underground stations beneath central London could potentially affect more than 3500 buildings and structures due to ground settlement. Part of the scope is to identify potentially affected structures, determine appropriate mitigation measures and create reports and drawings for issue to client and contractors detailing the damage assessment results.

A second aspect of the contract is to identify potential obstructions to the new tunnels. Obstructions in this context can be understood as objects that are physically in the path of the tunnel boring machines, for example building foundations, existing London Underground tunnels, or deep utilities. Obstructions can also be existing structures in the proximity of the tunnel alignment which might get damaged or have an impact on the construction works.

The array of disciplines involved in the project include structures, geotechnics, tunnelling, noise and vibration, commitments, interfaces, and heritage. These all generate and demand a huge amount of information every day on a project of such large scope. In addition to this there is a vast amount of historical information, surveys, reports and drawings from previous stages of the project, generated or collated by other consultants. It is vital to the success of the project that this information is readily available to all staff who require it, and that it is reviewed and updated where new or more accurate information is found. The number of users, the amount of data and information and the variety of systems, formats and data storage locations, combined with ongoing changes to the scope, the running tunnel alignment and other related components provides many challenges to the GIS and information management teams from Crossrail and the C122 contract. Information also has to be shared with various third party interfaces, including the client and the delivery partner, other consultants, agencies and external stakeholders.

The role of GIS and information management on the project was designed to primarily achieve two goals:

- Reduce risk resulting from unmanaged or badly managed data
- Improve efficiency in workflows and data access through the implementation of spatial technology and information management procedures

The approach of the GIS teams has been to re-use as much valid existing information as possible, as opposed to re-generate it. This has required the adoption of proper data management structures and procedures. Tools have been created, adopted and tailored to make them 'fit for purpose' and appropriate for the specific requirements of the project. For the majority of the engineers, managers and technicians on the project this has required a buy in to certain technologies and methods of working that are new to them. This has required a lot of mutual trust and transparency, which could only be achieved through good ongoing communication between all members of the team.

Technology components

The major technology components in use on the project are:

(1) Databases - MS Access, Oracle Spatial

(2) CAD - Bentley Microstation, ProjectWise

(3) Desktop GIS - ESRI ArcGIS, Bentley Maps

(4) Web GIS - Bentley Geo Web Publisher (Crossrail Maps)

(5) Office - Microsoft Office Suite (MS Word, MS Excel)

(6) Specialist Software - FME, HoleBASE, XDisp

(7) Document Control - DomDoc, Documentum

Databases

The main spatial database (Oracle Spatial 11g) is held and maintained by the 3 staff strong central Crossrail GIS team. It enables data storage as well as data to be published through Bentley Geo Web Publisher which forms the basis of the corporate web GIS system, Crossrail Maps.

The 2 staff strong C122 GIS team use a combination of the ESRI personal geodatabase format and Microsoft Access and for storing spatial and non-spatial data. That the personal geodatabase is based on MS Access has been an important success factor in this approach.

CAD

The Bentley software suite including MicroStation and ProjectWise is Crossrail's choice for CAD software. Procedures to update CAD models held within ProjectWise with the information held in the spatial database have been seamlessly integrated.

Desktop GIS

While the Crossrail GIS team uses the Bentley Maps and Bentley Administrator products, the C122 contract chose the ESRI ArcGIS software suite. This was decided following a review of the technology available, and the decision was made taking into account the requirements of the project. The Enterprise Licence Agreement (ELA) between ESRI and Arup has allowed the team to use licensed ESRI software on Crossrail at no additional cost to the project or client.

There have been no interoperability issues or losses due to the integration of software from different vendors, and indeed taking a non-application centric approach has played its part in the project's success.

Web GIS

The Crossrail Maps web GIS, which is built on Bentley Geo Web Publisher, provides a means to view and query information through an intuitive map interface, to all staff working on the wider Crossrail project. The tool is accessible on the Crossrail intranet through Internet Explorer, so no additional software has had to be installed on users PCs.

Microsoft Office suite

There was a realisation at the beginning of the project that there were very limited GIS or database skills amongst the engineers and design staff in the C122 project team. In order to enable people to gain access to the non-spatial information held about buildings and other obstructions, tools and procedures have been implemented that allowed the data users to stay within their comfort zone and area of expertise, yet still access the information that they need. The two most important elements of this are MS Excel-based database 'front-ends', which are linked to the Access database enabling instant data refresh, and MS Word based report templates, which enabled the automation of the report creation process.

Specialist software

A key component in the GIS implementation on the C122 project is the use of Safe Software's FME application. This software is used to extract, transform and load spatial data and is employed for a wide variety of tasks on the project. These include the conversion and output of DGN files from GIS data for updating CAD models, and the conversion of coordinates from British National Grid to London Survey Grid, which is in use on the project.

Other specialist software includes the use of XDisp to calculate ground settlement and damage caused by tunnelling, and HoleBASE which is a data management tool for ground investigation data.

The data centric approach taken has enabled the straightforward integration of all specialist software and data types used on this project.

Document control

Crossrail had originally implemented the collaborative document management system DomDoc at the time the contract started, but have since switched to Documentum. Useful information from previous phases of Crossrail is stored on DomDoc/Documentum and where related to specific spatial assets, is made accessible to users through hyperlinks from objects on the web GIS.

Document control also provided hundreds of document numbers for the automated damage assessment reports.

Implementation of the project

The approach described here is currently fully implemented and operational, and the project is expected to run into mid 2011. The project team currently numbers more than 200 staff, all utilising and benefitting from the tools and procedures outlined in this paper.

The information flow chart in Figure 2 outlines the approach, and has been implemented since the beginning of the C122 contract.

The components of the information flow chart will be expanded upon in the following sections.

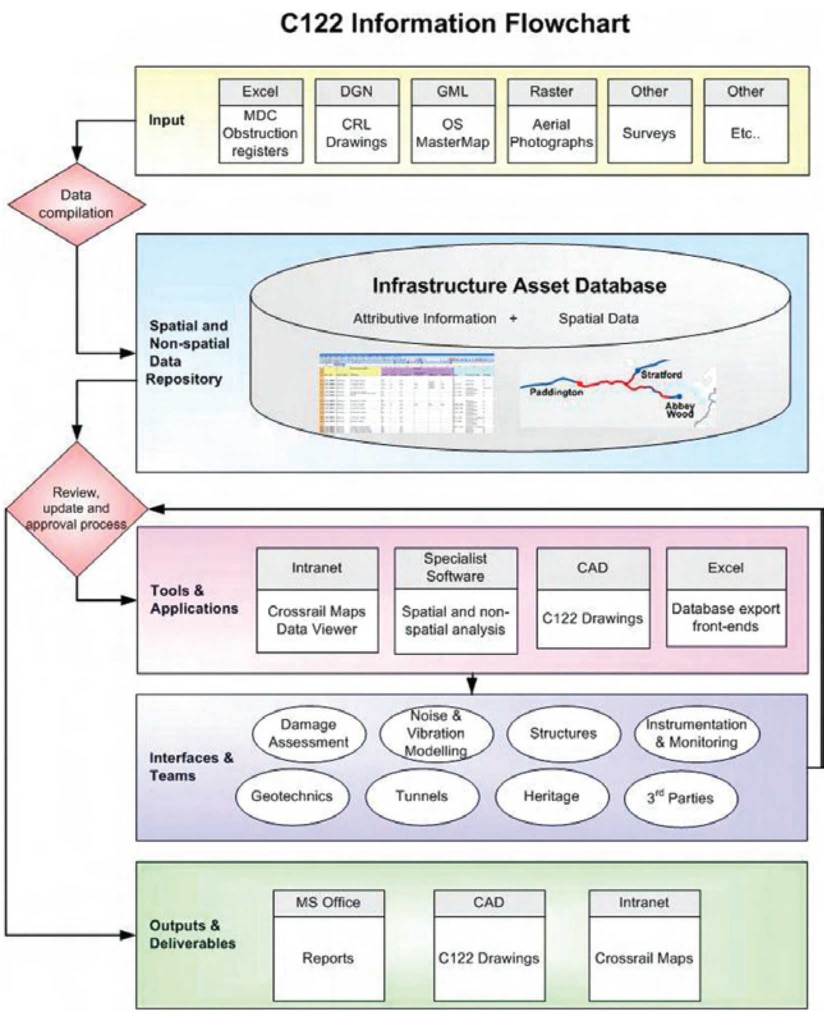


Figure 2 - Information flow chart

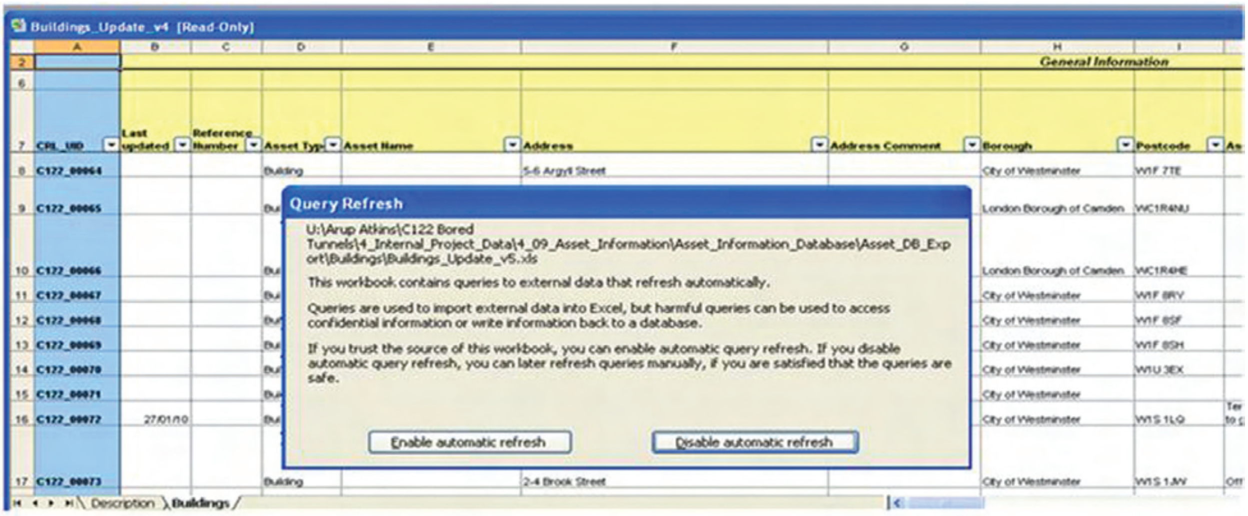


Figure 3 - Excel front end spreadsheet - automatic database update on opening

Implementation components

A centralised spatial and non-spatial data store

The project specification requires the creation of a database that holds all relevant information on different asset types along the route of the bored tunnels. These include buildings, sub-surface structures, above surface structures and utilities. It was immediately clear that it was going to be highly relevant to all disciplines on the contract to know where these assets are in relation to the planned alignment of the Crossrail running tunnels. For instance, if the only information available to the project team is that there are more than 200 piled buildings along the route, but their locations are unknown, this information is of limited value. However, if we are able to provide a location for each of the assets, along with other spatial information such as proximity to the running tunnel in both height and plan, and whether the asset is within the 10mm or the 1mm settlement contour, this information is of far more use. This meant that the database had to not only contain attributes and information, but also the precise location of each asset; it had to be a 'spatial database', managed in a GIS.

The starting point was four MS Excel spreadsheets which were inherited from various consultants from the previous scheme design phases of the Crossrail project. These spreadsheets contained a total of 5,000 records on assets which had been categorised as 'obstructions'. Unfortunately the information was inconsistent, not comprehensive and more importantly the majority of it was not geo-located; there was no spatial representation or a real world location reference.

For around the first three months the C122 GIS team, with the help of the building and structures teams, collated more information about these existing asset, and assigned spatial objects to each of them. At the same time data on new and previously missing assets was collected, resulting in a database of more than 18,500 relevant asset along the route of the Crossrail bored tunnels. By assigning a spatial object to each asset, the data is far more valuable to the GIS team and the project as a whole. Spatial searches and queries can be carried out, in combination with other non-spatial criteria provided by the engineers. The results of queries can be output as lists of assets, PDF plans or CAD drawings, tailored to what the engineers require.

The key to this principle is the centralised storage of all asset information. Attribute information such as asset owner, structure type and material is stored within an MS Access database, while spatial data is stored within an ESRI personal geodatabase. A common ID links the two elements together, while procedures and standards were setup detailing how information should be handled, edited, updated and distributed. Tools have been developed that enable the whole C122 team to access the information available, and also to facilitate the review and update of the information. These tools are all based on standard MS Office software, ensuring that there are no licence issues or other usage constraints. Spreadsheets, dubbed 'Excel front-ends', shown in Figure 3, are linked to the Access database and automatically refresh on opening, enabling the team to access the attribute data stored against each asset.

Access to spatial data via Crossrail Maps

Crossrail Maps is a web GIS, accessible through the Crossrail corporate intranet. It is accessible to everyone who works within the Crossrail IT network. Up to date spatial information from across the Crossrail project can be accessed through the web GIS, including address data, location codes, tunnel engineering design and station layouts. Additionally, Ordnance Survey data and information from third party organisations such as English Nature and the Environment Agency, along with aerial photography and street mapping data are available.

Crossrail Maps functionality includes tools that enable people to query the database through the map window. Hyperlinks provide direct access to reports, drawings or other related (historic) information on assets, including information sources.

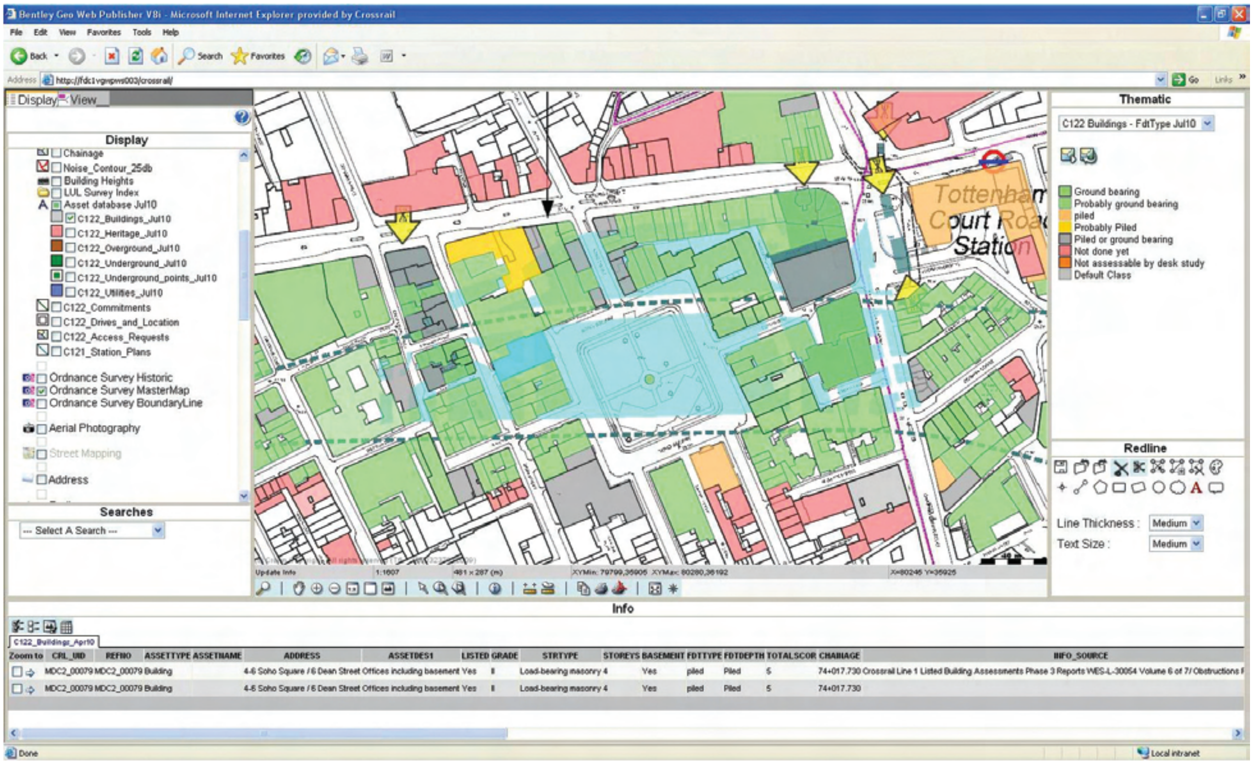


Figure 4 - Crossrail Maps web GIS

Assets themselves can be put into context with other map layers, such as historic maps, recent aerial photographs as well as Thames Water or National Grid data. Query results can be exported to Excel or other formats for further use.

Crossrail Maps is shown in Figure 4.

Reporter

Reporter is an Excel add-in application, developed by the Crossrail C122 GIS team. It contains a number of macros providing capability to process and summarise attribute data stored with the asset database. The tool enables the team to produce Word and Excel reports of information using pre-formatted templates.

In total more than 3000 damage assessment reports have to be created for structures along the bored tunnel route using the Reporter tools, for different phases of the project. To enable the engineers to easily identify which reports are required for which assets, a set of queries has been developed within the asset database. For instance, a building will require a different report to be created depending on whether it has listed status or not. To enable this identification process to be easily managed, a matrix has been set-up that shows the criteria for each report and was used to create the corresponding database queries. Figure 5 shows the complexity of the queries within the Reporter tool.

The Reporter templates were set-up in MS Word with links to the database to automatically insert text, images, maps and cross-sections into the relevant sections of the report. This data is all stored within the database and is subject to rigorous quality checking by the teams responsible before it is output and issued to the client.

C122 Reporter Tool - Report Matrix	27/07/2010			Attribute Query																										
					Foundation Type						Foundation Depth						Mitigation Type				Tunnel Drive (user selects)			Spatial Buffer				Others		
				Asset Type	Piled	Probably Piled	Piled or Ground Bearing	Ground Bearing	Probably Ground Bearing	All others	>4m	Piled	Foundation Depth	Probably >4m	Listed	Demolished	Tunnel	Shaft	Station	Portal	Drive YZG	Drive X	Drive H	Sett i 10mm	Sett i 1mm	Sett ii 10mm	Sett ii 1mm	Total DA Score	Damage Category	Total Sensitivity Score
Report Title	**Report DB Co**	**Database Query**	**Notes**																		X	X	X							
Running Tunnel: Phase 1 Generic Assess	RT_P1_GA	qryRptOutput_RT_P1_GA		Building												X	X				X	X	X	X						
Running Tunnel: Phase 2 Generic Assess	RT_P2_GA	qryRptOutput_RT_P2_GA		Building												X	X				X	X	X	X						
Running Tunnel: Phase 2 Individual Asses	RT_P2_IA	qryRptOutput_RT	Piled Buildings	Building	X	X	X									X	X				X	X	X	X						
Running Tunnel: Phase 2 Individual Asses	RT_P2_IA_GB	qryRptOutput_RT	Ground Bearing	Building				X	X		X		>4	X	No	X	X				X	X	X	X						
Running Tunnel: Phase 3.1 Individual Asse	RT_P31_IA	qryRptOutput_RT	Listed Buildings	Building											Yes	X	X				X	X	X	Yes				Grade I: all; Grade		
				Building											Yes	X	X				X	X	X	No	Yes					
Running Tunnel: Phase 3.2 individual Asse	RT_P32_IA	qryRptOutput_RT	Listed Buildings	Building											Yes	X	X				X	X	X		X			>2	>0	
Station, Shaft, Portal: Phase 2 Individual A	SSP_P2_IA	qryRptOutput_SS	Piled Buildings	Building	X	X	X									X		X	X	X	X	X	X	X						
Station, Shaft, Portal: Phase 2 Individual A	SSP_P2_IA_GE	qryRptOutput_SS	Non Piled Buildin	Building				X	X						No	X		X	X	X	X	X	X	X						
Station, Shaft, Portal: Phase 3.1 Individual	SSP_P31_IA	qryRptOutput_SS	Listed Buildings	Building											Yes	X		X	X	X	X	X	X		X					
Station, Shaft, Portal: Phase 3.2 Individual	SSP_P32_IA	qryRptOutput_SS	Listed Buildings	Building											Yes	X		X	X	X	X	X	X		X			>2	>0	
Stations Only: Phase 2 Individual Assessm	S_P2_IA	qryRptOutput_S_	Piled Buildings	Building	X	X	X									X			X		X	X	X	X						
Stations Only: Phase 2 Individual Assessm	S_P2_IA_GB	qryRptOutput_S_	Non Piled Buildin	Building				X	X						No	X			X		X	X	X	X						
Stations: Phase 3.1 Individual Assessment	S_P31_IA	qryRptOutput_S_	Listed Buildings	Building											Yes	X			X		X	X	X		X					
Stations: Phase 3.2 Individual Assessment	S_P32_IA	qryRptOutput_S_	Listed Buildings	Building											Yes	X			X		X	X	X		X			>2	>0	

Figure 5 - Reporter tool matrix

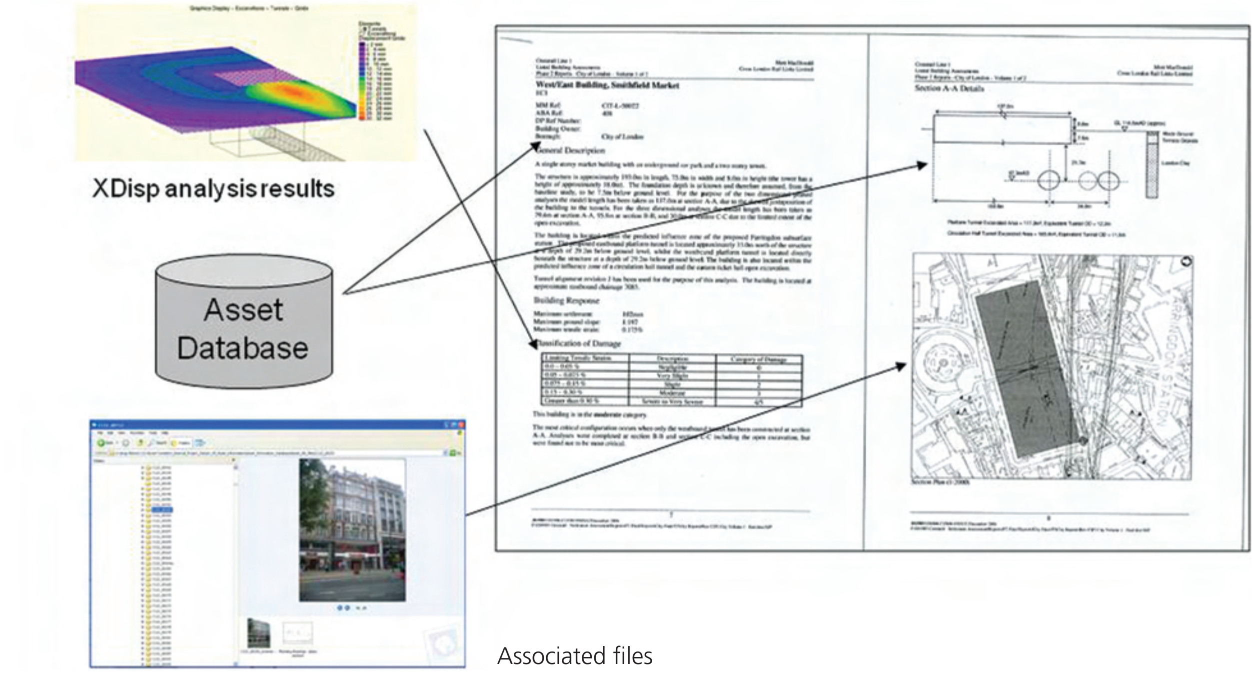


Figure 6 - Reporter information structure

As well as making the report far easier and quicker to produce, this automated method ensures consistency across all the deliverables produced. Figure 6 gives an overview of the Reporter information structure.

The Reporter tool creates two to three reports per minute fully automatically. Some of the engineers take the view that it would have been impossible to deliver the damage assessment reports at all in time, or at least not without significantly more effort, probably in the order of magnitude of hundreds of man days.

Automated update of CAD drawings

By integrating the asset database, ESRI Geodatabase and FME software, automated procedures for outputting particular spatial assets into CAD format based on set criteria have been developed. Within the Access database a CAD export tool has been developed to enable the GIS team to predefine queries for each particular CAD model. For example, a single CAD model may contain piled buildings within a 10.8m buffer of the running tunnel, or cut and cover tunnels that are within the 1mm tunnel settlement contour. The attributes required to select assets based on these criteria are stored in the database.

Once the queries have been defined and saved using the CAD export tool, the query is run which outputs an Access table containing the IDs of the required assets. It also triggers an FME batch file, which joins the table to the spatial data in a specific FME workbench, and outputs a DGN file containing the relevant spatial objects.

Once output, the DGN files are used to update the master CAD models on ProjectWise, which are used by the CAD team to create the deliverable drawings.

The process is illustrated further in Figure 7.

Multiple CAD export queries can be run at the same time. A typical set of drawings can contain around 15 separate CAD models, and where the process of selecting the correct spatial objects and outputting them to CAD used to take many hours, it can now be completed within a matter of minutes.

Data export into 3D CAD models

The CAD team on the project are creating complex 3D models of all known assets at each station location, and at specific running tunnel areas, to assist with clash detection. Using FME, spatial data from the asset database can be directly exported into a 3D CAD model as illustrated in Figure 8.

The ability to work in a 3D CAD environment is essential on this project. The integration of existing 3D CAD surveys and the information held in the asset database has brought many benefits to the design engineers on the project.

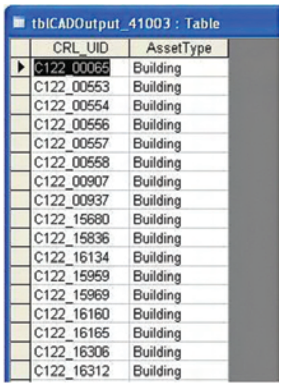
Data output auditing

Each time a CAD model is output from the asset database, a copy of the output table is stored within an audit database, allowing for the comparison between CAD outputs at two separate times. For instance, if the information stored about a building's foundations changes from 'ground bearing' to 'piled', as a result of new or better information being made available, this will affect the CAD model that the particular building is appears in, and therefore changes the resulting drawing. Using the auditing and comparison tools in the database, the GIS team can quickly check which assets have been added or removed between each revision of the drawing.

Figure 9 shows an example of a CAD model audit table. A similar audit process can be carried out on assets selected for different reports. This enables the team to determine if the number of buildings requiring a certain report has changed over time, with the addition of new information that informs the report selection criteria.

In an environment where the data and deliverable requirements are constantly changing, this ensures that the process of drawing

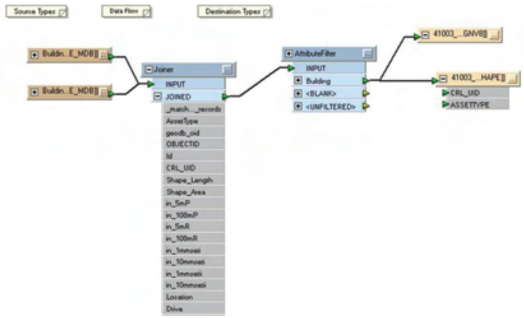
1. The CAD export query is defined and run



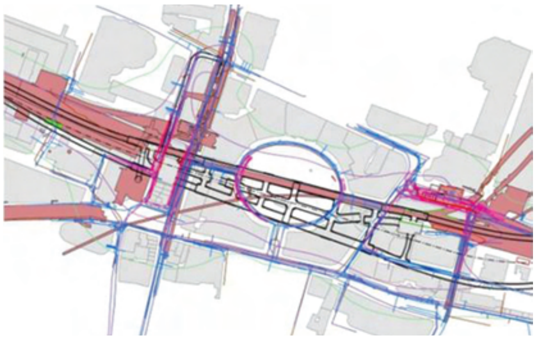
tblCADOutput_41003 : Table

CRL_UID	AssetType
C122_00065	Building
C122_00553	Building
C122_00554	Building
C122_00556	Building
C122_00557	Building
C122_00558	Building
C122_00907	Building
C122_00937	Building
C122_15680	Building
C122_15836	Building
C122_16134	Building
C122_15959	Building
C122_15969	Building
C122_16160	Building
C122_16165	Building
C122_16306	Building
C122_16312	Building

2. A table of the asset IDs required is output based on the query criteria

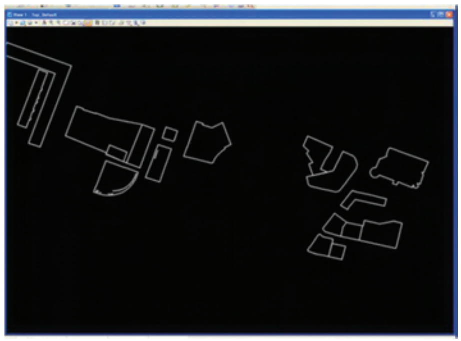


3. The Access tool triggers an FME workbench, where the asset IDs table is joined to the spatial data, which is output as a DGN

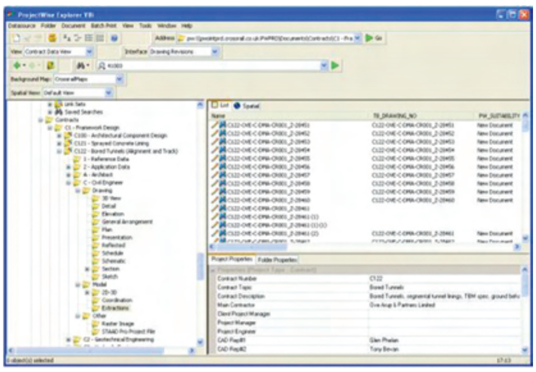


4. The spatial data is filtered from this…

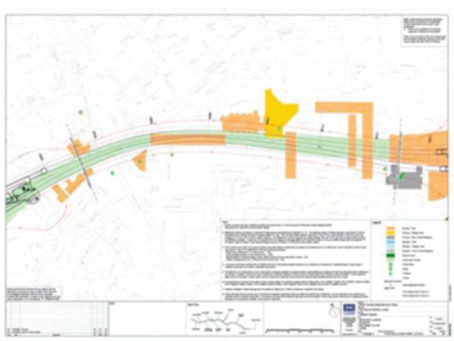
5. …to this…



6. The DGN file is used to update the relevant CAD model on ProjectWise



7. The DGN file is used to update the relevant CAD model on ProjectWise



8. The model is used along with others to create the final deliverable drawing

Figure 7 - CAD model output process

and report creation is as robust and traceable as possible.

Utilities damage assessment plans

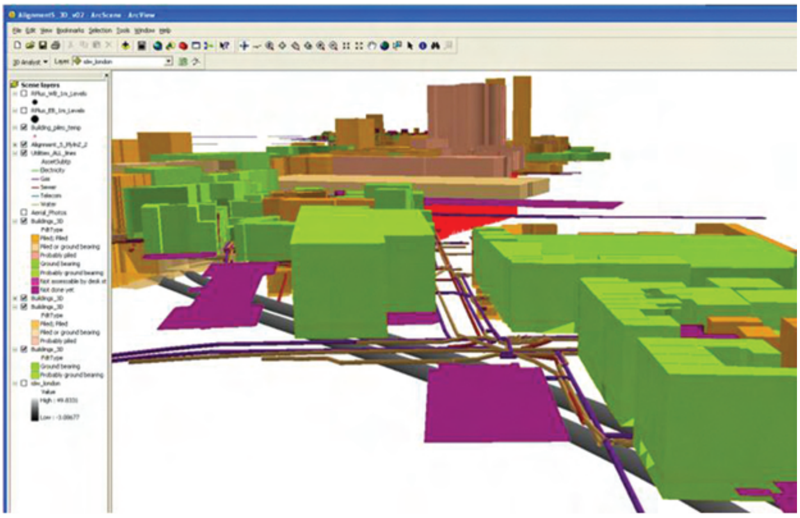
A major piece of work has been the assessment of potential damage that may be caused to utilities as a result of tunnelling. Water, sewer and gas pipes are assessed in groups by the damage assessment teams, and the results presented in reports which are issued to the client and the utilities companies. To visualise the results of these assessments, linear referencing is used to apply data to the indicative assessment lines that are used in the assessment models. This has proved to be an efficient and reliable method, especially considering that certain results and judgements change often on the basis of seeing the assessments visualised. New and revised data tables can be quickly applied to the assessment lines to see the potential effects of the tunnelling. Figure 10 gives more detail on the process.

Innovation and originality

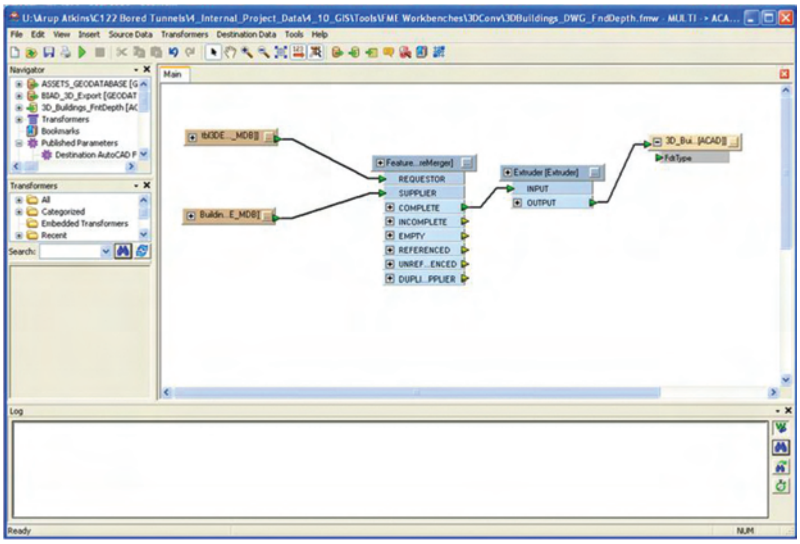
The C122 contract team set out to implement an intelligent spatial data management strategy on the bored tunnel project. This was implemented as a data centric approach, as opposed to being too reliant on specific applications and technology. Like other engineering projects, the C122 Crossrail team relies on data that is:

- Comprehensive
- Up-to-date
- Quality assured.

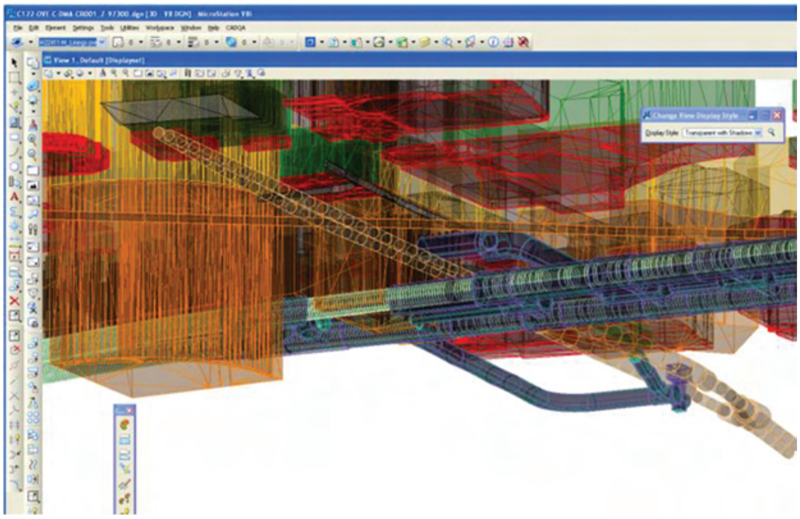
Especially on projects of this size we often experience the dilema that all data and outputs must be reviewed and approved in accordance with the project approval and quality assurance procedures before they can be issued. This process can take weeks, depending of the size of the project and the amount of data – often too late for other teams on the project. But this does not mean that the team cannot share pre-approved data internally and use it across the individual disciplines – as long as everyone understands the status of the information that they work with.



1. 3D Visualisation in GIS (ESRI ArcScene)



2. FME workbench creates 3D CAD model from spatial data and height and depth attributes held in the asset database



3. Resulting model is referenced into Microstation together with other 3D data

Figure 8 - Export into 3D CAD models

tblCompExp_41003_20100318_11_06_with_20100519_10_46 : T

CRL_UID	20100318_11_06	20100519_10_46	Comparison
MDC3_00233	-1	-1	No Change
MDC3_00274	-1	-1	No Change
MDC3_00320	-1	-1	No Change
MDC3_00216	-1	-1	No Change
MDC3_00156	-1	-1	No Change
MDC3_00088	-1	-1	No Change
MDC3_00107	-1	-1	No Change
MDC3_00092	-1	-1	No Change
MDC3_00091	-1	-1	No Change
MDC3_00140	-1	-1	No Change
MDC3_00306	-1	-1	No Change
MDC3_00235	-1	-1	No Change
MDC3_00237	-1	0	Asset Removed
MDC3_00241	-1	0	Asset Removed
C122_16793	-1	0	Asset Removed
MDC2_00104	0	-1	Asset Added
MDC3_00152	0	-1	Asset Added
MDC3_00178	0	-1	Asset Added

Figure 9 - CAD model auditing

The ability for all Crossrail employees and contractors to access the information on the Crossrail Maps web GIS was a benefit and an issue at the same time for the contract C122 team, because it meant that they could not use the tool for an internal review process for pre-approved and work in progress information.

The solution to this was that the Crossrail GIS team created a confidential area on Crossrail Maps that is only visible to the C122 team members. This is managed through Windows user account permissions and has enabled the team to upload pre-approved and work-in-progress data to share and for review internally. As the data was particularly dynamic this process meant that engineers realised the benefits of work in progress data when it mattered to them.

Figure 11 shows some of the work in progress data available to the C122 team in the confidential folder on Crossrail Maps.

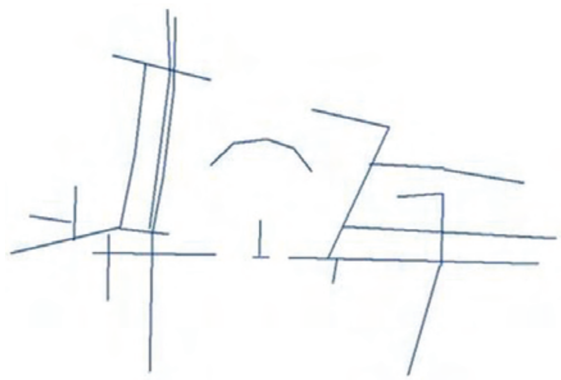
The ability to see and share pre-approved data enables engineers to work on the same information simultaneously and visualise spatial database information on a map. Due to innovations and benefits such as these, C122 staff soon became the main users of Crossrail Maps within the whole Crossrail organisation.

The same principle of user-account based permission to access data on Crossrail Maps was used for other confidential information, such as National Grid or Thames Water data.

Benefits of the approach

Cost savings achieved

The Crossrail example proves that there is a positive return on investment if GIS and information management procedures are implemented in the right way on a project. The cost-benefit diagram in Figure 12 is based on input provided

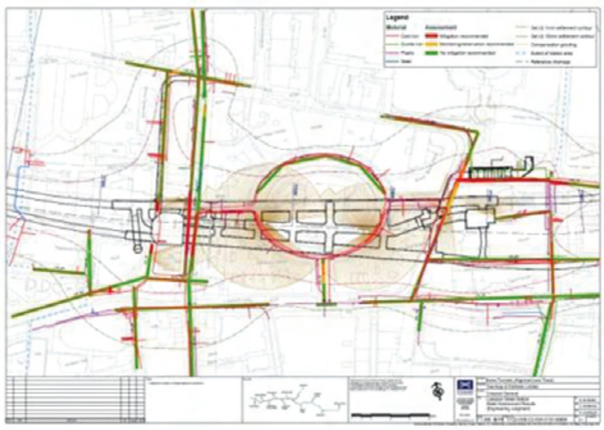


1. Lines used for utilities damage assessment converted into measured lines

	A	B	C	D	E
1	AssessLine	FromM	ToM	AssessResult	Utility
2	LIS_46	0.00	46.00	Mitigation recommended	WATER
3	LIS_46	46.00	72.00	No mitigation recommended	WATER
4	LIS_47	0.00	18.00	No mitigation recommended	WATER
5	LIS_47	18.00	28.00	Mitigation recommended	WATER
6	LIS_47	62.00	72.00	Mitigation recommended	WATER
7	LIS_47	72.00	96.00	No mitigation recommended	WATER
8	LIS_48	0.00	72.00	No mitigation recommended	WATER
9	LIS_48	72.00	130.00	Mitigation recommended	WATER
10	LIS_48	130.00	152.00	No mitigation recommended	WATER
11	LIS_49	0.00	96.30	Mitigation recommended	WATER
12	LIS_50	0.00	75.00	No mitigation recommended	WATER
13	LIS_50	75.00	82.00	Monitoring/observation recommended	WATER
14	LIS_50	82.00	131.50	No mitigation recommended	WATER
15	LIS_55	0.00	37.00	No mitigation recommended	WATER

2. Assessment results data is compiled into a database table

3. Linear referencing is used to apply the results data to the assessment routes



4. An assessment plan is output as a PDF

Figure 10 - Utility damage assessment plan process

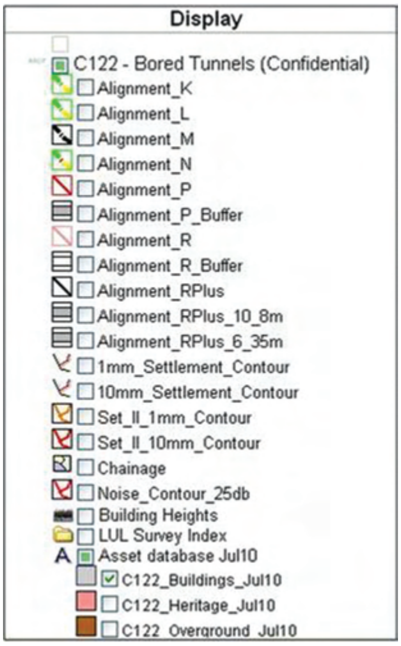


Figure 11 - Crossrail Maps C122 confidential folder

by the engineers and discipline leaders of the C122 team. As the contract is on a time basis, the client has a direct benefit from the time savings made.

The diagram shows how much time has been saved by implementing GIS and information management procedures and tools which were set-up by the C122 GIS team. There are three main areas of time savings which could be identified:

- Saved time for engineers finding information: the assumption is that without the tools 25 out of 200 staff would spend on average 3 hours per week to find relevant current or historic information (reports, drawings, or surveys, for example)
- Saved time for engineers in the creation of deliverables excluding drawings (reports, lists, mailings and databases for example). The peaks reflect deadlines, where normally the workload and the time spent would increase, but can now be saved due to more efficient tools and procedures
- Saved time for engineers but also CAD technicians for the creation of drawings. Since the drawings are created and updated in GIS there is less work for the CAD technicians. There is no manual adding or editing of assets required.

Costs for the GIS implementation and maintenance are shown as an orange line in the diagram.

The indicative return on investment diagram in Figure 13 shows the total of saved time (black line) and the GIS costs (time based, orange line) against this. The red line shows the accumulated saved time deducted by the GIS costs and reflects the return on investment. The break-even, where the investment is equal to the saved time, was reached during the second month of the contract. Accumulated saved time increases with the duration of the project.

Please note that this in an indicative diagram which excludes costs for licences or hardware.

What this clearly illustrates is that by implementing GIS together with good information management practices and procedures that can be used by the whole project team, considerable savings in time can be made. These savings translate themselves into efficient work practices which directly save project costs.

Streamlining and increased efficiency of business processes

Most GIS savvy people would testify this type of approach for making information accessible, and managing spatial data in an intelligent way, as very powerful and useful. However, the trouble that the GIS community often faces is in trying to quantify the benefits and put a value against usefulness.

Anne Kemp, Atkins' Chair for the Geospatial and Integrated Digital Solutions Technical Network, says: "It can be difficult for GIS professionals to communicate why and how their services deliver benefits to a project. To them, the importance of GIS - when a typical infrastructure project is all about understanding the geography, combining different geographical data types and creating tools to assist in making decisions – seems obvious. But this is not good enough – there have to be valid, substantive and demonstrable benefits – which will stand up to scrutiny. The Crossrail project has been a good opportunity for the project team to produce the evidence."

Ilka May, the lead GIS Consultant on this project, agrees "Implementing GIS on a project is often perceived as additional work and the first question always is: 'Will the client pay for it?' GIS people have a different view on this. We are able to prove that if GIS is properly implemented in accordance with the requirements of the project it will improve efficiency and save time and therefore money for the client."

Stepping back it is clear what the benefits of this approach are. These fit into two categories:

- Benefits to the client
- Benefits to the project team

These are detailed in Table 1.

Table 1 - Benefits of the implementation

Benefits to the client	Benefits to the project team
Better management and reporting on data and data gaps	Highly efficient information flow through central management of spatial and non-spatial data
Client systems are being used (this was stipulated in the scope)	Quick and easy access to information
Improved data assurance and quality	Improved decision making on the basis of more reliable information
Improved data interoperability with the client's systems	Visualisation of data through maps and images
No loss of data and information at handover stages between contractors, no re-work required, hence less money spent	No delay in data distribution
	Mitigated against the risk of out of date information
	Improved interoperability between specialist applications

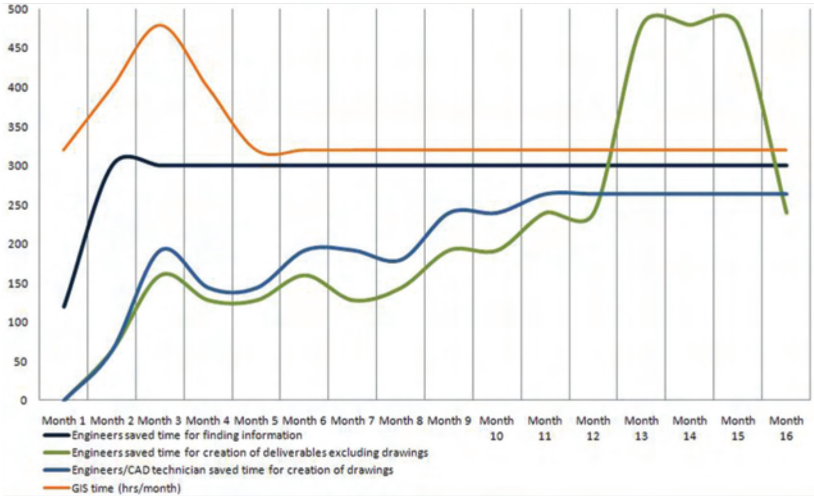


Figure 12 - Cost-benefit diagram

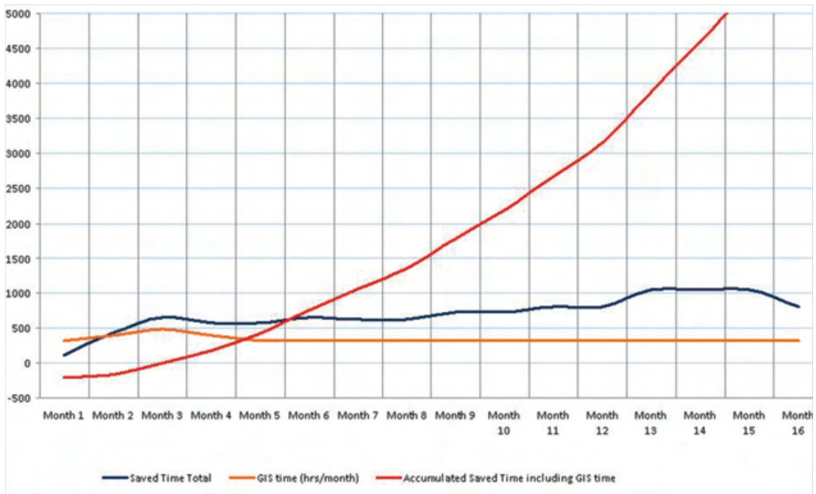


Figure 13 - Indicative return on investment on Contract C122 (hours based)

Increased accuracy in business decision making

By implementing the GIS and information management approach on the bored tunnels project, increased accuracy in business decision making has been achieved, through:

- Clarity of data and system ownership, responsibility and reliability
- Improved efficiency of workflows across all disciplines through better data management and automation
- The ability to track changes in asset information that allows quicker assessment of changed or new situations
- Ensured delivery of contract requirements.

Conclusion

As a result of implementing the approach described here, an excellent working relationship was established between the Crossrail and the C122 GIS teams, both benefiting from each other's input. C122 users across all disciplines appreciated the benefits of the GIS and information management system.

Alison Norrish, Design Coordinator for the C122 contract, says "With over four thousand buildings and countless other utilities and assets along the route of the tunnels, the assessment of ground movements and their impact is a major exercise. Our GIS team has fused a database of assets with the client's GIS mapping tool to allow the filtering, manipulation and display of information, saving countless individual engineering hours by the design team and allowing them to see the info quickly and geographically, and to plot drawings on an infinite variety of criteria. It is a great tool for engineer and client alike".

One intangible, but nevertheless important, factor is the happiness of staff and that people enjoy the work they are doing. This was significantly improved through the automation of some of the most repetitive tasks. An engineering project of this size and duration goes through many changes during its lifecycle, for instance changes in the railway alignment and amendments to station layouts. These changes often cause chain reactions on multiple disciplines and contracts. Sometimes these changes make weeks of people's work obsolete, which can be very frustrating for those involved. The frustration resulting from repetitive tasks on the project was significantly reduced through the ability to track and assess changes and to automate updates of reports and CAD drawings.

The implementation of the approach described here has been and continues to be a real success on the Crossrail bored tunnels project. It has been proven that projects of this scale and complexity can really benefit from employing GIS and information management processes and procedures, and it is hoped that this is a step towards these practices becoming the norm on large infrastructure projects.

Acknowledgements

With thanks to my Arup colleagues on the Crossrail C122 project, Ilka May and Andy Kervell, and Crossrail GIS staff Lewis Calvert, Fernando Branco and Milena Grujic.

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http://www.agi.org.uk/agi-awards/

Technical Networks - The essential framework for disseminating knowledge from projects to industry

Chris R Hendy

Head of Bridge Design and Technology

Highways & Transportation
Atkins

Phillippa Spencer

Senior Engineer

Highways & Transportation
Atkins

Madhusudhan Sundaram

Senior Engineer

Highways & Transportation
Atkins

Abstract

Within the civil engineering industry there are numerous methods of communicating knowledge. These include formal methods, such as through peer-reviewed papers, conferences and websites, and informal methods such as new teams coming together and sharing their experiences. However, knowledge management is more than just communicating lessons; it involves the creation, capture, sharing and leveraging of knowledge and without all these aspects present is unlikely to be effective. Capturing all useful knowledge in a project lifecycle and cascading the lessons learnt to industry, and indeed within organisations, presents a significant challenge to achieve consistently.

This paper presents a tried and tested solution which employs a named individual on each project, the Knowledge Capture Coordinator (KCC), in conjunction with a framework of technical, discipline-led, networks. In essence, the KCC is responsible for capturing learning on the project and feeding past learning into the start of the project, while the Technical Networks review, develop and validate the learning captured and ensure it is subsequently available and communicated to all who need to know in the most appropriate manner. This paper discusses in detail how this is achieved and elaborates on the role of the KCC and networks in driving continuous improvement. It is based on a process employed within the Highways and Transportation business of Atkins.

Introduction

Knowledge Management (KM) is the systematic process by which knowledge for an organisation to succeed is created, captured, shared and leveraged. Effective KM is essential for organisations to achieve strategic objectives such as:

- Improving performance
- Providing competitive advantage
- Encouraging innovation
- Achieving consistency and best practice across a business

Within the civil engineering industry there are various means of communicating knowledge to different sectors and individuals. Peer-reviewed papers in journals are a well established means of highlighting best practice and lessons learnt. The use of websites as a means for providing expert guidance has also become the norm over the past 10 years. Conferences and presentations led by industry committees also play a vital part in communicating knowledge and expertise across the civil engineering sector and encouraging dialogue and exchange of ideas. Whilst these are good examples of methods of distributing knowledge, they only cover the sharing of knowledge.

Creating, capturing and leveraging the knowledge itself can be much less systematic, whereas managing knowledge in a systematic manner is vital to achieve organisational strategies. Fundamental to this is capturing information "live" as it is created. This paper presents a systematic process of capturing learning on projects, passing it to relevant Technical Networks where it is reviewed and validated and feeding back the pertinent knowledge into projects and industry. It is based on a process employed within the Highways and Transportation business of Atkins.

A proposed process for capturing and reinvesting knowledge

A formal framework for knowledge management

Construction projects cover the whole cycle of development, design, construction, maintenance and eventual decommissioning. The potential learning on a typical construction project is considerable. Whilst some of this learning will transfer with the individuals working on a project, much will not unless

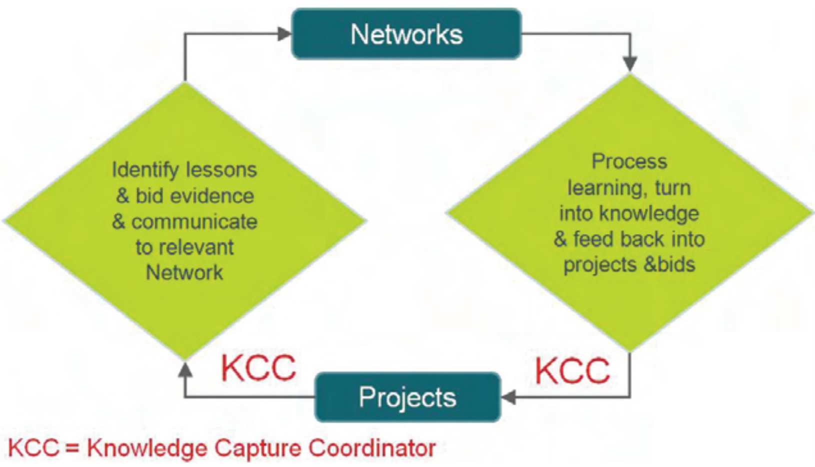


Figure 1 – Illustrative Knowledge Management process employing the KCC role and Technical Networks

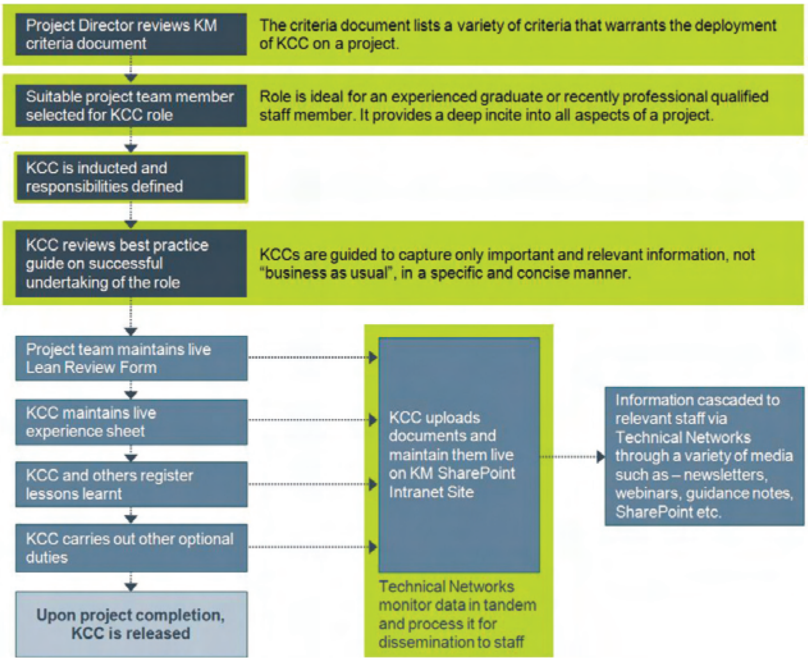


Figure 2 - Knowledge capture co-ordinator lifecycle

there is a formal process which enables reviewing and cascading relevant information for future use. One such process, involving a specific project role known as the Knowledge Capture Co-ordinator (KCC), is shown in Figure 1. This is a project role in use in Atkins. The process is dependent on the existence of a supportive community of Technical Networks available to help process and cascade learning. Networks and their characteristics are discussed in later in this paper.

The KCC role is designed to facilitate capture of data and ensure that previous learning is fed back into new projects. These are two key aspects of the role. Firstly, the KCC facilitates the collection of project learning and its communication to Technical Networks who can review, develop and validate the information and then ensure that the endorsed learning is communicated to all who need to know. Secondly, the KCC helps to ensure that all relevant previous learning is brought back into new projects, so repeat mistakes are minimised and the commercial benefit of previous learning is brought to bear. Altruistic organisations, those seeing marketing potential or those whose clients require learning on their projects to be disseminated widely, can also cascade the learning to industry as a whole once it has been captured and reviewed; some examples are noted in following sections. The process thus creates a cycle of knowledge which provides an auditable trail of captured knowledge and its re-use, together with an opportunity for a spiral of improvement, irrespective of continuity of the teams involved.

The Knowledge Capture Co-ordinator role

Overview of KCC role

The KCC role is deployed on all projects which will benefit from its implementation. Some typical, and by no means exhaustive, project characteristics likely to make the role beneficial include:

- Limited experience of project type within the local delivery team
- Input required from outside the immediate team
- Work is for a new client
- Departures from standard to be submitted
- New construction techniques, materials, design or analysis methods to be used
- Research & development involved
- Project has important marketing potential
- Not "business as usual"
- There is a specific client requirement to manage "lessons learnt"
- Innovative ideas are to be implemented.

The qualities of the individual KCC on a project are key to the success of the role so the appointment should be based on suitable locally developed criteria. Important characteristics of an effective KCC include:

- Ability to network with numerous individuals across projects and with external parties
- Assertiveness and confidence to ask appropriate questions
- Energy and enthusiasm to motivate self and others
- Desire to improve project and own performance
- Conscientiousness and thoroughness, with the determination to revisit and investigate actions to ensure they are closed out
- Ability to command respect of teams and secure their engagement
- Good communication skills and the ability to listen to others.

The KCC role has core responsibilities (common to every project) to ensure that learning is created, captured, shared and leveraged. These include:

- Capturing lessons learnt (and ensuring others do also) in a Lessons Learnt database
- Ensuring project teams complete, and keep updated, a Lean Review Form to ensure previous learning is utilised, new ideas that are successfully deployed are captured and that records of savings generated through the project are kept live and up to date
- Maintaining a live Project Experience Sheet

The core responsibilities are discussed further in the following sections. The process for appointing the KCC and the lifecycle of the role is illustrated in Figure 2.

The KCC may have other optional responsibilities as defined by the Project Manager at the induction stage. These are generally aimed at improving project communication and efficiency internally, such as producing a project newsletter and progress reports, and are not discussed further in this paper.

Figure 3 - Processing of information via typical Technical Networks

Capturing lessons learnt for review, validation and communication in a Lessons Learnt database

Capturing lessons learnt is a key aspect of continuous improvement but such lessons must be reviewed and validated by responsible parties (the Technical Networks) before they are cascaded to designers for re-use as "best practice".

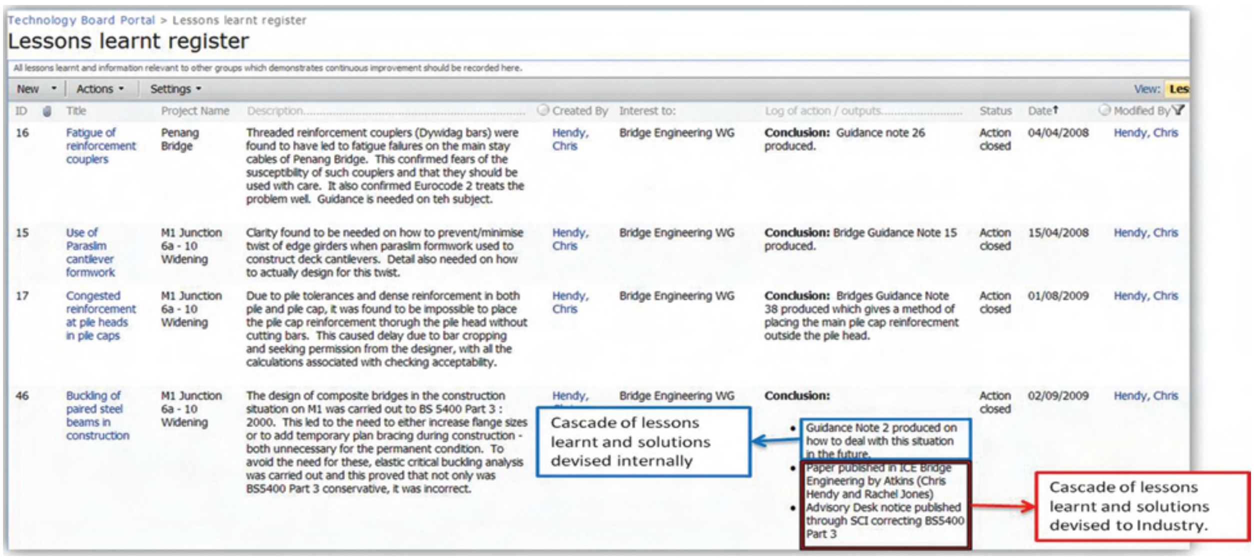
Lessons learnt may be captured in a database. In Atkins' process, the KCC is responsible for ensuring that information on problems encountered, new solutions provided and other project learning is recorded in a central 'lessons learnt' database that is visible to all staff in the organisation. On input, it is essential to identify the target audience for the lesson and the party who will carry out the validation and communication.

The system adopted in Atkins requires each lesson to be assigned to a particular Technical Network. Once the lesson has been entered, an alert is sent via RSS feeds to the relevant Technical Network's intranet site (see example site in Figure 5), requiring its action. It is then the responsibility of the Network to review the information and determine how best to use and communicate it to others. This simple process is illustrated in Figure 3, showing some typical Technical Networks in operation in the Atkins Highways and Transportation business.

Once the lesson learnt has been actioned, this is noted in the register and the continuous improvement cycle is closed. Figure 4 shows an example of a lesson learnt in the database which has been registered and subsequently closed out.

Although it is visible to all for information, the database is not the primary means of facilitating re-use of learning. The database is a tracking device, allowing monitoring of the process whereby lessons are captured, reviewed, processed and then communicated to staff. Until an action is concluded, the data does not constitute a verified piece of knowledge for re-use as it has not been endorsed by the relevant Network. Typical input to the database includes:

- Problems encountered (with or without a good solution being offered)
- New ideas used (with or without an indication of the saving generated)



Technology Board Portal > Lessons learnt register

Lessons learnt register

All lessons learnt and information relevant to other groups which demonstrates continuous improvement should be recorded here.

New | Actions | Settings | View: Les

ID	Title	Project Name	Description	Created By	Interest to:	Log of action / outputs	Status	Date	Modified By
16	Fatigue of reinforcement couplers	Penang Bridge	Threaded reinforcement couplers (Dywidag bars) were found to have led to fatigue failures on the main stay cables of Penang Bridge. This confirmed fears of the susceptibility of such couplers and that they should be used with care. It also confirmed Eurocode 2 treats the problem well. Guidance is needed on teh subject.	Hendy, Chris	Bridge Engineering WG	**Conclusion:** Guidance note 26 produced.	Action closed	04/04/2008	Hendy, Chris
15	Use of Paraslim cantilever formwork	M1 Junction 6a - 10 Widening	Clarity found to be needed on how to prevent/minimise twist of edge girders when paraslim formwork used to construct deck cantilevers. Detail also needed on how to actually design for this twist.	Hendy, Chris	Bridge Engineering WG	**Conclusion:** Bridge Guidance Note 15 produced.	Action closed	15/04/2008	Hendy, Chris
17	Congested reinforcement at pile heads in pile caps	M1 Junction 6a - 10 Widening	Due to pile tolerances and dense reinforcement in both pile and pile cap, it was found to be impossible to place the pile cap reinforcement thorugh the pile head without cutting bars. This caused delay due to bar cropping and seeking permission from the designer, with all the calculations associated with checking acceptability.	Hendy, Chris	Bridge Engineering WG	**Conclusion:** Bridges Guidance Note 38 produced which gives a method of placing the main pile cap reinforecment outside the pile head.	Action closed	01/08/2009	Hendy, Chris
46	Buckling of paired steel beams in construction	M1 Junction 6a - 10 Widening	The design of composite bridges in the construction situation on M1 was carried out to BS 5400 Part 3 : 2000. This led to the need to either increase flange sizes or to add temporary plan bracing during construction - both unnecessary for the permanent condition. To avoid the need for these, elastic critical buckling analysis was carried out and this proved that not only was BS5400 Part 3 conservative, it was incorrect.	Hendy, Chris	Bridge Engineering WG	**Conclusion:** • Guidance Note 2 produced on how to deal with this situation in the future. • Paper published in ICE Bridge Engineering by Atkins (Chris Hendy and Rachel Jones) • Advisory Desk notice published through SCI correcting BS5400 Part 3	Action closed	02/09/2009	Hendy, Chris

Figure 4 – Typical lessons learnt register tracking actions to conclusion

Typical output from the processed lesson includes:

- Guidance notes on avoiding/addressing particular problems or implementing new solutions
- New design tools developed
- Updated strategy to approaching markets
- Other communications to pass on learning.

Ensuring re-use of learning and capturing evidence of savings created using Lean Review Form

The call to deliver "more for less" challenges teams to introduce the learning from previous projects into new ones to eliminate waste and create efficiencies through improved solutions. This is the basis for 'Lean' design and hence tracking Lean behaviours is encouraged through the use of a Lean Review Form which is completed by each technical discipline involved in a project.

The Lean Review Form ensures that project teams address the key aspects of Lean delivery. It prompts consideration and gathers evidence on projects of:

- Getting it "right first time" through re-use of learning
- Continuous improvement
- Cost savings produced
- Health and safety design improvements
- Improved cost estimating

The KCC works with all project disciplines involved to ensure they capture savings achieved, new ideas that have been implemented (to be entered in the Lessons Learnt database) and previous learning which has successfully been reapplied. Where there are multiple disciplines working on the project, there will be multiple Lean Review Forms in operation. Consistent with other mandatory responsibilities, the Lean Review Form is also maintained live during the course of the project.

Capturing a record of experience

In order to demonstrate capability and experience to potential clients, the key points of a company's delivery on each project need to be documented. It is often observed that organisations attempt to compile this record sometime after the project has finished when key team members with the requisite information have moved on to other projects. Clearly this is inefficient, with the potential to be inaccurate also. The KCC is thus also responsible for producing and maintaining the project's experience sheet as a live document so that, in draft, it is available to all throughout the project and contains accurate and current information.

Technical Networks and their importance in Knowledge Management

Key features of Technical Networks

The implementation of the KCC role and process described previously is entirely dependent on the presence of effective Technical Networks for each technical discipline who are able and, above all, motivated to process and cascade learning to those who need to know. Getting the right representation on these networks is critical. The right representatives will be:

- Enthusiastic for the stated aims of the network
- Respected and influential in their local team
- Keen to share skills and experience; both to ask and to give

The wrong individual will "take" from the network and neither "give" to the community as a whole nor cascade information to their local team. Such behaviours can severely hamper the functioning of a network.

Technical Networks should have a shared action plan which includes processing new learning on projects. It is essential that networks have a full geographical coverage to ensure universal buy-in and there should be regular communication that reaches the right people every time. An important facility for a network to have is a means of asking questions of the community as a whole and it is vital that queries raised always reach those who can help (even if they choose not to); an email distribution group is crude but effective in this respect while postings on intranet sites may not always be seen by the key people in time.

Whilst it is increasingly desirable to minimise travel costs and the associated carbon footprint, it is beneficial for members of networks to meet in person from time to time; twice a year might be a reasonable target if the members of the network are well distributed. Face to face contact engenders a culture of trust between teams and is a very effective means of exchanging ideas, understanding capabilities and facilitating sharing of resources. The informal networking element is very important in cementing personal relationships and trust in the networks and it often pays to have a social event, such as a dinner, the night before a meeting of the Network.

Networks will only function well if there is an established team spirit and pride in supporting network objectives and this takes time to develop because there are many issues of trust to be overcome and developed. It goes without saying that the leader of the network is a critical position in making this work. Without a suitable leader to engender feelings of trust and community, the network will not perform to its potential. Establishing shared objectives is critical in this respect. There will be many reasons to collaborate across a network including:

- Sharing marketing material and ideas to improve profile
- Sharing training material and activities
- Providing a consistent and clear technical career path that values team working
- Sharing guidance notes which drive consistency and best practice, whilst eliminating mistakes
- Sharing workload and resources
- Encouraging innovation
- Providing guidance and direction on complex subjects such as sustainability and climate change
- Networking and socialising (the benefits of both of which should not be under-estimated)

Team spirit and personal commitment will be further enhanced by:

- Celebrating success and the creation of new ideas through holding awards
- Publicly acknowledging the contributions of those who share their learning (by email, postings

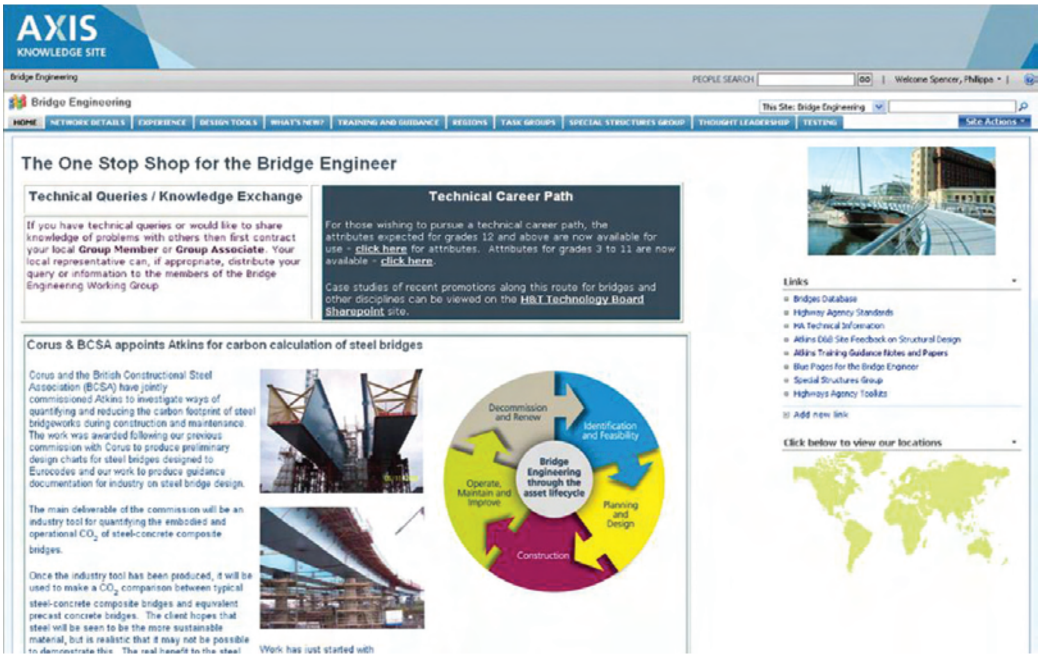


Figure 5 - Typical Knowledge Site

on intranet sites or other means)

- Valuing sharing of knowledge in career progression
- Celebrating re-use of learning from previous projects on new projects
- Ensuring all can ask questions without fear of ridicule; this is a fundamental issue of trust which the Network Leader must help establish.

Absolutely key to all of this is to recognise that such communities, and the implicit trust within them, take time to establish.

The use of intranet knowledge sites is hugely effective in aiding collaborative working and making the resulting knowledge readily available to all to use. Figure 5 shows a typical knowledge site. Such sites typically have two generic types of area: one presents the “finished product” of information for use by all staff, while the other is a “work in progress” area which is invisible to all but the relevant working groups preparing the information. The design of such sites is, however, the topic for a paper in itself and is not covered further here.

Role of Technical Networks in Knowledge Management

The section above discussed what ingredients make a cohesive Technical Network that will work together for the good of the whole organisation rather than in a series of silos. However, without a formal means of capturing information on projects, the knowledge disseminated by Networks will be, at best, ad hoc and will not fully capitalise on all of the potential learning within the organisation. This is not to say that sharing such ad hoc information is undesirable; it is just likely to be incomplete and lacking in focus on what the businesses strategy demands.

One process to gather information in a systematic way is described previously through the KCC role and KM process. As discussed, the Technical Networks are fundamental to the success of this process. This is because they perform two key functions. Firstly, they must review, validate and process the information passed to them to ensure that it is correct and appropriate to circulate to the rest of the organisation as “best practice”. The information captured may be in many forms. It may be a new idea that worked well on a project and it is being proposed to implement it elsewhere; it may be an idea that didn’t work well with a recommendation that it be avoided in future; it may just be something unexpected which happened and the reasons for this may or may not be understood by the project team. Whatever the information, it needs to be carefully scrutinised by those best placed to process the data and generate conclusions for use by the rest of the business. These conclusions need to carry the full weight and authority of the network so they can be used with confidence. Secondly, they must find an appropriate means of cascading the knowledge to other engineers. The information is useless unless it reaches those that need to know. There are a variety of ways to do this and each Network needs to find the techniques that work best for them. Usually it is best not to rely on one single method of communication (such as placing a note on an Intranet site) but to use a variety of media. The more important the lesson, the more means of communication are likely to be employed to ensure that the target audience is reached.

To review, validate and process the information, engineers with the right expertise need to be found to carry out the work. The Technical Network must be structured in such a way that a dedicated part of it (e.g. a working group within it) does an initial review of the information and then selects the most appropriate person or persons to process it further. This will include assigning one or more reviewers for the output so that the guidance produced can be deemed to carry the full endorsement of the Network. Often the individuals with the right expertise will be well known, but this will not always be the case for niche specialisms or large Networks. Managing skills and locating the right expertise is another key function of a Technical Network. This entails identifying the skills necessary for the business's chosen markets, identifying any deficiencies and putting in place plans to fill the skills gaps. Once the desired skills have been established, the Network needs to be able to define and assess competency levels for those skills and locate individuals with the desired competencies on demand. The personal development plans for individuals will then be strongly influenced by these skill requirements.

Skill identification and management within a Technical Network

A key function of a Technical Network is its ability to manage its skills. One key feature of this function is being able to locate staff with the right skill at the right competency level; the right person could be in any geographical location, particularly in a large organisation.

There are several challenges in establishing a competency framework and skills search facility. Firstly there is the challenge of identifying the list of skills in the first place. For technical disciplines with multiple markets, the core skills needed may differ from client to client and region to region. If the skills list is too long, it will be cumbersome to use, complete, update and validate. There will also end up being overlap between the skills whereby a competency in one will automatically bring competency in another. If the list is too short, the skills will become too generic and of little use. In Atkins, the skills list is set by the relevant Technical Network in a democratic way and is updated at defined periods so that the list of skills reflects the markets in which work is executed.

Secondly, the number of competency levels to be used needs to be agreed. In determining these, there needs to be clarity as to what the competency levels will be used for. If there are too many levels, there will be too much debate over which level has been attained. If there are too few, it will not be possible to distinguish the real experts from those with a good working knowledge, which may be a problem for technical consultancy projects but may not for routine design projects. The authors' experience is that 5 levels of competency is usually sufficient, but some organisations advocate more depending on the time of discipline.

Thirdly, there is the challenge of ensuring all staff have an up to date record of their competency level against each of the specified skills. Attempts to produce a central skills matrix usually fail, as do attempts to complete a matrix office by office and then combine them, because it places the emphasis on the organisation to maintain the large volume of data up to date. One simple approach is to set out the skills and competency levels on a standard template (for example, as in Figure 6), and make

BRIDGE ENGINEERING WORKING GROUP SKILL MATRIX			
Name			
Staff number			
Business Unit			
Location			
Grade			
1 = Awareness of area 2 = A working knowledge 3 = Competent 4 = Able to monitor others 5 = Expert	Measure: Knows where to look of who to ask Works under direction Works with little or no direction Gives direction to others in Atkins Industry Leader	SKILL LEVEL	DESIRED SKILL LEVEL
		1 to 5	1 to 5
Arched Structures			
AASHTO - Concrete			
AASHTO - Steel			
Bearings			
Bearing replacement			
Cable supported structures			
CAD - structures			
CCTV masts and lighting Columns			
Concrete Deterioration			
Concrete mix design and admixtures			
Concrete Repairs			
Confined space			
Contract - Specification			
Contract - Spec Concrete 1700 Series / Mix Design			
Contract - Specification Steel 1800 Series			
Contract - BoQ			
Cost - Construction			
Culverts			

Figure 6 - Extract from example skills template

Clear Search

Run Current Search   Change Search logic

Concrete Deterioration		4	or greater
OR			
	is		or greater
OR			
	is		or greater
OR			
	is		or greater
OR			
	is		or greater

Allow staff from the following Offices	
Abu Dhabi	Yes
Bangalore	Yes
Belfast	Yes
Birmingham	Yes
Bristol	Yes
Cambridge	Yes
Cardiff	Yes
Chelmsford	Yes
Conwy	Yes
Cork	Yes
Croydon	Yes
Denmark	Yes
Derby	Yes
Dubai	Yes
Dublin	Yes
Edinburgh	Yes
Epsom	Yes
Exeter	Yes
Glasgow	Yes

RESULTS

Staff Number	Staff Name	Business Unit	Location	Grade
31326	Alan Kerr	WO0060 Capital delivery solutions, Water Ops, W&E	Belfast	14
16005	Alan Mordey	CQ0020_STRUCTURES DESIGN & CONSULTANCY	Birmingham	15
121023	Bijan Hassanzadeh	AI0020	Chelmsford	13
46759	Bogdan Tutoveanu	TU0030_COST REIMBURSEABLES	Ipswich	12
31083	Chris Hendy	AR0001_TE EPSOM OFFICE	Epsom	16
50856	Chris Monks	CQ0020_STRUCTURES DESIGN & CONSULTANCY	Birmingham	12
130591	Chris West	CQ0020_STRUCTURES DESIGN & CONSULTANCY	Birmingham	14
46006	David Rowlands	CN0020_WALES	Cardiff	13
33691	Dermot McAdam	(no text)	Belfast	14
139392	Gayathri Tella	RY0010	Glasgow	10
35702	Graham Stevens	AI0020	Cambridge	13
122981	Homayoon Sadeghi Pouya	CQ0020_STRUCTURES DESIGN & CONSULTANCY	Birmingham	12
45912	James Henderson	CN0020_WALES	Swansea	13
107727	Jim Kilcar	CP0020_TSOL NORTH EAST	Stockton	14
30350	John Deegan	CM0020_STAFFORD	Birmingham	13
48288	Niall McKay	CP0020_TSOL NORTH EAST	Stockton	13
141908	RICHARD J BARTON	CQ0020_STRUCTURES DESIGN & CONSULTANCY	Birmingham	15
49794	Ronnie Tosh	RY 0010	Glasgow	14
41352	Steve Salter	CQ0020_STRUCTURES DESIGN & CONSULTANCY	Birmingham	12
45089	Trevor Madden	CQ0020_STRUCTURES DESIGN & CONSULTANCY	Birmingham	11

Figure 7 – Example output from the Skills Trawler

it the responsibility of individual staff to keep it up to date, like a CV, and posted on a prescribed area of the intranet system. The skills template then needs to be integral to the Personal Development Review (PDR) process at which the assessment levels are ratified and development actions are formulated around the business need and personal desire for individuals to up-skill in certain areas. The level of competency should also link into the career path guidelines in place in the organisation to inform career progression and promotion. The Technical Network should also take an overview of the self-assessment scores to maintain consistency across the business.

Lastly, once the competency assessment for all staff is complete, it is necessary to develop a search facility so the right staff for the right project or assignment can be located. There are many solutions for this from putting the individual staff competency assessments within a database which they amend on-line, to simpler solutions where individual templates in Microsoft Excel are uploaded to a defined area on an intranet site for searching. One simple solution in use in Atkins following the latter approach uses a Skills Trawler, essentially a Microsoft Excel file containing macros, which reads all the skills templates that have been uploaded by individuals associated to that network and collates the information into one skills matrix. It is then possible to search for skills based on skill level, office location, staff grade etc. For example, if advice is sought on "concrete deterioration", it would be appropriate to search for someone with a level "4 or greater" competency in that skill as shown in Figure 7.

Tangible benefits derived from the KCC role

As a result of the KCC role and its integration with Technical Networks a number of tangible benefits have been seen on Atkins' transportation projects:

- The capture and processing of lessons learnt enables best practice guidance to be produced which, when followed, generates savings on subsequent projects, as recorded in the Lean Review Form. It also provides a means of highlighting issues to the attention of industry. For example, as a result of the lessons learnt capture process, the following examples of guidance were written and cascaded to industry through journals and publications by the Bridge Engineering Network:
  - SCI Advisory Desk note AD 330, "Open top box girders for bridges", New Steel Construction, February 2009. This was produced following some unexpected problems with twisting of box girders during construction on a project.
  - "Lateral buckling of plate girders with flexible restraints", ICE Bridge Engineering, March 2009. This was produced following identification on a project of a more economic design methodology for paired steel beams during construction than proposed in the design code and this led further to an update of the codified design rules.
  - Recommendations for the production of assessment versions of the Eurocodes, to be published ICE Bridge Engineering March 2011. This was produced following use of Eurocode 2 on a project for assessment and strengthening of a viaduct leading to very much reduced costs. It was identified that, with care and experience, the new Eurocodes could be used in assessment of certain existing structures leading to reduced costs, so the paper was written setting out the basis for this.
  - Design of the Olympic Park Bridges H01 and L01, to be published IABSE Structural Engineering International. This was produced following identification of potential dangers in the misinterpretation and misapplication of a particular Eurocode clause for the design of arches.
- The Lean Review Forms allow not only savings obtained on a project from the prompted re-use of previous learning to be identified, but it allows savings generated across an entire portfolio to be identified. The savings can be split between those which the organisation itself benefits from and those passed to the client. It also identifies the savings produced from new ideas implemented which may themselves be re-used once they have been processed by the Technical Networks and made available as best practice guidance to staff. The re-use of one of the lessons mentioned above alone saved an estimated £5M in strengthening costs on a subsequent project.

Conclusions

Knowledge Management is fundamental to continuous improvement and improved project delivery. The KCC role described in this paper is designed to ensure that the learning process is accelerated. In essence, the KCC is responsible for capturing learning on the project and feeding past learning into the start of the project, while the Technical Networks review, develop and validate the learning captured and ensure it is subsequently available and communicated to all who need to know in the most appropriate manner. Selecting the right person for the KCC role and providing clear guidance for the role is critical to its success, as is the presence of cohesive Networks to take ownership of the KM process within their discipline.

Once the process is running effectively, the following benefits are likely to be observed and are themselves a measure of the success of its implementation:

- Reduced numbers of mistakes
- Improved efficiency in the design process
- Production of leaner designs
- Stimulation and encouragement of innovation
- Increased cross-application of technologies through the interaction of Technical Networks
- Captured evidence of how learning was captured and fed back in to other projects
- Captured evidence of savings produced.

References

1. Dent RJ and Montague KN (2004) Benchmarking knowledge management practice in construction, CIRIA C620, London, UK.
2. Fahey L and Prusak L (1998) The Eleven Deadliest Sins of Knowledge Management, California Management Review, Vol 40(3) 265-276
3. McInerney C (2002) Knowledge Management and the Dynamic Nature of Knowledge Journal of the American Society for Information Science and Technology, 53(12):1009–1018
4. Nonaka I and Krogh G (2009) Perspective Tacit Knowledge and Knowledge Conversion: Controversy and Advancement in Organizational Knowledge Creation Theory, Organization Science, Vol. 20(3) 635-652
5. Tan HC, Carrillo PM, Anumba CJ et al. (2010) Development of a Methodology for Live Capture and Reuse of Project Knowledge in Construction Journal of Management and Engineering Vol. 23(1) 18-26

Pedestrian comfort guidance for London

Gillian Iversen
Senior Consultant
Highways & Transportation
Atkins

Abstract

This paper presents the research and development of an assessment tool to evaluate the real impact of street furniture and available footway on pedestrians. Guidance for understanding the implications of different preferences for space in different area types is also discussed.

Introduction

Footway space and, in particular, the width of footways, represents the infrastructure that supports walking as a mode of transport, and is an important factor in encouraging or hindering walking.

There has been increasing recognition of the value of walking for economic development, public health and to reduce transport emissions. This can be seen in the UK through initiatives such as "smarter choices" which encourages people to walk, cycle and make more use of public transport in order to unlock public health benefits and reduce transport emissions. However, the existing tool set for assessing pedestrian behaviour and providing evidence on pedestrian needs is not widely used, in part because of a lack of clear methodology and guidance. As a result, the needs of pedestrians are often sidelined when streets and places are being designed, as there are limited ways to quantify and assess the needs of pedestrians.

Transport for London (TfL) is the main agency for transport services in London, and reports directly to the Mayor. The role of TfL is to implement the Mayor's Transport Strategy, being the Mayor's Plan for London's transport for the next twenty years. Following on from the objectives set out in the Walking Plan for London, TfL aim to encourage walking through the planning and design of the pedestrian environment. TfL therefore identified a need for a clear and consistent method to assess pedestrian activity to inform the provision of pedestrian infrastructure and commissioned Atkins to undertake a research project, culminating in the easy to use Pedestrian Comfort Guidance for London document.

This paper will examine:

- The value of walking
- The existing pedestrian analysis toolkit used when planning streets and space
- How the Pedestrian Comfort Guidance for London improves the existing pedestrian analysis toolkit
- The research supporting the Pedestrian Comfort Guidance for London is reviewed
- Further work to develop the pedestrian toolkit further is also presented.

The value of walking

Encouraging walking can unlock a number of benefits for both places and individuals. In the UK, a number of national government bodies are funding research and practical projects to encourage walking for the following reasons:

- **Walking brings health benefits to individuals.** The National Health Service in the UK is funding a range of projects to increase individuals' health including prescribing exercise and working with local authorities to increase walking in schemes such as "Go London".
- **Walking can provide an alternative means of transport** for short journeys, thereby reducing the demand for private vehicles or public transport. This is one of the key drivers behind TfL's initiatives to increase walking in the capital.
- **Community benefits:** An area with high pedestrian flow through it will usually feel safer and more vibrant than an area with low pedestrian flow.
- **Walking brings economic benefits:** for example, Colin Buchanan's study for Commission for Architecture and the Built Environment (CABE) "Paved with Gold" found that "the quality of a high street can add at least 5 per cent to the price of homes and to the level of retail rents". Likewise, TfL has been developing ways to quantify the benefits of pedestrian improvements to develop business cases for schemes throughout the capital.

The economic benefit of walking is perhaps the most tangible benefit, and is often the motivation behind transport improvement schemes.

As a recent example, the Oxford Circus Diagonal Crossing project in Central London, see Figure 1, was motivated by the financial concerns of stakeholders in the area. Oxford Circus is at the heart of the West End and one of the most renowned junctions in the world, marking the convergence of London's two most famous retail streets, Oxford Street and Regent Street. However, a combination of high volumes of pedestrians, together with the constrained nature of the space and the prevalence of street clutter and stationary pedestrians meant high levels of pedestrian congestion were a normal part of the Oxford Circus pedestrian experience for large parts of the day. This was starting to deter both retailers from opening new stores in the area, and shoppers from visiting.

Figure 1 - Oxford Circus diagonal crossing, London.

Understanding the needs of pedestrians in the area, and quantifying the benefit of redesigning the crossing was thus a key part of the design development and business plan for the site.

The existing pedestrian analysis toolkit

As encouraging walking is increasingly important in the development of designs for streets and places, it is useful to determine what tools are available for understanding pedestrians' needs. A typical process for designing a street or place is summarised in Figure 2.

The clear focus for this study was the first two stages highlighted in red: understanding behaviours and providing evidence to set priorities. As the titles suggest, these data and evidence gathering stages help to set the priorities for a design. The importance of being able to measure and quantify the needs of all modes is therefore clearly evident as part of developing an overall balanced scheme.

For vehicles, the methods for understanding behaviours, and using these to set priorities, are well defined through national and local standards (such as Department for Transport guidance reported through the Design Manual for Roads and Bridges or WebTAG). However, measuring and quantifying pedestrian activity on streets and public spaces does not have similarly well defined standards and methodologies.

This section summarises the existing pedestrian toolkit available for understanding behaviours, gathering evidence and setting priorities. The subsequent tools for quantifying pedestrian benefits as part of business case development, and methods for testing and informing detailed design, are the focus of separate research and practice.

Understand behaviours

The first stage of a design process is to understand the behaviour of users in the area. For pedestrians, the important questions are:

- How many people are in the area?
- Where are they travelling to and from?
- How are they conducting their journey – do they take shortcuts, do they cross the road safety etc?
- What do they think of the area?

Methods for measuring pedestrian activity are similar to vehicles – counting the number of users, measuring speed and understanding origin and destination points. However, as there are no clear standards for data collection, there will inevitably be inconsistencies between studies of the overall data needs, the required quality (e.g. sample size) and so on.

Evidence and set priorities

Once pedestrian activity data has been collected, it needs to be put into context. This is to allow the design team to set priorities. For example, if an area has a high number of pedestrians, and low traffic flow, the design solution will be different from an area that has both high pedestrian and high traffic flow.

Basic diagrams or maps of how pedestrian activity varies over the site provide a useful overview of the data, and allow some comparison with vehicles in the area.

The primary method of quantifying pedestrian flow is to measure capacity using a Level of Service assessment. This involves calculating the level of crowding of the footway in pedestrians per metre of clear footway width per minute (ppmm). The calculation requires pedestrian activity data and the clear footway width, accounting for any barriers, street furniture (e.g. bus shelters) or blockages (such as guard railing) that permanently reduce the space available for walking.

This result can then be categorised by a Level of Service (LoS) metric (Fruin or Highway Capacity Manual Platoon)

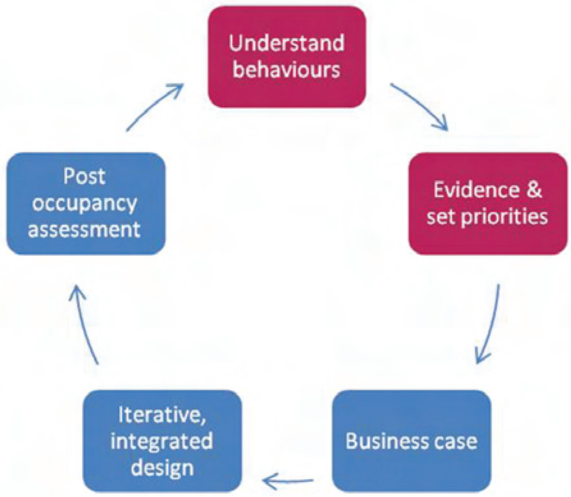


Figure 2 - Process for designing streets or places

TRANSPORT PLANNING

which classifies crowding (pedestrians per metre of clear footway width per minute - ppmm) on a sliding scale from A (very comfortable) through to F (low comfort). This can allow a measurable priority for the design to be set, such as maintaining or improving pedestrian comfort.

However, although these methods are a good basis for assessing pedestrian capacity there are a number of question marks over their use, namely:

- Which method (Fruin or Platoon) should be used and when?
- What activity level should be assessed? Maximum or average?
- How many locations should be assessed? How should these be measured?
- Which category level is appropriate in different situations? This may also vary depending on geographical location where, for example, a category which may be considered satisfactory in London, may be seen as too busy in other less dense cities.

The final point is particularly important, whereby decisions of which category (B,C etc) is acceptable are being made on a project by project basis. This is unsatisfactory, especially in London where the scale of pedestrian activity and diversity makes designing pedestrian environments a unique and challenging task.

Pedestrian comfort guidance and the existing pedestrian analysis toolkit

Launched in early 2010, the Pedestrian Comfort Assessment, described in the Pedestrian Comfort Guidance for London (PCGL) document, is being used by TfL and its partners as a consistent assessment toolkit for pedestrian planning. Already, the guidance has been adopted across a number of schemes such as individual Crossrail stations, Barclays Cycle Hire locations, and major schemes such as the redesign of the Piccadilly Circus area of London.

The primary objective of the PCGL document is to assist those responsible for planning streets to create high quality pedestrian environments through a clear, consistent process during the planning and implementation of transport improvement projects. The guidance is designed to be easy to use: at 20 pages long, it is a short document, with an accompanying spreadsheet to record data and generate results.

The document fills a number of gaps identified above, being the understanding of behaviours, alongside evidence and setting priorities. The guidance has been developed to support the design process through encouragement of planners to undertake quantitative pedestrian assessments, confident that the results are supported by a robust evidence base.

The PCGL document is detailed below, highlighting the key benefits of the guidance.

Understand behaviours

The aim of this section of the Pedestrian Comfort Guidance for London is to standardise the process of data collection and encourage use of the guidance on projects of all sizes. However, it is recommended that on more complex projects, the data collection methodology is agreed at the outset of the project to ensure that it meets the projects needs.

The guidance supports design teams seeking to understand pedestrian behaviours by:

- **Defining a basic requirement and method for data collection, defined by area type**. This provides guidance for people planning a project. However, the needs of the specific project should always be discussed with stakeholders in case more data is required than the basic level.
- **Providing a standardised approach for the assessment and review of comfort on footways and crossings**. This details how the footway should be measured, which data should be used for the assessment, and clear formulae for calculating the level of crowding.
- **Providing an excel template** for recording data, undertaking the analysis and reporting results.

Evidence and set priorities

The aim of this section of the Pedestrian Comfort Guidance for London is to allow designers to understand what the observed / forecast level of crowding means in practice. This will provide evidence for the need for changes to a design to support pedestrian movement or, where this cannot be achieved within the design, how to limit the impact of an uncomfortable footway. This will help to set priorities for the design, and also to balance the needs of different modes of transport.

The guidance supports design teams generating evidence and setting priorities by:

- **Examining pedestrian comfort along the whole pedestrian journey, rather than at a single location**. This approach was adopted for two reasons, being; A) to encourage users to consider the wider context of their scheme, and B) to enable users to devise pragmatic solutions. For example, a crowded section of footway that cannot be widened is less of an issue if the surrounding area is clear of obstructions. In this situation the design could mitigate the impact of the narrow footway by clearing the obstructions in the local area.
- **Including the real impact of street furniture and static pedestrians within the assessment**. Previous methodologies have applied the same space reduction for all types of street furniture.

However, the impact of street furniture varies depending on its use. The guidance provides measurements for a variety of street furniture so that the clear footway width can be accurately measured.

- **Providing clear standards** of what level of comfort is required for different user behaviours within a variety of area types, from high streets to transport interchanges.
- **Providing evidence of the impact of different levels of comfort** in different situations and area types so that the design solution can be tailored to the area.
- **Suggesting mitigation measures** to support the development of design solutions.

Table -1 Survey methods

	Why	What	How	Outcome
Pedestrian Flows	The flow survey forms the base for all of our other surveys which ask "what a re the levels of pedestrian activity in this location?"	CCTV footage was captured from 8am to 8pm at each site. Sites were surveyed on a normally busy day – whether this was a weekday or weekend differed by area type. A 5 minute sample was counted every half an hour, and the results factored up to get a pph value for each hour of the 12 hour survey	Using a digital counter, each pedestrian is recorded with a direction of movement and timestamp. The time- stamping of each record allows detailed analysis of patterns of pedestrian movement and behaviour (10 second intervals) a level of detail unobtainable with on site counts using conventional tally counters	• Time profile of flows per site • Directional profile of flows per site • Creation of activity profile for different area types • Identification of "key" hours for further examination in other surveys
Pedestrian Walking Speeds	Fruin identified pedestrian walking speed as a useful measure of the pedestrian experience on a footway. The freedom fro an individual to choose his or her own preferred walking speed shows that there is sufficient footway width for free movement and suggests the environment is suited to the type of pedestrian activity which characterises the site	Using CCTV footage, a random selection of pedestrians were observed to record their walking speed. This was carried out for the same sample periods as the flow surveys to allow comparison	The time taken for the selected pedestrian to walk between two predetermined gate lines will be recorded (pedestrian walking in both directions will be studied) Topographical surveys were used to determined the exact distance between the two gate lines, and this figure was used along with time recorded to complete the journey to calculate walking speed	• Average minimum and maximum walking speeds per site • Average minimum and maximum walking speeds for different area types • Analysis of how speed relates to area type, time of day, people per hour and footway width
Pedestrian Intersections	By observing and recording observations, collisions and bumps between pedestrians in different street environments, in different area types and different times of day, a better understanding of the effect of high flows, bi-directional flows and the effect of obstructions on pedestrian movement can be gathered	The following types of pedestrian interactions were recorded • Overtaking • Group contractions • Close proximity • Deviation around a static object/pedestrian • Minor weave (a deviation of a single step, with little change in speed) • Major weave (multi step deviation or significant change in speed) • Bump/Collision/Fall	Each observer was trained in order to identify different types of interaction. The observer views the CCTV footage of a defined zone within each site and records the different types for individuals, groups and two or three and groups of four or more	• Detailed breakdown of interaction type and number per site • Analysis of how changes in pedestrian activity (level of pph and directional weight) may result in restricted movement for pedestrians such as increased interactions with each other and with static obstructions.
Passing Distance	The current formula for measuring and categorising pedestrian capacity requires a measure of clear footway width against which pedestrian flows are compared. Existing guidance for measuring buffers are general, and do not take into account the type of obstruction being faced or particularities of location. These buffers need to be updated in order to incorporate such factors into calculations of pedestrian comfort	This space or buffer that pedestrians leave between themselves and other elements on the footway in different location types	An innovative method was developed for this survey. A number of snapshots were selected for analysis. These snapshots were transformed to fit onto a grid, using accurate topographical measures From these snapshots the buffers that pedestrians leave between themselves and other elements on the footway were then measured and recorded	• Analysis of the space pedestrians leave between themselves and others, including pedestrians walking in groups and those walking individually • Analysis of the space pedestrians leave between themselves and the wall and kerb edge • Analysis of the space pedestrians leave between themselves and street furniture, especially objects such as bus stops or ATM's that generate activity
Questionnaire Survey	Public opinion of a selection of sites was collected to identify thresholds of pedestrian comfort in different areas. This will give us insights into perceptions of pedestrian comfort in different location types and at different types and at different times of day.	Questions focused on pedestrian opinion of the comfort of the footway and on the impact of capacity, such as whether crowding affected their use of the area, willingness to return and length of stay	On site questionnaire surveys were conducted on each of the unobstructed sites and a selection of obstructed sites (back to footway bus stands, cycle parking, public seating, loading bays and ATM sites were surveyed). These questionnaire surveys took place on a similar day as the CCTV footage day (note: as the presence of the questionnaire survey team on site could affect other data being collected questionnaires were undertaken on a separate day)	• A comparison of the measures pedestrian comfort ratings with user opinions of the level of comfort • A preferable minimum pedestrian comfort • A preferable and minimum pedestrian comfort rating fro different types. • An understanding of public opinion of street furniture
Crossing Surveys	Pedestrian behaviour on crossings differs from the behaviour found on footways due to factors such as signal timings and personal safety. In order to better understand pedestrian activity and behaviour on crossings, a selection of crossings were observed. Due to the difficulty of capturing a whole crossing on CCTV, surveys were conducted on-site	The in site surveys need to cover the following sets of data: • Pedestrian flows and compliance with red man signal • Queue snapshots of the people waiting, and approximate locations • Signal timings • Measurements	A series of six snapshots were recorded on site during the peak hour. Snapshots were recorded when queues formed around crossings or, if the crossing had an island where people were waiting. People waiting on the road/outside of the island guard rail were also recorded Signal times were also recorded on-site using a stopwatch. The signal data was used to calculate the relative people per hour (pph) and the pedestrians per metre (PPMM) on the crossings arms	• The data collected allowed a comparison between non – compliant behaviour (crossing on the red man, waiting outside of the island area) and the capacity on a) the crossing arms and b) the size of the queue that forms on the crossing island

TRANSPORT PLANNING

Research supporting the Pedestrian Comfort Guidance for London document

Although a key benefit of the Pedestrian Comfort Guidance document is its ease of use, the research behind the document is the real benefit to users as it provides a robust evidence base for setting priorities and making design decisions.

The research program behind the guidance is the first large-scale study into these issues for over a decade, involving a detailed study of 75 sites across the TfL road network.

New research techniques were applied, and bespoke techniques developed for the research, where observed behaviour was combined with collected stated preferences of comfort to enable the guidance to distinguish between the needs of pedestrians in different area types.

The research program had three stages:

(1) Literature review and gap analysis to define the brief for the research

(2) Research program: detailed analysis of pedestrian behaviour at over 75 sites

(3) Analysis of data, with subsequent definition of the guidance

Literature review and gap analysis

At the beginning of the project, Atkins undertook a detailed review of academic literature relating to pedestrian comfort and existing sources on standards and planning for pedestrians. Over 20 sources were reviewed in depth, with key documents including:

- Pedestrian: Planning and Design - John Fruin
- Towards a Fine City for People - Gehl Architects
- Urban Space for Pedestrians - Boris S. Pushkarev and Jeffrey Zupan
- Department for Transport (UK) - Manual for Streets
- Department for Transport (UK) - Inclusive Mobility
- Transport for London – Streetscape Guidance

This review identified a number of knowledge gaps, but potentially more importantly that existing methodologies for conducting pedestrian comfort analysis were either inconsistent or ill defined. The key areas identified for further research and development were therefore:

- Up to date research on pedestrian flow, space, speed and comfort.
- An updated methodology for undertaking an assessment including what should be assessed and how to measure the site.
- Real vs perceived comfort levels, and what the impact is for designers.
- Real impact of street furniture on space for movement.

Research program

The research program was designed to be comparable with the existing Level of Service metric by measuring pedestrian activity, speed and restricted movement. However, a key benefit of the programme was the ability to infill a number of gaps in the understanding through the use of innovative techniques. For example, Closed Circuit Television (CCTV) footage was used to measure the distance pedestrians leave between themselves and street furniture, or between themselves and other pedestrians.

The majority of the surveys were undertaken using CCTV, allowing the same sample period to be examined for each survey type. Table 1 summarises the methods used for each survey, and its subsequent value to the overall study aims.

Analysing data and defining the guidance

The results of the studies were used in a comprehensive assessment of comfort, including across different area types, the tolerance to different comfort level, and the real impact of street furniture. The output of the analysis was used to:

- Refine the methodology for measuring capacity: a clear and consistent methodology is currently lacking in the UK.
- Develop a new classification method "Pedestrian Comfort Level" tailored to London: The research found that both the Fruin and Platoon LoS categorisations do not accurately reflect the pedestrian experience in London, as the intervals between the categories are too wide, so overestimating the level of comfort in many environments. A categorisation scale with smaller intervals between categories was introduced, with the impact of each new category on movement clearly stated using photographs and a description of the percentage of people walking along the street who would experience restricted movement (walking close to others, deviation from desired route etc).
- Provide guidance on the appropriate Pedestrian Comfort Level in different area types and times of day: The research into footway capacity highlighted various differences in pedestrian activity, pedestrian behaviour and public opinion in different area types. This allowed distinctions to be made when setting guidance on which capacity level is appropriate for five area types: Office and Retail, Transport Interchange, High Street, Residential and Tourist Attraction. This provides evidence for planning streets that operate at different levels of comfort at different times of the day.

Future research

This paper has presented the benefits of measuring pedestrian activity and including pedestrian needs in the design of streets and places. The PCGL document fills a number of key gaps in the pedestrian assessment toolkit, providing robust evidence of the needs of pedestrians in differing situations and area types.

However, although the Pedestrian Comfort Guidance for London document is a useful tool for assessing schemes and providing robust evidence for the design process, it needs to be considered alongside existing working practices and the experience of professionals working on the design of streets and places. This is because it does not address many issues related to pedestrian movement such as desire lines, land use, public realm design and perceptions of safety.

Finally, the research behind the PCGL document provides a springboard for future research questions. These include more detailed research into pedestrian behaviour when crossing the road, the impact of temporary obstructions and post implementation reviews of pedestrian perceptions of comfort.

Bibliography

1. CABE, Colin Buchanan (2007) Paved with Gold http://www.cabe.org.uk/files/paved-with-gold-summary.pdf [accessed 2 September 2010]
2. Department for Health Exercise referral systems: A national quality assurance framework http://www.dh.gov.uk/en/Publicationsandstatistics/Publications/PublicationsPolicyAndGuidance/DH_4009671 [accessed 2 September 2010]
3. Department for Transport (2010) Smarter Choices Home Page http://www.dft.gov.uk/pgr/sustainable/smarterchoices/ [accessed 2 September 2010]
4. Greater London Authority (2010) Mayor's Transport Strategy, Executive Summary http://www.london.gov.uk/sites/default/files/MTS_Executive_Summary_complete.pdf [accessed 2 September 2010]
5. Greater London Authority (2010) Transport for London Definition http://www.london.gov.uk/priorities/transport/working-partnership/transport-london [accessed 2 September 2010]
6. Greater London Authority (2004) Walking Plan for London http://www.tfl.gov.uk/assets/downloads/corporate/walking-plan-2004.pdf [accessed 2 September 2010]
7. Jane Jacobs Death and Life of Great American Cities 1961
8. NHS London (2010) Londoners urged to get walking as part of a campaign to get the Capital active http://www.london.nhs.uk/news-and-health-issues/press-releases/2009-press-releases/londoners-urged-to-get-walking-as-part-of-a-campaign-to-get-the-capital-active [accessed 2 September 2010]
9. RUDI Local authority transport policy to pay more attention to pedestrians, cyclists, and good urban street environments http://www.rudi.net/node/21668
10. Transport for London (2010) Pedestrian Comfort Guidance for London, London
11. Transport for London (2010) Pedestrian Comfort Guidance for London: Technical Report, London (unpublished)
12. Transport for London (2010) Pedestrian Comfort Guidance for London: Technical Appendix London (unpublished)

Re-engineering the Mississippi River as a Sediment Delivery System

Harley S Winer
Senior Engineer III
Atkins North America

Abstract

An environmentally sound concept of maximizing the benefits of the Mississippi River sediment load is proposed by allowing the river to naturally change its course to the Atchafalaya while maintaining navigation and flood control in the present channel of the Mississippi. A sediment lean, minimum necessary flow is determined for the lower Mississippi River to ensure navigation and freshwater needs. Some of the political and economic ramifications are anticipated and discussed. The need for a new engineering study is addressed. Finding an effective way to utilize the sediment load of the Mississippi River is essential if the coastal wetland ecology of southern Louisiana is to survive and flourish. Allowing the river to naturally change its course is ultimately the only viable option. The short comings of other means of using the river sediment load are discussed.

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Introduction

The coastal wetlands of Louisiana are disappearing at an alarming rate. Rates of land loss vary depending upon the period of measurement and the methods of analysis. The semi official web site LA Coast (http://www.lacoast.gov) reports that 1900 square miles (4921 square kilometers) have been lost since 1932. This is an average rate of three quarters of a hectare per hour or an American football field every half hour. If this land loss rate continues unabated, most of the Louisiana coastal wetlands would be non-existent by 2050. The LA Coast web site has maps of the projected land that could be lost by 2050 (http://www.lacoast.gov/landloss/index.htm).

The continuing loss of Louisiana coastal wetlands has both economic and environmental implications. A large portion of the nation's oil and gas comes from or is transported by pipeline through the Louisiana coastal area. The ports along the lower Mississippi River combined are the nation's largest port complex and combined is arguably the largest in the world. Louisiana also leads the nation in the harvest of seafood. The fringing coastal wetlands are nursery grounds for the commercial fish or the food chain that supports them. The fringing coastal wetlands of Louisiana are a predominant proportion of the fringing wetlands along the Gulf of Mexico. The potential demise of the Louisiana coastal wetlands could disrupt the ecology of the entire Gulf of Mexico.

The causes of the land loss are numerous. They are both natural and man-made. Natural causes are the settlement and consolidation of the alluvial deposits and the down-warping of the continental plate due to the enormous weight of these deposits from the Mississippi River. Some of the human induced causes are the dredging of navigation channels and oil and gas access canals. These are avenues for salinity intrusion that can kill freshwater marsh plants and boat wakes that erode the unconsolidated marsh soils. The spoil banks resulting from the dredging of these canals can alter and interrupt drainage patterns. Other causes of land loss are agricultural practices of constructing levees and draining wetlands (eventually the agricultural levees break and the land becomes open water); the extraction of sub-surface fluids (oil, gas, and produced waters) which reduces pore pressures and increases soil consolidation and thus subsidence; and the construction of flood control levees that prevent floods from depositing sediment in the river overbank area.

Chief among the human induced causes is the disruption of the natural delta building process that historically built the Louisiana coastal area. Penland[2] described the transgressive depositional system in which the river built a delta in the depositional mode until the delta being built created a longer less hydraulically efficient channel. When the river switched channels to a more hydraulically efficient channel, the abandoned delta becomes transgressive. Wave energy attacks the edges of the abandoned delta. Silts and clays are transported away by currents while sands stay in the swash zone. Eventually a sand beach lines the shoreline. Overwash processes make the sand shoreline transgressive and this process adds more sand to the shoreline. As the rest of the delta subsides the sand shoreline eventually becomes a barrier island. The Chandeleur Islands are an example of such a barrier shoreline. Some underwater sand shoals (e.g. Ship Shoal, Trinity Shoal) are examples of former barrier islands that have subsided below the water surface.

Brief history

Prior to the Civil war, two alterations to the Mississippi River system started a chain of events that began shifting the flow of the river toward the Atchafalaya. These were a cut made in the Mississippi at Trumbel's Bend and the removal of a thirty mile long log jam at the head of the Atchafalaya. By removing the log jam, the Atchafalaya became navigable. However, the increased flow began to scour a bigger channel which started a very gradual feedback loop wherein the enlarged channel allowed increased flow which in turn further enlarged the channel and its capacity.

After the 1927 flood, the shifting distribution of flow was increasingly obvious to scientists and engineers.

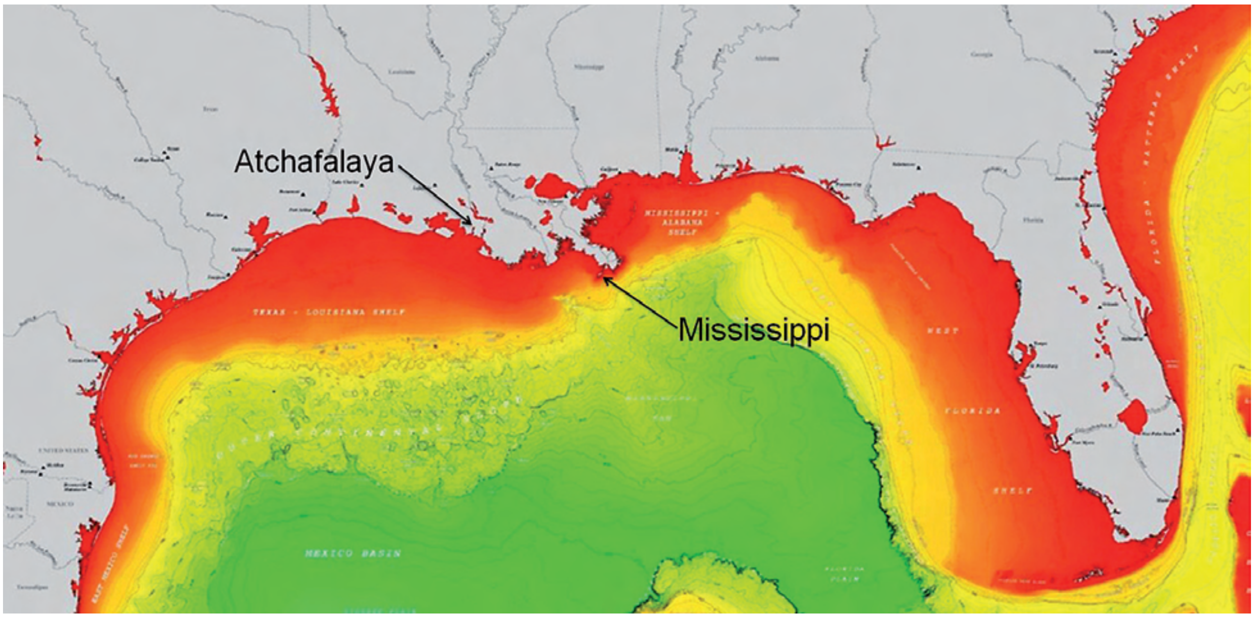

Figure 1 - Location of the mouth of the Mississippi and Atchafalaya Rivers and the amount of shallow continental shelf adjacent to their mouths. Red areas adjacent to the coast is shallow water, green are deep water.

When the cut was first made at Trumbel's Bend and the log jam removed at the head of the Atchafalaya, flow in Old River (so named because it was the old channel prior to the cut at Trumbel's Bend) would sometimes go west to east and other times go east to west, depending upon the relative magnitudes of the flows in the Red and Mississippi Rivers. As the distribution of flow shifted toward the Atchafalaya, flow in the Old River channel was almost entirely east to west, from the Mississippi to the Atchafalaya. A greater proportion of the flow of the Mississippi River was being diverted at Old River and going into the Atchafalaya River.

A study commissioned by the Mississippi River Commission[1] concluded that the Mississippi River was in the process of shifting its course. Further, the report stated that if nothing were done to counter this shift, the Atchafalaya would capture the predominant flow of the Mississippi by 1972. This could have disastrous consequences in terms of flood control and navigation for the developed region of the lower Mississippi River. The paradigm for addressing this problem was "How do we prevent the River from changing course?" Thus the Old River Control Structure was authorized, funded and built.

In the authorizing language, Congress came up with a political compromise to maintain the distribution of the total inflow from the Mississippi and Red Rivers the same as it was at the time of the authorization. This distribution was and is 70 percent going down the Mississippi River and 30 percent going down the Atchafalaya River. This helped to insure the freshwater, navigation and flood control needs of the lower Mississippi River but had unintended environmental consequences. Holding the river in its present alignment means that nearly the entire sediment load of the Mississippi River is deposited in deep water off the edge of the continental shelf. Thus, this sediment does not go toward building land to compensate for other areas of coastal Louisiana, which are suffering large land loss rates due to subsidence and lack of new sediment. Had the river shifted to the Atchafalaya valley, the sediment load would be deposited in shallow waters and build new coastal wetlands. The disadvantage of the present distribution of flow in terms of sediment management can be best understood by looking at Figure 1. The Southwest Pass mouth of the Mississippi River is at the edge of the continental shelf whereas the mouth of the Atchafalaya is a considerable distance from the edge of the shelf. This of course means that sediment coming out of the Atchafalaya has the opportunity to be deposited in shallow water and thus build land whereas the sediment coming out of Southwest Pass gets deposited in deep water and does not build any land.

The distribution of flow at the Old River Control Complex has been fixed by legislation. Seventy percent of the latitude flow (latitude flow refers to the combined flow from the Mississippi and Red Rivers crossing the latitude of the Old River Control Complex) is to go down the Mississippi River and thirty per cent of the latitude flow is to go down the Atchafalaya. This was the distribution of flow at the time of the enabling legislation more than half a century ago. Although the enabling legislation provided for the same distribution of sediment, more than the allotted seventy percent goes down the Mississippi River.

The Old River auxiliary structure, which was built after the 1973 flood weakened the Old River Control Structure, was specifically designed so that the inflow channel would capture a sediment rich flow, and thus help to correct the sediment distribution problem. However, shortly after the Auxiliary Structure was put in service, a hydroelectric power plant with a sediment lean inflow channel was installed to take advantage of the head differential between the Mississippi and the Old River Outflow Channel. (See Figure 2 for a map of the existing structures at Old River.)

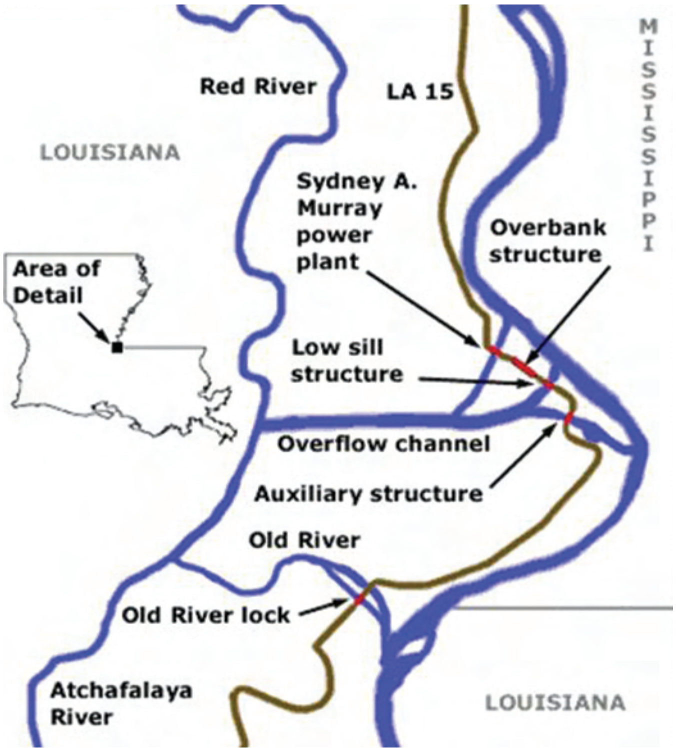

Figure 2 - Map of existing Old River Control Complex

Because of the operation of the hydroelectric plant, the legislated sediment distribution has not been met in recent years. Despite the fact that the Atchafalaya sediment load has been less than legislated, there is sufficient sediment so that there is delta formation at the mouth of the Atchafalaya and the Wax Lake Outlet. This is one of the few places in Louisiana where there is actually land accreting. Figure 3 shows the recent delta growth at the mouth of Wax Lake Outlet.

An alternative

If the sediment load in the Atchafalaya River were to be significantly increased, the rate of delta growth at Wax Lake Outlet and the mouth of the Atchafalaya River would also be significantly increased. In a previous paper Winer[4], the author proposed a new paradigm for the management of the lower Mississippi River. "How can freshwater, navigation, and flood control be provided for the lower Mississippi as the River is allowed to naturally change its course to the Atchafalaya?". A framework for an answer was provided in that paper. Changing Congressional legislation with such a drastic change of river management will be a long process that will affect numerous interests. Only a detailed engineering study can begin to do that. The problems in implementing this new paradigm were addressed in four parts. First is reconfiguring the Old River Complex through new construction and channel realignments.

An engineering and management plan was put forward to redistribute the latitude flows while preserving flood control, navigation and freshwater supplies to the lower Mississippi River below Old River, and maintaining electric generation at the hydroelectric plant. The second is providing for the crossings of existing utilities, roads and pipelines as the Atchafalaya channel grows to accommodate larger flows. The third is the problem of passing increased Atchafalaya flows through the present bottleneck of the Teche Ridge. Fourth is the issue of increased stages throughout the Atchafalaya and the Red River due to backwater effects as the flow line changes with a longer delta. There are probably numerous other problems with a proposal as grandiose as this; only a detailed study with input from all of the many stakeholders can uncover these concerns and articulate them so that they might be addressed.

Opposition

A radical proposal such as this will most likely be met with considerable political opposition. The present distribution of the latitude flows is mandated by Congressional legislation. A detailed engineering study is required to determine the feasibility of this proposal. A feasibility study is only the first step in the process of obtaining the required authorization to change the present operation. Examining this proposal is of great importance now that the loss of coastal estuaries in coastal Louisiana is being recognized as a serious national problem.

Although there may be considerable opposition to changing the status quo, implementation of this new paradigm will allow for natural delta growth at the mouth of the Atchafalaya River and could also significantly reduce dredging costs on the lower Mississippi River deep draft ship channel. Allowing the river to naturally shift its course will enable the river to create coastal wetlands to replace those that are constantly being lost due to storms, subsidence, natural forces, and the interventions of man. Healthy and abundant Louisiana coastal wetlands are essential to the overall health of the entire Gulf of Mexico.

The average annual sediment load of the lower Mississippi River (below Old River Control Complex) is estimated to be 131 million tons. Because the natural delta building processes have been drastically altered, a very large portion of this sediment goes off the edge of the continental shelf and is deposited in the deep waters of the Gulf of Mexico. The deltaic processes that built the landmass of southern Louisiana have been altered by the river engineering that has provided flood protection to the local residents and navigation benefits to the region and the nation. To date only a small portion of the sediment load is directed into marsh building through the beneficial use of dredged materials program and engineered sediment diversions. To counter the steady loss of the Louisiana coastal wetlands, it is imperative that a method be found to fully utilize the Mississippi River sediment load by directing its totality into wetland creation.

Figure 3 - Delta growth at Wax Lake Outlet and Atchafalaya River mouth

It is unrealistic to think in terms of removing existing levees so as to allow the river to naturally flood its banks and deposit sediment in the adjacent wetlands. Annual flooding is not an option for the citizens of the developed areas along the Mississippi River corridor. A more realistic approach would be to reconfigure the plumbing at the Old River Control complex so that the predominance of the sediment load will go down the Atchafalaya River where it will end up being deposited in the shallow waters of Atchafalaya Bay and the continental shelf, thus creating a large expanse of marsh lands that will help to compensate for the loss of wetlands in the rest of southern Louisiana.

Since the flow and sediment distribution at Old River is mandated by existing legislation, any change in the operation of Old River would require new legislation. The purpose of this paper is to advocate for a comprehensive analysis, in the form of a feasibility study, of the benefits and drawbacks of redirecting the sediment load at Old River. The arguments in favor of such a study are many fold.

First and primary is the impact of coastal land loss in southern Louisiana and the potential impact upon the entire Gulf of Mexico ecosystem. It is imperative that new coastal wetlands be created to replace the on going loss in the rest of Louisiana. It is not feasible, either monetarily or technically, to pump all of the sediment out of the lower Mississippi. Only by directing the flow into shallow water and allowing for natural deposition, is it feasible to utilize the sediment load for land building. Redirecting the sediment load to the Atchafalaya River will have another benefit of reducing the maintenance dredging in the lower Mississippi River. This will be a quantitative monetary benefit.

The technical problems of maintaining navigation, flood control and freshwater supplies to the lower Mississippi River corridor are easily solved, as are the issues of how to reconfigure the control structures at Old River to accomplish the proposed flow regimes. These have been presented in previous papers. Possible solutions to the technical problems are discussed in greater detail in previous papers[4,5].

A minimum flow of 8500 cubic meters per second (300,000 cfs) is sufficient to keep the salt water

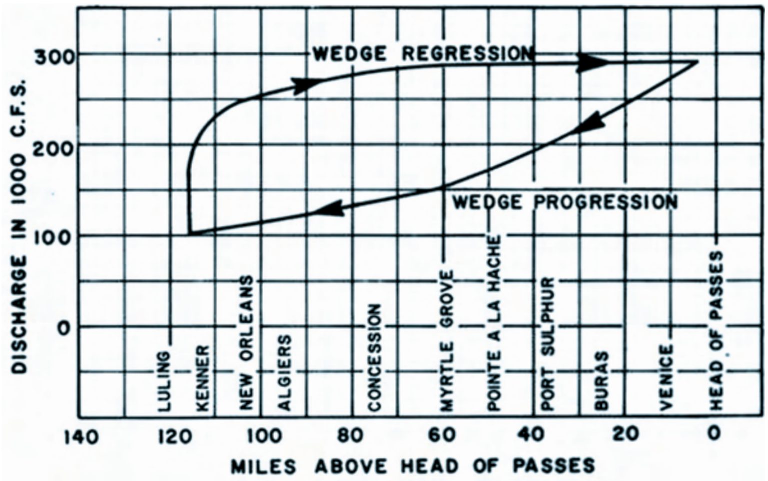

Figure 4 - Position of toe of saltwater wedge above Head of Passes versus flow in the Mississippi River (from Soileau et. al., 1989).

wedge out of the channel so that industrial and municipal intakes will draw freshwater. Figure 4 shows the curve[3] of how far up the Mississippi River the toe of the saltwater wedge will travel above Head of Passes with decreasing flow in the river. A flow of 300,000 cfs will push any existing saltwater wedge out of the river back to mile zero and will not allow it to advance.

Relocating pipelines and bridges that span the Atchafalaya River and allowing the Atchafalaya River channel to grow with gradually increasing flows so as to totally accommodate the Mississippi River and its sediment load is a very tractable engineering problem. The number of pipelines and bridges and the associated costs can easily be quantified.

As noted in the previous paper[4] a bigger channel through the Teche Ridge will be necessary to accommodate the increased flows. As the delta extends out the flow line will increase and the communities of Morgan City, Berwick and others will require increased flood defences or eventual relocation. Since all of the land on both sides of both the Atchafalaya and Wax Lake Outlet are developed, a larger channel through the Teche Ridge will require the acquisition of real estate and the relocation of residents and businesses.

Change

Changing the flow distribution as proposed will result in significant change to the south Louisiana landscape. However, as already pointed out, not allowing a change in the flow distribution will also result in significant change. The coastal wetlands will continue to subside and erode and coastal communities will become increasing vulnerable.

With increased sediment in the Atchafalaya, there will result some significant changes. First the Atchafalaya swamp will become transformed. More of the Atchafalaya basin will get filled in with the increased sediment load. Some swamp will remain but predictably it will not be as extensive as it is now. New swamp may develop further out into the gulf as delta building progresses, but that is not easy to predict.

Finally the Port of Morgan City will need to be relocated along with some industries, particularly oil rig fabrication that requires deep draft. Not only will the relocation of communities be inevitably necessary if increased flows were to become the regime for the Atchafalaya, the relocation, or abandonment, of the navigation channel is essential if land building from the sediment delivered by the river is to be maximized. Just as an inlet with jetties is incompatible with natural sand bypassing, a dredged navigation channel is incompatible with natural delta building. A dredged navigation channel provides a pathway for the bed load to be deposited in deep water, and over time the deep channel, in the case of the Atchafalaya, will be extended until it reaches the edge of the shelf, just as the Southwest Pass channel has reached the edge of the continental shelf.

Other methods of maintaining the coastal wetlands are not sustainable. In the next few paragraphs three of these are examined. First the proposal that dedicated dredging can be done to utilized the sediments of the river; second the concept of diverting the navigation channel somewhere below New Orleans; and last is the Third Delta Concept.

Dedicated dredging

Dedicated dredging is often proposed as a solution to the Louisiana coastal land loss problem. Dedicated dredging means the creation of land through the process of placing sediment using dredging techniques. Dredging costs have increased significantly in the last few years. There are numerous reasons for this and chief among them is the increased cost of fossil fuels. The contractual cost for dedicated placement of sediment delivered over any distance is well over $10 per cubic yard. If the Mississippi River sediment load were to be harvested and distributed over the southern Louisiana landscape, this would entail a dredging cost of over one billion dollars per year which is not sustainable. Aside from the costs, there are technical problems involved in capturing the sediment load. Dedicated dredging is not a sustainable solution to the land loss problem in Louisiana. The cost of land created through these type projects is in the neighborhood of $100,000 per hectare ($40,000 per acre). Creating an average of 5000 hectares per year to match the estimated land loss would require half a billion dollars per year with no end in sight. It is doubtful that such a program would be politically sustainable. But it is technically unsustainable because the problem is a lack of new sediment; moving the existing sediment around fails when nature rearranges the sediment with intense storms.

River diversions

There have been numerous proposals to divert the Mississippi River below New Orleans so that it can be a natural system below the diversion point and thus build a delta from its sediment load. There are two main variations on this theme. One is to divert the river and maintain the navigation channel in its present alignment going out Southwest Pass. The other variation is to divert the navigation channel. This option is known as "hang a right" or "hang a left" depending upon whether the navigation channel is diverted to the east or the west. Both variations include navigation locks to prevent the river flow from going down the navigation channel. Some proposals include the concept of sail through locks where the gates are separated by a sufficient distance so that gates can be opened and closed without requiring the ship to stop and tie off. Either way these concepts are fatally flawed because, without riverine (or tidal) flow, a deep draft channel from deep water through the estuary is vulnerable to being filled in by occasional strong tropical events. At some point the channel will have to cross open water where it will be completely vulnerable. The experience of the Mississippi River Gulf Outlet (MRGO) should serve as ample warning. Hurricane Georges in 1998 filled in most of the channel that was in open water. It took months and nearly $20 million dollars to open the channel. Hurricane Katrina in 2005 repeated the filling of the channel. This time the channel was de-authorized due to political opposition to reopening it. The entrance to the Mississippi River, having dozens of ship transits per day, cannot be left vulnerable to the vagaries of nature. The alternative of diverting the river flow and leaving the present channel as the navigation channel would leave that proposed channel vulnerable without riverine flow.

The proposed diversion of the Mississippi River at Old River while maintaining a minimum necessary flow in the present channel will maintain sufficient riverine flow within an existing confined channel so as to prevent catastrophic closure from tropical storms.

In addition to the problem of a vulnerable channel, these concepts will require the expensive dredging of a new channel through existing marsh, basically destroying marsh in order to create marsh.

The Third Delta

Information on the Third Delta Conveyance Channel (TDCC) concept can be found on the web at (http://dnr.louisiana.gov/ crm/ ciap/projectproposals/Region2/ Regional/2A.%20third%20delta. pdf). It is a proposal to divert two to three hundred thousand cfs of Mississippi River water at a point downstream of Donaldsonville, construct a channel 30 miles long to a bifurcation point from where the eastern branch will go an additional 35 miles to Little Lake in the Barataria estuary and the western branch will cross the Bayou Lafourche ridge and feed sediment to the Terrebonne estuary. This concept has gained support among many people in the Barataria and Terrebonne estuaries where land loss rates are very high.

This proposal has the potential to deliver large quantities of sediment to be deposited in shallow water. The cost of construction would be high. Numerous shallow pipelines will need to be relocated, hundreds of millions of cubic yards of sediment will need to be dredged and beneficially placed, and the developed Lafourche ridge will have to be crossed. The engineering challenges are not insurmountable, but the political and economic challenges are large. In the end, however, this proposal is as flawed as the present system of pinning the river channel in its present alignment. It is flawed because it is an attempt to make the river go somewhere other than where the river wants to go. The only sustainable option is to allow the river to go where it naturally wants to go.

Conclusion

This paper is an advocacy for working with nature rather than trying to control nature. The paper has presented a framework on how the river can be allowed to change course within the constraint of maintaining navigation on the Lower Mississippi River, preserving freshwater supplies of water to populations and industry and allowing for the sediment load of the Mississippi River system to be part of a natural delta building process. The concept is to manage the shift of the river channel to the Atchafalaya and divert a small fraction of the water into the existing lower Mississippi channel.

The paper has discussed other popular options and shown their weaknesses. Only the river can continually deliver sediment and build land. If a policy such as is advocated herein were to be adopted, several hundred years in the future, the river will have created a vast delta and the river will again want to change its course. But that is a problem best left to future generations.

References

1. Fisk, H.N., 1944, Geologic Investigation of the Alluvial Valley of the Lower Mississippi River, U. S. Army Corps of Engineers, Mississippi River Commission, Vicksburg, MS.
2. Penland, S., Boyd, R., Suter, J.R., 1988, Transgressive depositional systems of the Mississippi Delta plain; a model for barrier shoreline and shelf sand development, Journal of Sedimentary Research; November 1988; v. 58; no. 6; p. 932-949.
3. Soileau, C.W., Garrett, B.J., Thibodeaux, B.J., 1989, Drought Induced Saltwater Intrusion on the Mississippi River, Proceedings of Coastal Zone '89 Conference, ASCE.
4. Winer, H.S., 2006, A New Paradigm for Managing the Lower Mississippi River, Proceedings of the International Conference on Coastal Engineering, ASCE.
5. Winer, H. 2007, "Re-Engineering the Mississippi River as a Sediment Delivery System," Proceedings of Coastal Sediments '07 Conference, pp. 712-721, ASCE.

Translocating wildlife habitats: A guide for civil engineers

John Box

Associate Director

Water & Environment

Atkins

Kat Stanhope

Principal Ecologist

Water & Environment

Atkins

Abstract

There is increasing public and regulatory pressure to retain or replace established wildlife habitats during project design and delivery. However, they can also be moved. Habitat translocation is an effective and long-standing technique that can be used to rescue or salvage habitats which would otherwise be lost. Retention of a wildlife habitat within a site, even in a different location, allows their ecosystem functions to be maintained, for example the landscape structure, visual screening and wildlife corridor provided by a hedgerow. It takes years for newly planted or newly sown habitats to attain the maturity and complexity of established habitats. This paper presents three case studies which demonstrate how habitats can be successfully translocated and retained on site, albeit in different locations, resulting in wide-ranging project benefits.

Introduction

Wildlife in Britain is subject to continuing pressures due to the loss of and damage to their habitats. Valuable habitats for wildlife, such as hedges, species-rich grassland, ponds and mature trees should be retained in situ in the planning and design of infrastructure, built development and mineral extraction projects. However, this may not be possible because of physical constraints to site design and layout, access to the site, and commercial and financial reasons. The 2010 International Year of Biodiversity, and the growing awareness of wildlife issues throughout the development and construction industry, means there is an increasing challenge for project managers to retain wildlife habitats within a site. Biodiversity and wildlife can result in good media stories and provide useful publicity about a project.

Habitat translocation is a simple tool that can be used to rescue or salvage wildlife habitats which would otherwise be lost and to incorporate these into the design and layout of new infrastructure, built development or mineral extraction projects. Mature hedges that provide landscape structure, visual screening and wildlife corridors can be excavated and moved to a new location rather than cut down and chipped during site clearance operations (Figure 1).

Figure 1 - Mature hedges, which provide landscape structure, visual screening and wildlife corridors, can be excavated and moved to a new location rather than being cut down and chipped

Figure 2 - Moving a one metre section of hedge at Lightmoor in September 2007

Figure 3 - Placing a hedge section in the receptor trench, which had a slow-release fertiliser and water-retaining gel

Species-rich vegetation and its associated animals can be moved to new green areas and open spaces rather than bulldozed into topsoil mounds. Wetland vegetation and invertebrate rich sediment can be taken from an existing pond that needs to be filled for development and moved to a new pond. Established hedges, species-rich vegetation and ponds are valuable wildlife habitats and it can take years for newly created, planted and sown habitats to attain the same degree of maturity and complexity. Translocation ensures that native species of local provenance are used rather than imported plants.

New legislation driving change

Legislation is driving change through new legal duties placed on public authorities and public bodies in Britain to conserve biodiversity in carrying out their functions (Nature Conservation (Scotland) Act 2004; Natural Environment and Rural Communities Act 2006).

Public authorities in England and Wales must have regard to the purpose of conserving biodiversity in carrying out their functions, and in Scotland public bodies and office-holders must further the conservation of biodiversity (a tougher form of words). Public authorities include Government Ministers, government departments, local authorities and statutory undertakers. This is often achieved through the land use planning system.

Lists of habitats and species of principal importance for the conservation of biodiversity have been published for England[3], Scotland[11] and Wales[14]. Such habitats may support species given legal protection in Britain by the Wildlife and Countryside Act 1981 (as amended) or the Conservation of Habitats and Species Regulations 2010[1]. These habitats and species are a material consideration when assessing a development proposal that, if carried out, is likely to result in harm to the species or its habitat.

Dealing positively with wildlife issues can help to demonstrate to regulators and key voluntary bodies that a project has good environmental credentials.

Well known projects which have involved major habitat translocation include:

- Manchester Airport second runway,
- Stansted Airport,
- Channel Tunnel Rail Link,
- Heathrow Terminal 5,
- M4 extension at Twyford Down, and M6 Toll Road,

as well as mineral extraction at Durnford Quarry and Thrislington Plantation[2,4,5,9,10,12]. These projects featured large scale habitat translocations that were specific to each scheme and which were carefully designed and planned into the overall programme.

Smaller scale rescue or salvage translocation can be used to move ecologically important habitats, such as hedges, small trees, pond vegetation and sediments, and areas of species-rich grasslands, wetlands or heathlands. The likelihood of a successful outcome and the risk of failure are significantly influenced by the translocation methodology.

Translocation methods and options

Engineering and ecological skills are required in the selection and preparation of the receptor site, such that its landform and environmental characteristics match those of the donor site in terms of aspect, slope, soil characteristics (especially pH and nutrients) and hydrology.

The plant and machinery needs to be appropriate for the habitats being moved. For example, using low ground pressure tyres or tracked machinery to avoid soil compaction, and using large buckets to maximise the length, width and thickness of turves so that disruption to the vegetation is minimised.

Habitats are best translocated in the autumn when the soils are warm and moist and new root growth is possible before winter. Translocation in spring has a greater risk of failure as the roots may not develop before the stresses of summer; translocation in summer is very risky because the vegetation will have the greatest demand for water at a time when the supply of rainwater is lowest and the root system has been disrupted.

Following translocation, the habitats will require appropriate after-care similar to that required for newly created habitats and landscapes (e.g. cutting grasslands, trimming hedges, watering in dry weather) and monitoring to assess success and determine what, if any, remedial treatment may be required.

Translocating mature and complex habitats provides landscape structure, visual screening and habitat diversity more quickly than habitat creation

Figure 4 - New growth from translocated hedge and ash stump at Lightmoor in May 2008

Figure 5 - Translocated hedge with protective fence and showing new growth in July 2008

using seeds or nursery grown materials. Retaining features within a site, even in a different location, keeps their ecological functions, such as corridors for wildlife to move along and to provide connections between habitats.

In carbon terms, translocating features such as hedges within a site may be more environmentally sustainable and have a smaller overall carbon footprint than planting a replacement hedge using nursery grown trees and shrubs together with protective rabbit guards and stock-proof fencing.

A further driver for salvage translocation is the ecosystem services provided by wildlife habitats – such as flood mitigation, noise reduction, air quality improvement and visual screening. There is new official guidance in the UK on ecosystem services which will be used to support the UK cross-Government Public Services Agreement 28 on the natural environment that explicitly calls for the value of the services provided by the natural environment to be reflected in decision-making[6,13]. The benefits of these ecosystem services, and any losses as a result of development projects, need to be included in project cost/benefit models. Retaining wildlife habitats on a site - albeit translocated to a different part of the site - can maintain these valuable ecosystem services.

This paper sets out three case studies involving ground engineering and construction projects where salvage translocation was used to relocate important habitats within each site where these habitats could not be retained in situ. Each case study includes background information about the project, the method used to translocate the habitat, and the results of each translocation derived from post-translocation monitoring.

Case study 1: Lightmoor Urban Village, Telford

Background

Approximately 100m of hedgerow, within the footprint of the proposed development for the Homes & Communities Agency and the Bournville Village Trust, was assessed as being of considerable age and species richness. The evidence for this assessment included signs of past hedge-laying (e.g. large horizontal stems) and very thick woody stems (e.g. hawthorn stems up to 150mm in diameter at base), the presence of a 'bank and ditch' feature, and a diversity of woody species including hazel, ash, holly, field maple, common hawthorn and blackthorn with a ground flora containing bluebells.

Hazel, field maple and bluebells are associated with ancient woodlands and their presence indicates that the hedge was a remnant of an old hedgerow. This section of hedge could not be retained in situ and was translocated as part of the earthworks programme to form one side of an area of open space, thus creating a wildlife corridor with a small woodland at one end and an established hedge at the other.

Pragmatic judgement was required in deciding that sections of recently planted hedge within the site should not be translocated because their value was not considered to merit the extra costs involved in translocation compared to merely clearing them from the site.

Method

Approximately 100m of hedgerow was cut to a height of 300 – 500mm at the start of 2007 to prevent birds nesting. Ash and field maple trees up to 225mm in diameter were reduced to about one metre in height. The translocation was undertaken in late September 2007 at the start of the earthworks programme.

A trench was dug at the receptor area immediately prior to the hedge translocation to prevent the receptor trench drying out. The base of the receptor trench was scarified and slow-release fertilizer (20:4:10 N:P:K with mycorrhizal additive) and water-retaining gel was spread along the trench.

The hedgerow was dug out in sections (approx 1.5m width x 1m length) across the line of the hedge to a depth of at least 1m using a tracked 360° excavator with the largest ditching bucket available (Figures 1 & 2). During the excavation, a chainsaw was used to free roots and branches where necessary to prevent them being torn.

Sections of hedge with thick horizontal stems were moved without severing the stems and were transported immediately to the receptor trench before the next section of hedge was excavated. These hedge sections were placed in the receptor trench in the order in which they were removed and soil used to backfill any voids and gaps (Figure 3). Subsequent watering during the autumn was undertaken in dry conditions.

Figure 6 - Removing turves from the Boston Spa site with a modified bale-cutting bucket in spring 2006

Figure 7 - Placing turves on the new landscape bund at Boston Spa

Monitoring

Monitoring in May and July 2008 found abundant new growth within the translocated hedgerow and the ash tree stump which was translocated as part of the hedgerow (Figures 4 & 5), although there was evidence of die-back in holly which can be hard to establish once moved. In August 2009, there was extensive new growth on nearly all of the woody plants within the hedge (species include hazel, common hawthorn, holly, blackthorn, ash, field maple, dog rose).

The holly showed die-back in 2008 but in 2009 there was healthy regrowth with an average annual growth of 200mm in 2009. The old hawthorns with large horizontal stems were showing severe die-back in 2009, but the younger hawthorns show healthy growth. Further monitoring will be undertaken to see if the older hawthorns show regrowth in future.

The success of the 2007 translocation resulted in another hedge translocation in late 2009 using the same methodology; this demonstrates that that the earlier translocation brought tangible benefits to the overall development.

Case study 2: British Library book depository, Boston Spa, Yorkshire

Background

The British Library receives around three million new items every year requiring 12km of new shelving and the site of the British Library book depository in Boston Spa required an extension to cope with future storage. An existing earth bank supporting species-rich grassland with abundant orchids (pyramidal orchid, common spotted orchid, bee orchid) was within the footprint of the proposed extension and the planning authority, Leeds City Council, required the retention of this valuable ecological feature.

The solution was to translocate the species rich grassland to two new landscape bunds constructed using limestone spoil excavated from the foundations of the new building. The bunds are located close to other areas of species rich grassland that are being retained on the site that are part of the Thorp Park Trading Estate Site of Ecological or Geological Interest (SEGI), a non-statutory site of importance for nature conservation.

The retention of approximately 8,000m^3 of spoil on site saved some £250,000 in landfill costs, contributed to a reduction in lorry movements through the nearby villages and reduced the overall carbon footprint of the development.

Method

The species-rich grasslands covered an area approximately 130m in length by 10m wide on a steep northeast facing slope. There were two distinct grassland communities – short open grassland covering around 900m^2

Figure 8 - Turves were also transported by a flat-bed trailer and placed by telehandler

Figure 9 - Translocated grassland on the main landscape bund at Boston Spa

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that was typical of calcareous soils and taller grassland covering around 400m² that was more characteristic of neutral soils. The receptor sites were the north-east face of the main landscape bund (the same aspect as the donor site) and the south-east face of the adjoining bund (as an additional site). The landscape bunds were designed with a surface layer of at least 1m of limestone over the materials used to construct the bund in order to mimic ground conditions at the donor site. The bunds were graded to give slopes of 1v:2h and were not covered with topsoil or treated in any other way.

The translocation involved carefully excavating turves that were 1m by 0.5m and 300mm deep using a tracked 360° excavator with a modified bale-cutting bucket (turf box cutter) (Figure 6). The turves were either placed directly by the excavator onto the toe of the south-east face of the bund that was very close to the donor site (Figure 7) or placed on a flat-bed trailer for transport to the other bund where they were placed at the base of the north-east face by a telehandler fitted with a wide bucket (Figure 8).

Each turf was carefully placed to ensure a tight fit with the adjacent turves and was pressed down by the bucket to expel air from between the turves and the underlying substrate. Turf offcuts and soils from the donor site were used to fill any gaps between turves and along the four external sides of the translocated turves. Voids between or under the turves were not permitted because the air spaces would cause drying out of the fragile grassland root system.

Rain during the latter part of the translocation operation caused some problems with vehicle movements on site but meant that watering of the turves immediately after translocation was not required. The translocation works took about three weeks to complete in late February to March 2006. Initial inspections of the translocated turves in May and June 2006 found that key species such as cowslips and pyramidal orchids were already flowering.

The tops and slopes of the bunds that did not have translocated turves were not topsoiled or seeded. Once the surface soils have weathered, natural colonisation and natural succession will produce orchid-rich and diverse grasslands of high nature conservation value because of the proximity of a good seed source in the translocated turves and the other adjacent species-rich grasslands that were retained in situ.

Monitoring

The species rich grasslands on the site, both the translocated grasslands and other areas of species-rich grasslands in the SEGI, are subject to a 10 year monitoring and management schedule with reports submitted to Leeds City Council as required by planning conditions. The success of the translocation and the continuing habitat management is measured against targets such as the presence and abundance of key plant species, the ratio of herb species/grass species, the sward structure and height, and the extent of bare ground.

Monitoring the translocated grasslands in June 2008 found that many of the targets had already been reached just two years after translocation. There was very little slippage of turves or gaps between the turves and the species-rich grassland was flowering well and contained pyramidal orchids, common spotted orchids and cowslips (Figures 9 and 10).

Bee orchids were not recorded in the translocated turves in June 2008; however, only one bee orchid was found in all the species-rich grasslands elsewhere on the British Library site and their absence in the translocated turves in 2008 may have been due to weather conditions rather than the translocation process.

The initial monitoring results provide a basis for cautious optimism about the final outcome although it is too soon to say that the vegetation on the receptor site is the same as the original vegetation of the donor site.

Case study 3: i54 Strategic Employment Site, Wobaston Road, Wolverhampton

Background

i54 Wolverhampton is a 90ha site to the south of Junction 2 on the M54. Since 2002, the site has been subject to an extensive programme of preparation works by Advantage West Midlands and its joint venture

Figure 10 - Wildflowers flourishing in the translocated grassland in June 2008

partner Wolverhampton City Council including removal of contaminated soil, earthworks to form development platforms, the construction of a site spine road and footpaths/cycleways, and the provision of drainage and landscaping. A new access from the M54 is proposed.

Engineering consultants Mouchel are working with Atkins (responsible for ecology) and Potterton Associates (responsible for landscape design) to deliver the project. Implementation of the Landscape and Nature Conservation Management Plan prepared by Atkins and Potterton Associates in 2007 has resulted in ecological enhancement and mitigation involving valuable habitats and legally protected species (water voles, badgers, great crested newts and nesting birds).

The principal green infrastructure elements include the retention of boundary hedgerows and ancient woodland together with

Figure 11 - Digging up selected wetland plants at the Wolverhampton site in September 2007

Figure 12 - Fen and swamp vegetation in the surface water attenuation area at the Wolverhampton site in 2009

the enhancement of existing copses, ponds and watercourses and the creation of footpaths. Habitat translocation of fen/swamp vegetation, as well as sections of hedgerow and a young oak tree, has enabled these features to be retained within the site albeit in different locations.

Translocation of fen/ swamp vegetation

The fen/swamp vegetation had developed on heavy clay soils and was dominated by reed sweet grass with great willowherb and tufted hair grass, as well as soft rush, meadowsweet, lesser pond sedge and brooklime. The translocation was undertaken in September 2007 using a tracked 360° excavator with a digger bucket to take approximately 500m² as turves from the wetter areas of the fen/ swamp vegetation as these had the greatest ecological value (Figure 11).

Figure 13 - Hedge and 10m tall oak tree immediately after translocation at the Wolverhampton site in October 2006

The turves were placed at four locations within the receptor site to 'seed' it with aquatic planting. The receptor area was a large expanse of low lying land which had been previously shaped and compacted as a surface water attenuation area. This area receives surface water drainage from the i54 site as the final stage in a sustainable drainage system involving a series of newly created swales and ponds along a watercourse that runs through the site and discharges into the adjacent brook.

The whole of the i54 site is subject to the Landscape and Nature Conservation Management Plan, which includes annual monitoring in spring and summer with an annual report being submitted to the planning authority and other interested parties as part of planning conditions. Monitoring of the fen/ swamp vegetation in 2008 and 2009 showed very successful re-growth of aquatic plants. Much aquatic vegetation appears to have developed of its own accord from the existing seedbank and plant roots in this area. However, the translocated turfs clearly stand out as areas of more established vegetation and provide structural diversity within the new wetland habitat.

The vegetation was originally classified as reed sweet grass swamp before translocation and the vegetation can still be classified as this community in 2009. This surface water attenuation area is not yet receiving surface water runoff as the development plots are yet to be constructed. As a result, a mosaic of habitats has developed with wetter areas indicated by brooklime and drier areas with meadowsweet and great willowherb (Figure 12). Nettle, broad-leaved dock and spear thistle are also present but these species are expected to decrease as the area starts to receive surface water runoff and becomes wetter.

Mallard and lapwing have been recorded and, in 2009, three pairs of snipe and two pairs of little ringed plover nested in this area.

Translocation of hedge and oak tree

Two 120m long sections of hedgerow were translocated in late October/ early November 2006 together with a small oak tree about 10m high and around 40 years old. These were moved to the western boundary of the site where they linked to existing hedges that were strengthened with new hedge planting to create continuous wildlife corridors around the i54 site boundaries.

Both hedgerow sections were heavily coppiced before translocation. A receptor trench was excavated by a 360° tracked excavator which then moved the hedge in 2m long sections using a toothed bucket; a power saw was used to cut roots and branches where necessary. The excavator also moved the oak tree by easing it slowly from the soil and moving the whole tree together with its rootball. The hedgerow sections and the tree were placed in the receptor trench and immediately backfilled (Figure 13). Aftercare comprised watering in dry periods.

Figure 14 - Hedge at the Wolverhampton site in April 2007, some 6 months after translocation

The monitoring of the hedges is by one annual visit in summer using fixed point photography and measuring growth rates. The translocated hedges showed no evidence of die-back but had abundant new growth of up to 400mm in April 2007, some six months after translocation (Figure 14), and both the translocated hedges and the oak tree showed healthy new growth in 2008 and 2009.

Conclusion

Translocation is not a new technique. Individual trees have been moved since at least 1700 by wealthy landowners. Techniques were devised by landscape designers, such as Capability Brown to carefully dig up mature trees whilst maintaining the root system and move them on specially designed machines[8].

Habitat translocation can be used in a planned and designed way through the application of guidance such as the UK Highways Agency Design Manual for Roads and Bridges[7] and the CIRIA best practice guide to habitat translocation[2].

The three case studies presented in this paper demonstrate that important ecological habitats can be retained during the development of a site, even if rearranged and in different locations. Habitat translocation is an effective technique that enables mature and complex ecological resources to be retained on a site or in the vicinity of a site.

This maturity provides landscape structure, visual screening and habitat diversity more quickly than habitat creation using seeds or nursery materials

The retention of a habitat within a site allows ecological functions associated with the habitat to be retained within a site - for example, the habitat connectivity and wildlife corridor provided by a hedgerow. Translocation can generate ecological resources for new habitat creation schemes - such as moving wetland vegetation from an existing pond to a new one - and ensures that native species of local provenance are used rather than imported plants.

The success or failure of habitat translocation depends on four critical factors:

- Matching the environmental context of the receptor site to that of the donor site
- Using appropriate plant and machinery for the habitats being moved
- Translocating habitats at the right time of year
- After-care and monitoring as with any newly created habitat.

There is a growing evidence base for both success and failure in habitat translocation which underpins the application of these critical factors to the particular set of circumstances on any given site. Habitat translocation has as much chance of success as habitat creation.

The probability of a successful outcome can be established by reference to experience and to published case studies so that the reasons for success or failure can be identified[2,4,5].

Monitoring of habitat translocations over the long-term is very important in identifying the success of both the translocation technique and subsequent management of the habitats, thus allowing remedial actions to be implemented. Furthermore, the data from such monitoring will result in greater understanding of the ecological and engineering limitations associated with habitat translocation, improved and cheaper habitat translocation methodologies, and an increase in the likelihood of success.

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References

1. Abbatt J. 2007. New rules demand more care of protected species and habitats. Civil Engineering 160: 148.
2. Anderson P., Groutage P. 2003. Habitat translocation - a best practice guide. C600, CIRIA, London.
3. Biodiversity Action Reporting System (2010). England Biodiversity List. http://www.ukbap-reporting.org.uk/news/details.asp?x=45 (accessed 24 January 2011).
4. Box J. 2003. Critical factors and evaluation criteria for habitat translocation. Journal of Environmental Planning and Management 46: 839-856.
5. Bullock J.M. 1998. Community translocation in Britain: setting objectives and measuring consequences. Biological Conservation 84: 99-214.
6. Department for Environment, Food and Rural Affairs 2007. An Introductory Guide to Valuing Ecosystem Services. PB12852, Defra, London, 2007. http://www.defra.gov.uk/environment/policy/natural-environ/documents/eco-valuing.pdf (accessed 24 January 2011).
7. Highways Agency 2001. Translocation. In: Design Manual for Roads and Bridges, Volume 10, Section 4, Part 1 (HA 84/01), section 3.5, p. 3/3. http://www.standardsforhighways.co.uk/dmrb/index.htm (accessed 24 January 2011).
8. Mabey R. 2007. Beechcombings: the narratives of trees, pp.102-103 & illustration facing p.83, Chatto & Windus, London.
9. Palmer D., Wilbraham P. 2008. Heathrow Terminal 5: twin rivers diversion. Civil Engineering 161: 25-29.
10. Roberts G. 2000. The relocation of ancient woodland. Quarterly Journal of Forestry 94: 305-307 & 310-312.
11. Scottish Natural Heritage 2005. Scottish Biodiversity List. http://www.snh.gov.uk/protecting-scotlands-nature/biodiversity-scotland/scottish-biodiversity-list/ (accessed 24 January 2011).
12. Trueman I., Mitchell D., Besenyei L. 2007. The effects of turf translocation and other environmental variables on the vegetation of a large species-rich mesotrophic grassland. Ecological Engineering 31: 79-91.
13. UK Government (2007). PSA Delivery Agreement 28: secure a healthy natural environment for today and the future, Her Majesty's Stationery Office on behalf of Her Majesty's Treasury, London, October 2007. http://webarchive.nationalarchives.gov.uk/+/http://www.hm-treasury.gov.uk/d/pbr_csr07_psa28.pdf (accessed 24 January 2011).
14. Wales Biodiversity Partnership 2009. Habitats and species of principal importance in Wales lists. http://www.biodiversitywales.org.uk/legislation__guidance-20.aspx (accessed 24 January 2011).

Delivering wetland biodiversity in the London 2012 Olympic Park

Ian Morrissey

Senior Environmental Scientist

Water & Environment

Atkins

Mike Vaughan

Principal Engineer

Water & Environment

Atkins

Abstract

Appointed as the river edge engineers for the London 2012 Olympic and Paralympic Games, Atkins is responsible for the design of the wetland and river edge habitat enhancement works in the Olympic Park. Working alongside the Park's Landscape Architects (LDA – Hargreaves), and with the Olympic Park Biodiversity Action Plan (BAP)[2] document providing the reference for biodiversity requirements we are creating nearly 2.5ha of wetland habitat. This paper provides detail of the range and extent of the wetland habitats and pays particular reference to the innovative use of on-site wetland plant trials to inform the design. These trials were implemented in response to identified environmental constraints in order to reduce the risk of wetland plant failures in the final design through the assessment of wetland plant species performance and the suitability of a range of plant installation techniques.

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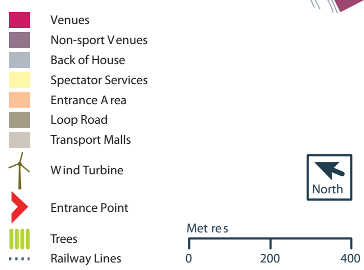

Figure 1 - Schematic of Olympic Park layout showing waterways (blue) and main landscaping areas (green)

Extents

The wetland habitats have been designed to provide ecological gains aside the Olympic Park's waterways and to meet the conditions of the Olympic Park BAP[2] document, when completed, they will form an area the size of three football pitches containing 40 wetland plant species at a total of approximately 300,000 wetland plants, making it one Europe's largest urban river edge habitat enhancement programmes to date. Located in east London's Lower Lea Valley, the River Lee and its associated waterways form an arterial route through the Park and as such will provide a significant focus during both the Games and following transformation into the Legacy period (Figure 1).

The vision to bring the once neglected river systems into the Park landscape by opening up the river corridor and reducing bank slope angles will significantly improve access to the public, connecting visitors with the

Figure 2 - Wetland bowl area in North Park showing newly planted wetlands protected from wildfowl grazing by orange fencing

various wetland elements including reedbeds, wet woodlands, amphibian ponds and ecologically enhanced river edges. From a wetland habitat diversity perspective the Olympic Park can be broadly divided in two. To the north lies the ecological park, containing the majority of the wetland features alongside the River Lee, and in the south, around the stadium island, there are smaller ecological gains for the Waterworks River and Bow Back canal systems.

The focus in the North Park will be the wetland bowl area containing two online reedbeds in direct hydrological connectivity to the river (Figure 2). These total over 5,000m^2 and will be composed mainly of common reed *Phragmites australis*, a UK BAP priority habitat known to support an abundance of insect, amphibian and bird life.

Within the reedbeds, wetland channels have been designed to increase habitat complexity, maximise reed edge extent and provide refuge for a range of fish species including eel *Angulia angulia*, a London 2012 BAP priority species. An additional reedbed to the north of the bowl will provide an extra 550m^2 of Phragimtes wetland habitat.

Bio-engineered banks (2.5km in total), planted with wetland species

Figure 3 - Wet woodland under construction

will maintain longitudinal ecological connectivity through the North Park, connecting the larger wetland features including two wet woodland habitats and a number of amphibian ponds. Two new wet woodlands, totalling 0.4ha will provide off main river habitat, with excavated channels maintaining hydrological and ecological connectivity with the River Lee. These areas were designed to retain waters from the Lee during periods of impoundment as a result of the new water level control structures at Three Mills Island. The wet woodland habitats are being planted with a mix of shade tolerant sedge species and typical wet woodland trees e.g. alder, on raised features within the landform (Figure 3). Shallow depressions were also designed into the wet woodlands to hold standing water and provide a range of moisture gradients across the habitat. Marginal wetland flowering plants are being used in such areas adding species richness to the habitats.

Wetland planting is also being provided for three new amphibian ponds, summing 0.2 ha. All of the ponds are fed by drainage waters from the Park's concourse, with the largest having been designed with an adjustable feed from the River Lee to allow maintenance of a permanent water level. The maintained water body will be planted with a range of plants including oxygenating submerged aquatics e.g. rigid hornwort *Ceratophyllum demersum* and species such as water forget-me-not *Myosotis scorpioides* to provide suitable egg laying sites for newts. A series of log walls installed alongside the ponds will increase ecological value through the provision of habitat for invertebrates and hibernation sites for amphibians.

Planting trial

In order to inform the wetland designs and reduce the risk of wetland plant failures in the final design an on-site riverside planting trial was undertaken to investigate and advise on wetland plant species selection, plant installation techniques, species specific planting elevations and potential environmental constraints to wetland plant establishment and performance. Of particular concern to the designers were the effects of

Figure 4 - View of planting trial following installation in 2008

changing river levels arising from the impoundment of the River Lee to the south of the Park. Flow control structures have changed the river regime from an inter-tidal habitat to a freshwater fluvial system experiencing a twice daily rise and fall in river level. The fluctuation in level in the River Lee resulting from tide-lock at Three Mills Island is typically 400mm. This was identified as presenting a potential constraint to the successful establishment of the wetland plants along the river edge as well as having implications the range of species that were suitable for installation and hence overall wetland habitat biodiversity[4].

The planting trial was established in the North Park in September 2008 along a 50m length of river bank extending from mean low water level (2.4m AOD) to a top bank level of approximately 4.2m AOD, an elevation of 1.8m. The trial platform was divided into eight separate treatment areas across which a total of 15 species of wetland plant were trialled for a period of 12 months (Figure 4). Plants were installed at a standard density of $20m^2$ across the trial area to ensure that comparative analysis between treatments could be undertaken. The influence of bank form was also qualified with the plants being installed on both sloped (1 in 2.5) and terraced platforms. The trial areas and treatments are outlined below.

- Area A – Pre-established coir pallet installation area trialling priority species on terraced platform.
- Area B - Pre-established coir pallet installation area trialling priority species on 1 in 2.5 slope.
- Area C – Plug and bare root planting area on 1 in 2.5 slope.
- Area D - Plug installation through coir pallet to trial bank-side species.
- Areas E, F, G and H – Pre-established coir pallet versus plug planting trial areas.

Figure 5 - The authors inspecting root development in pre-established coir pallets

The relative success of the different planting techniques and approaches was investigated through the assessment of the performance of wetland plants installed on the river bank as either pre-established coir pallets (wetland plants established in a 2 x 1 x 0.1m coir coconut fibre mattress pegged on to the planting platform (Figure 5), or plug plants (wetland plants grown in root trainers and plugged either directly into the river bank substrate, or through jute geotextile into river bank substrate). Their performance was monitored by a combination of biometric measurement undertaken seasonally and by monthly repeat fixed-point photography of the individual trial areas. This information was used to assess plant establishment rates across the trail platform. Biometric monitoring undertaken at trial included the calculation of:

- Percentage cover obtained by installed species.
- Percentage cover of weed plants (defined any colonising plant in addition to the installed species).
- Percentage of planted area with no plant cover i.e. bare substrate.
- Average height obtained by installed species.

The influence of fluctuating water levels on plant performance was also examined by running the planting trial in conjunction with a water level monitoring programme. River levels were gauged with a Troll 500 with level and temperature sensors, based on a silicon strain gauge. A data logger was used to record river level at 15 minute intervals throughout the trial period allowing the calculation of inundation depth and frequency across the trial platform in response to fluvial flow variability and the effects of tidal fluctuations (Figure 6).

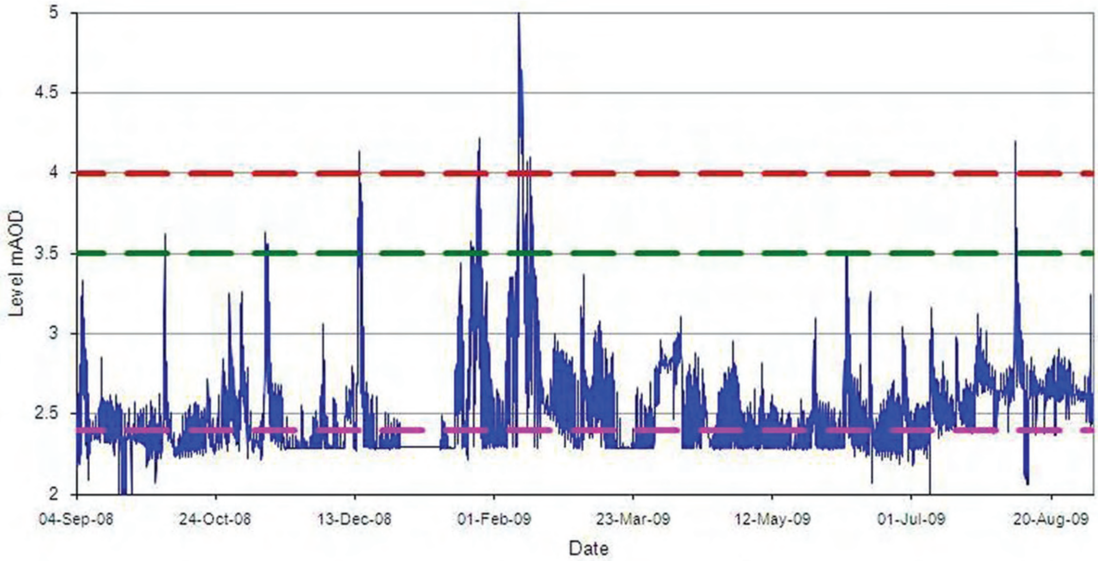

Figure 6 - River level record (mAOD) during trial period (2008/2009) showing periods during which lower (pink solid), middle (green dashed) and upper (red dashed) trial areas were inundated

Level exceedance values calculated from this flow record allowed the performance of plants obtained from the biometric monitoring to be correlated against these flow statistics.

Moving forward

In response to the wetland plant trial findings poor performing and failing species were removed from the final planting arrangements and replaced with those plants shown to have established successfully over the year long monitoring period. The trial results were used to make informed decisions on the appropriate topographies across the larger wetland features and the need for fine adjustments to be made to the final planting scheme. This took the form of relocating certain species to more appropriate river bank levels in order to ensure inundation frequencies were suitable.

The trial successfully validated our approach of using pre-established coir pallets as the preferred planting installation technique to be adopted along the river banks and larger wetland features. This was demonstrated by the greater success of plants when installed as pre-established coir pallets compared to the same species trialled as plug plants. For example, percentage cover and height achieved over the trial period by reed sweet-grass *Glyceria maxima* installed as pre-establish coir pallets was 75% and 425mm respectively, compared to 2.5% and 250mm for the same species when installed as a plug plant. The trial also provided some very useful pointers as to the potential effects that land contamination could have on the final planting arrangements with leachate causing plant failures in certain trial areas.

This highlighted the value of the remediation programmed to be undertaken after the trial by the Enabling Works contractor along the river banks. The negative effects of wildfowl grazing on plant establishment were also observed at trial resulting in the incorporation of temporary wildfowl fencing into the landscaping programme to exclude bird grazing pressure up to the Games period.

Conclusion

The design and implementation of the on-site planting trial proved a vital tool in allowing us, as designers, to evolve and finalise the approach in the delivery of wetland habitat improvements in the Olympic Park. The eventual aim is for the quality of the individual wetland habitat elements to meet appropriate criteria for their designation as a site of Borough Importance[3]. The ethos of maximising opportunities and designing for wider ecology, coupled with the commitment to monitoring and undertaking appropriate management, will go a long way to achieving this goal. The environments created will provide a significant betterment to the habitat diversity and ecological connectivity of the waterways in the lower Lee Valley, transforming a once neglected river corridor characterised by high levels of contamination and a prevalence of invasive species, into an area containing 2.3ha of new wetland Olympic Park BAP habitat.

References

1. Crawford, R. M. M. 1996. Whole plant adaptations to fluctuating water tables. Folia Geobotanica et Phytotaxonomica. 31:7–24.
2. ODA (2008). Olympic Park Biodiversity Action Plan, www.london12.com, accessed January 2011.
3. Wansbury, C. and Jackson, R. 2010. The Olympic Park – A Biodiversity Action Plan in Action. In Practice, Bulletin of the Institute of Ecology and Environmental Management. Number 70, December 2010.
4. Leyer, I. 2005. Predicting plant species' responses to river regulation: the role of water level fluctuations. Journal of Applied Ecology. Volume 42, Issue 2, pages 239–250.

ATKINS