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ATKINS

Technical Journal

Papers 001-015

FOREWORD

I am delighted to introduce you to the first issue of the Technical Journal.

This journal will bring together technical papers that have been produced across the Atkins group by our own exceptionally talented members of staff. It showcases the technical excellence that thrives within Atkins. This issue is dedicated to our Highways & Transportation capability but future issues will feature papers from across the whole Atkins portfolio.

Atkins is a multinational engineering and design consultancy, providing expertise to help resolve complex challenges presented by the built and natural environment. Whether it's the concept for a new skyscraper, the upgrade of a rail network, the modelling of a flood defence system or the improvement of a management process, we plan, design and enable solutions. The papers contained in this journal demonstrate how we, as a group, rise to the challenges presented to us and provide our clients with innovative, cost effective and sustainable solutions.

I hope you enjoy the papers presented as much as we enjoyed producing them.

Chris Hendy

Chair of TSol Technology Board

Transport Solutions
Highways & Transportation

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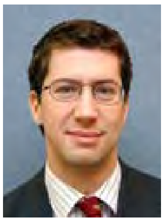

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Abstract

The Dubai Metro Light Rail scheme is a flagship project in the United Arab Emirates. It will be the longest, fully automated rail system in the world, and is currently one of the largest civil engineering projects under construction. The first section of the rail system is due to be opened in September 2009. This paper describes the scheme outline and contractual set-up for the viaduct design and discusses the design and construction of the viaduct substructure. In particular, the design methodologies used for the piled foundations, single reinforced concrete columns and prestressed concrete pier heads are discussed as well as the design of elastomeric bearings used extensively for most of the viaduct spans. Seismic loading governed the design of many of the foundations and the seismic analysis and design methodology adopted are discussed, together with specific reinforcement detailing requirements. Rail-structure interaction analysis and design are also covered. Other critical design issues resolved include, fatigue performance of cranked reinforcement and the treatment; of the onerous construction loading from overhead gantries used to erect the precast deck segments.

1. Introduction and project background

In July 2005, the Government of Dubai Road and Transport Authority (RTA) awarded a design and build contract to the Dubai Rapid Link (DURL) consortium for the construction of the first and second stages of the Dubai Metro Red and Green Lines. The DURL consortium comprises the Japanese companies Mitsubishi Heavy Industries, Mitsubishi Corporation, Obayashi Corporation and Kajima Corporation together with Yapi Merkezi of Turkey. Construction of the infrastructure and stations was the responsibility of a joint venture between Kajima, Obayashi and Yapi Merkesi (Japan-Turkey-Metro joint venture or JTMjv). The JTMjv appointed Atkins as their designer in 2006. This project organisation is illustrated in Figure 1.

The Dubai Metro will be a driverless, fully automated metro network and will be the longest fully automated rail system in the world. Completion of the first section of the Red Line is planned for September 2009, followed in 2010 with the first section of Green Line.

A further Blue Line (along Emirates Road) and Purple Line (an airport express route) are planned for subsequent years. The metro route map is illustrated in Figure 2, with a more detailed Red Line route map shown in Figure 3.

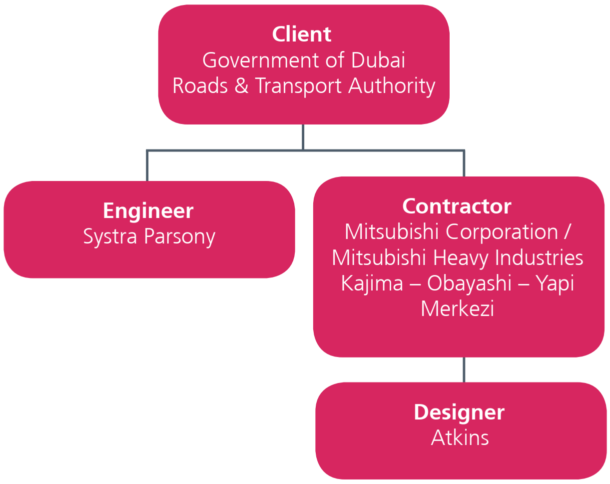

FIGURE 1
PROJECT ORGANISATION

STRUCTURES

FIGURE 2
PROPOSED ROUTE MAP

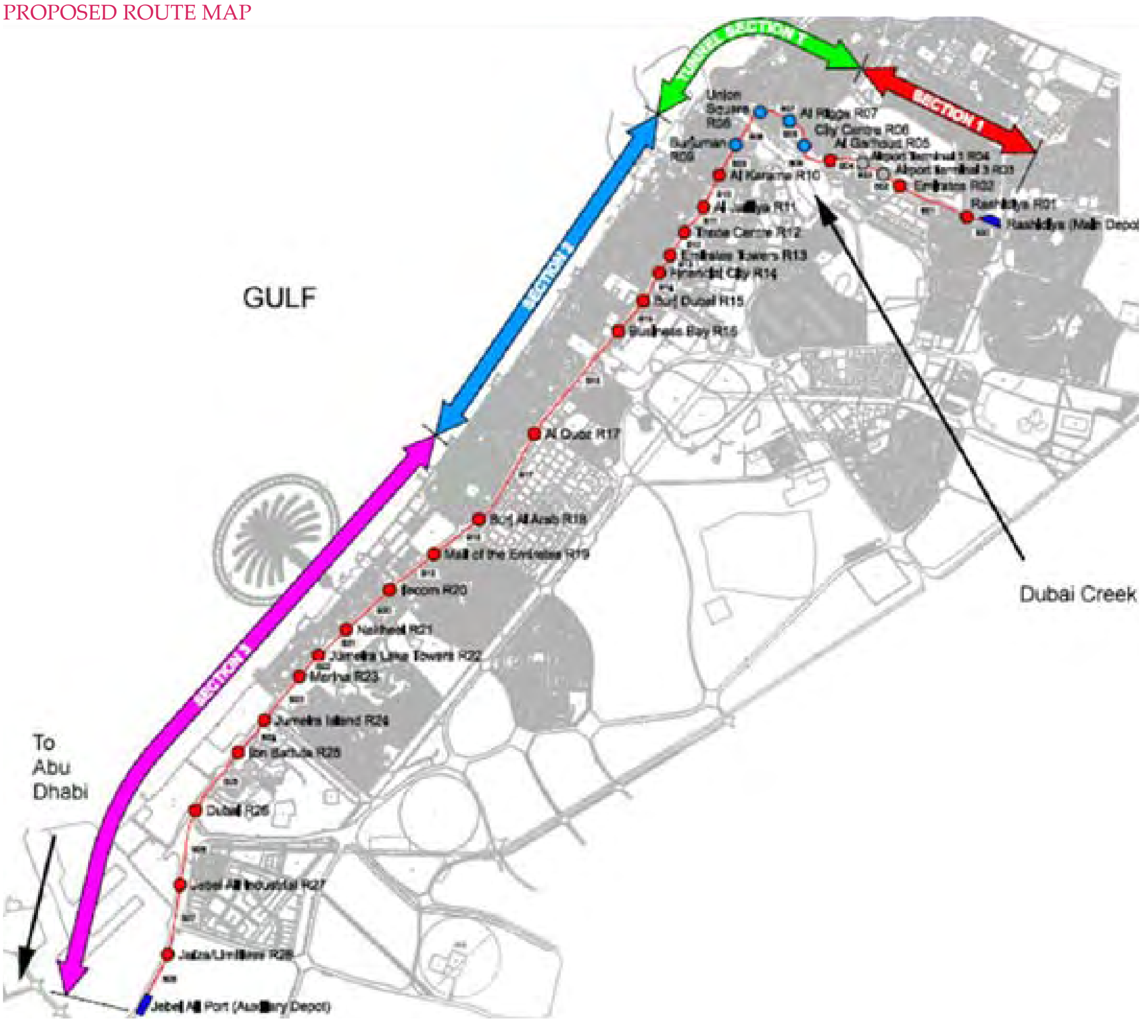

FIGURE 3
RED LINE ROUTE MAP

Design of the Dubai Metro Light Rail viaducts - Substructure

The £1.5 billion, 52 kilometre long Red Line connecting Rashidiya to Jebel Ali port comprises 42 kilometres of elevated viaduct with 22 overground stations, 5.5 kilometres of tunnels with 4 underground stations, 2.5 kilometres of at-grade section and 2 depots. The £800 million, 24 kilometre long Green Line runs around the city centre connecting Festival City to the airport free zone and comprises 16 kilometres of elevated viaduct with 12 overground stations and 7 kilometres of tunnels with 8 underground stations (of which two are shared with the Red Line).

This paper discusses the design and construction of the viaduct substructure. A further paper[1] covers the design and construction of the viaduct superstructure.

FIGURE 4
CONCEPTUAL VIADUCT FORM

FIGURE 5
TYPICAL SECTION OF U-TROUGH DECK AND PIER HEAD

FIGURE 6
PROTOTYPE DECK SEGMENT ILLUSTRATING TYPICAL SECTION AND FINISH

2. Viaduct form

The proposed form of the viaduct was architecturally-led in appearance. Figure 4 gives an artist's impression of the proposed viaduct at the conceptual design stage and much of this form has been retained in the final detailed design.

The viaduct superstructures were typically formed from U-shaped cross sections as illustrated in Figures 5 and 6. The post-tensioned precast segmental deck segments were cast using either long line or short line moulds.

The following superstructure forms, all of post-tensioned segmental construction, comprise the majority of the scheme:

- 1-span (single span) decks – Simply-supported U-section decks constructed by the span-by-span method from an overhead gantry
- 2-span (twin span) continuous decks – U-section decks constructed by the span-by-span method from an overhead gantry and made continuous over internal supports by subsequent in situ concrete stitching of adjacent decks
- 3-span continuous decks – Comprising a combination of U-section and box-section decks, erected by crane using the balanced cantilever method
- Station spans – 3- or 4-span continuous U-section decks constructed by the span-by-span method from an overhead gantry, subsequently made continuous over internal supports by stitching adjacent spans together
- Single track decks – Simply-supported U-section decks constructed by the span-by-span method from an overhead gantry (similar to 1-span decks). These decks are used at depots and bifurcations of the main lines at the largest stations

The viaduct substructures generally comprise reinforced concrete piers, with flared pier heads to support the deck, and reinforced concrete abutments. Pier heads for the single spans, twin spans and station spans were constructed using thin precast reinforced concrete shells which were infilled with in situ concrete and prestressed in stages once erected on site. The pier heads for the single track and 3-span continuous internal piers are of in situ reinforced concrete construction. All piers and abutments are founded on large diameter bored piles.

3. Piled foundation details

Dubai lies directly in the Arabian Desert and much of the geology comprises fine sand overlying sandstone and mudstone. The fine, upper sand layers consist mostly of crushed shell and coral and are a combination of mobile dune sands and sabkha deposits. These overlie calcarenitic, Aeolian deposits (sands, weakly cemented sands and weak sandstone) and calcisilitite. Beneath these, between around 20 metres and 40 metres below existing ground level, are Jurassic conglomerates, mudstones and siltstones.

The geotechnical design parameters were derived by Atkins' dedicated geotechnical team in Dubai and agreed with the Engineer's representatives locally. Results of the extensive ground investigations and their interpretation were collated into several ground reports for typical sections along the full metro route. The geotechnical design parameters in the soil and rock layers were derived from SPT-N values and unconfined compressive strength (UCS) respectively.

The vast majority of the viaduct spans are supported on single circular reinforced concrete columns with flared pier heads to support the decks, although a few portal structures are used in specific locations. Most single columns are supported on 2.2m and 2.4m diameter bored monopiles for speed of construction and to minimise the footprint required for excavations in the congested urban environment. Figure 7 shows a typical monopile detail.

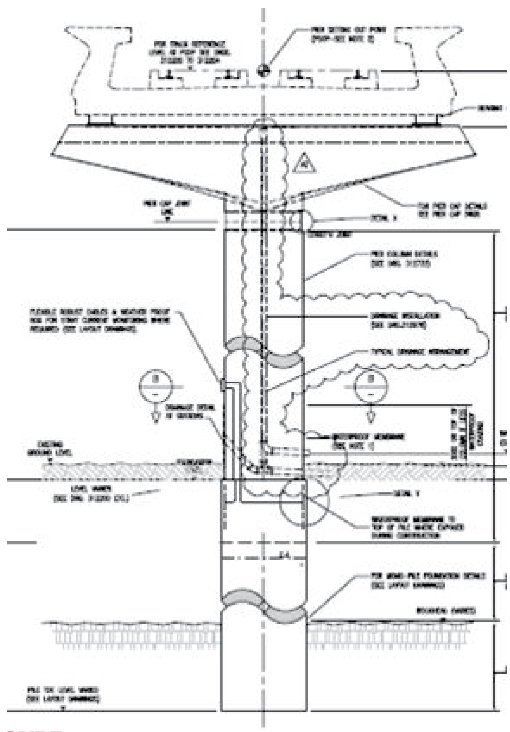

FIGURE 7
TYPICAL MONOPILE ARRANGEMENT FOR SIMPLY-SUPPORTED SPANS AND FINISH

STRUCTURES

FIGURE 8
CONSTRUCTION OF TYPICAL PIER ON A MONOPILE FOUNDATION

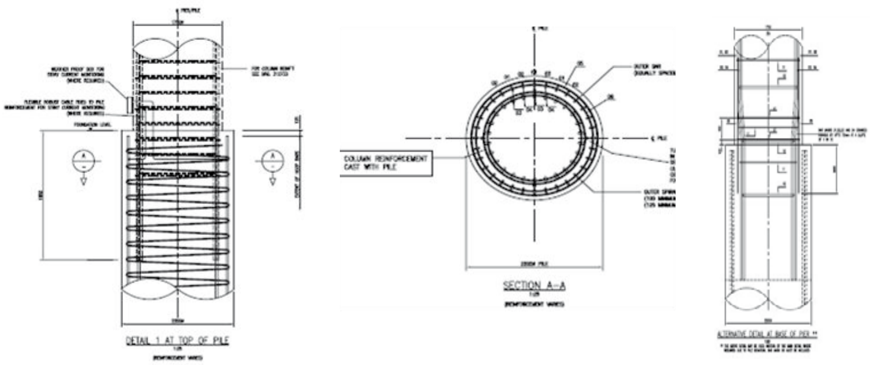

FIGURE 9
TYPICAL PIER-PILE CONNECTION DETAILS

The use of large diameter piles suited the interface with the circular piers. Typically diameters of 1.75m or 2.0m were used for the piers supported on 2.2m diameter piles, and 2.2m for piers supported on 2.4m diameter piles. The connection detail between the pile and pier was constructed like that for a pile cap; the pile was broken down and pier starter bars introduced, making allowance for piling tolerances. Figure 8 shows a typical monopile foundation under construction with the column starter bars in place and Figure 9 illustrates the alternative cranked reinforcement connection details that were adopted depending on the relative sizes of pier and pile and the percentage reinforcement content.

The piles needed to be large enough to resist the significant moments that are generated from lateral seismic loading (see Sections 6 and 7) and from out-of-balance forces from the deck due to horizontal alignment curvature, wind loading, eccentric train loads and other effects. These moments increase down the length of the pile towards a peak at the effective point of fixity and reinforcement was provided and curtailed to suit the specific force and moment envelopes generated from a range of load cases for each foundation.

Critical to the use of monopile foundations was the calculation of pile length. For foundations with only a single pile, these needed to be suitably conservative as there is clearly no potential for load distribution between adjacent piles as is possible within a pile group.

The original design basis proposed an allowable pile working load (Qallow) as the sum of the shaft resistance (Qs) divided by a factor of safety of 2 and the end bearing resistance (Qb) divided by a factor of safety of 3. The final resistance calculation method, as agreed with the Engineer, uses Qs / 3 and ignored end bearing due to the potential problems that could arise if a monopile were to bear into a local void in the weak sandstone or mudstone layers. Initial pile testing indicated that twice the calculated ultimate shaft resistance and 50% of the calculated ultimate end bearing resistance would give good agreement to the test results and thus potentially a more refined Qallow = Qs + Qb / 2.5 might have been adopted, but the design was completed using the more conservative calculation method previously approved. Pile shaft resistances were calculated using the following relationships depending on soil type and pile construction method:

- 1.6×*SPT-N* + 6 kPa (after Decourt[2]) for soils
- k×(*UCS design value*)$^{0.5}$ (after Zhang and Einstein[3]) for rock, with k = 0.35 for polymer modified water supported pile shafts and *k* = 0.25 for bentonite supported pile shafts as minimum values.

4. Design methodology for ULS and SLS

The design of the viaducts was based on BS 5400:Part 4[4] and associated British Standards, with additional International Standards used to supplement the scope in such areas as seismic loading and detailing, and rail dynamic factors. The American Concrete Institute technical design standard ACI 358.1R-92[5] was used to determine the dynamic factors to be applied to the vertical train loading for deck longitudinal design for the continuous spans. For the simply supported spans, the dynamic factors were derived from bespoke dynamic analyses for the respective span lengths. For transverse design, the recommended dynamic factors of BS 5400:Part 2[6] for RL loading of 1.2 to 1.4 were verified again using a finite element dynamic analysis. In accordance with the ACI code, the dynamic impact factors were not applied to the design of viaduct foundations, but were included in the pier head and bearing design. The maximum operating speed of the trains is intended to be 90 kph; the maximum design speed was taken as 100 kph.

The usual BS 5400 load combinations from 1 to 5 were assessed to determine critical design load effects. In addition, a sixth load combination was added to cover seismic loading (see Section 6). Other specific load cases considered included temporary loading from gantries (Section 5) and vehicular collision.

Typically, class C32/40 was used for the pier and pile cap concrete. Class C32/40 concrete was also used for the piles, but to account for possible weakening during the placement (under bentonite or polymer modified suspension fluid) the cube strength design value used was reduced by 10 MPa. All reinforcement used in the substructure design was high yield, type 2 deformed bars with a yield stress of 460 MPa. The aggressive ground conditions meant that durability considerations were paramount. As a result, additional waterproofing was applied to the top 5 metres of all piles to improve resistance to chloride attack and pile cover to reinforcement of 120 mm was used to improve resistance to sulphate attack. Generally crack widths (under combination 1 loads) were limited to 0.2 mm. Fatigue of reinforcement was also a critical design consideration in some locations (see Section 8).

The construction programme called for the initial design of over 1200 unique foundations in the first 9 months, to take the viaduct construction off the critical path. This was achieved through automation of the bulk of the design process and the use of conservative simplifying assumptions in the early stages of design. As the team got ahead of the programme, the conservatism was removed from the process and more refined calculation methods introduced into the automated procedures to optimise the designs for the foundations yet to be constructed.

The design programme capitalised on the locations of the UK-based structures team and the Dubai-based alignment team, which handled setting-out and local issues such as utility diversions. Advantage was taken of the staggered weekends between Dubai and the UK; the alignment of a given section was distributed at the start of the UK week and the appropriate design data added and sent back to Dubai at the end of the UK week. Coupled with the automation process developed, this allowed a peak output of 100 bespoke designs per week to be achieved.

5. Design during construction – gantry loading

The majority of the simply supported decks and 2-span continuous decks were constructed by overhead gantries (illustrated in Figures 10 and 11). The temporary loading from the various gantries used on the scheme was defined by the temporary works subcontractor, VFR (a consortium comprising VSL, Freyssinet and Rizzani De Eccher), appointed to undertake the deck construction. These loading regimes were continually developed throughout the design programme, as various configurations of gantry were developed to cater for the many permutations of span configurations and access restrictions on site.

The gantry loads included the effects of the most severe loading configuration carrying deck precast elements and also the unloaded case, when the gantry was potentially subject to stronger winds. In some locations, the gantries were also required to travel over previously constructed (by the balanced cantilever method) 3-span continuous decks and the temporary effects of these conditions needed to be designed for. Precast deck segments were mostly delivered to their required location at ground level, but in some locations, where access was more difficult, some segments were delivered over the previously constructed deck using special transporters. The additional load effects from these cases on permanent works also needed to be considered in the detailed design.

For the substructure, the temporary construction load cases were generally not governing for the pier and pile designs as they were typically less onerous than the seismic design effects. The design of the pier heads however, was extremely sensitive to the gantry loads (and in particular to the torsion induced) as discussed in Section 9 below.

FIGURE 10
USE OF OVERHEAD GANTRIES TO CONSTRUCT TYPICAL VIADUCT SECTIONS

FIGURE 11
USE OF OVERHEAD GANTRIES TO CONSTRUCT TYPICAL VIADUCT SECTIONS

6. Seismic design methodology and detailing

Seismically, Dubai is in a very stable zone. The nearest seismic fault line, the Zargos Fault, is 120 km from the UAE and is unlikely to have any seismic impact on Dubai. Nevertheless, the Employer's Requirements specified that the viaducts should be designed for earthquakes in accordance with AASHTO LRFD[7], classing the viaducts as "essential", the site as Zone 2 and using an acceleration coefficient of 0.12. The site coefficients were determined in accordance with AASHTO LRFD on the basis of the relevant geological profile and geotechnical data for the foundations. Soil profile types I and II were found to be appropriate for the entire route, giving site coefficients of 1.0 and 1.2 respectively (modified to 1.0 and 1.5 respectively where the flexibility of the elastomeric bearings was included in the seismic analysis).

AASHTO LRFD was only used to determine the load effects from a seismic event. Once obtained, these effects were combined with the other appropriate coexistent load effects and the foundations designed in accordance with the resistance rules of BS 5400:Part 4 as for other non-seismic load cases.

In general, the single mode elastic method was used and the fundamental period of vibration was determined by modelling individual piers using the computer software *LUSAS*[8] as described in Section 7 below. Multimodal analyses were also used for continuous structures and irregular single span arrangements (see Section 7). The horizontal elastic seismic response coefficient for a given natural period of vibration, C_{sm}, was obtained from AASHTO LFRD:

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \leq 2.5A$$

where T_m is the period of vibration of the m[th] mode (in seconds), *A* is the acceleration coefficient and *S* the site coefficient.

There were no contract requirements for seismic serviceability performance, so columns were designed to form plastic hinges at their bases under a seismic event at the ultimate limit state. Potential plastic hinge zones were confined to the base of the piers and the top of the piles so that the substructure could be readily inspected for damage after an earthquake. Plastic hinges were promoted in these regions by dividing the elastic seismic design forces by appropriate response modification factors, *R*, for the respective elements (based on the provisions given in AASHTO LRFD) with:

- *R* = 1.5 for elastomeric bearings where account of the bearing flexibility had been included in the natural frequency analysis
- *R* = 2.0 for continuous structures with mechanical bearings

The onset of plasticity controls the magnitude of forces that can be transmitted to the rest of the structure, which is particularly beneficial in the event of an extreme earthquake whose magnitude exceeds the design value. However, over-strength of the hinge zone in bending could lead to greater forces being attracted than expected. For such cases, it is important that the piers have adequate shear strength to enable a ductile response to develop, rather than permit a brittle shear failure to occur. To guard against such failure, the piers were designed for a shear force that corresponds to the achievement of the over-strength moment of resistance in the plastic hinge zone at the base of the piers. AASHTO recommends using an over-strength factor of 1.3 to account for the difference between mean material properties and 5% lower fractile values. In addition, further over-strength is provided because of the use of partial material factors. These need to be removed to determine the potential difference in strength between design values and realistic in situ values. The over-strength factor to account for the removal of the material partial factors will be between the values of 1.15 and 1.5 (the BS 5400:Part 4 values for steel and concrete respectively). For the typical column reinforcement percentages used for Dubai Metro, this factor was approximately 1.22.

To ensure that the plastic hinge zone is confined to the base of the pier, the flexural design of the pier above the plastic hinge was based on bending moments which are consistent with the over-strength moment of resistance in the plastic hinge zone and the corresponding over-strength shear force. Superstructure-pier connection forces were based on the lesser of the elastic seismic design forces divided by a response modification factor, *R*, of 1.0 or the shear force that corresponds with the over-strength moment of resistance in the plastic hinge zone at the base of the pier. This was applied to the bearing design too.

Reinforcement detailing for the viaducts was generally in accordance with BS 5400:Part 4. However, special additional requirements for seismic detailing were taken from AASHTO LRFD. In particular there is a need to ensure that the piers have adequate ductility as discussed above. This was achieved by providing adequate transverse reinforcement in the potential plastic hinge zones, to prevent buckling of the longitudinal reinforcement and to provide confinement to the concrete core. This confinement reinforcement was continued into the body of the pile caps, where used, in accordance with the code. Transverse reinforcement could be formed by either spiral or hoop reinforcement, provided that conventional laps were not used in the plastic hinge zone and hoop reinforcement was anchored into the body of the column with adequate hooks. The latter would have given increased congestion and reduced space for concrete vibrators, thus spiral reinforcement was used for the majority of foundations. Lapping of longitudinal reinforcement was also kept outside the potential plastic hinge zones so as not to compromise ductility and over-strength shear and moment design away from the plastic hinge zone.

7. Seismic analysis

The seismic analyses of the straight sections of simply supported spans were carried out in accordance with AASHTO LRFD for multi-span bridges with "regular" spans provided they met the span ratio and pier stiffness criteria even though the number of spans exceeded six.

The single mode elastic analysis uniform load method was used. Other structures were designed using a multimodal response spectrum analysis in which the presence of the rails was ignored.

All of the seismic analyses were performed using several finite element beam and shell models analysed in *LUSAS*. The models included the pile supports either with equivalent cantilevers or complete piles with soil springs. Equivalent cantilevers were determined from the results of the soil-structure interaction analyses. This was an iterative process to ensure that the stiffness derived was appropriate to the mean load level. Accelerations were determined using the seismic response spectrum defined in AASHTO LRFD. The site coefficients used in the response spectrum definition were dependent on the bearing type used as noted in Section 6.

Initially, parametric studies of the single span models for the simply-supported structures were developed for a range of variables to enable rapid automated design of individual, unique foundations. Variables included pier diameters, pier heights, soil properties, span arrangements, bearing types (from rigid mechanical bearings to a range of elastomeric bearing stiffnesses) and acceleration direction were considered. Results for a typical set are illustrated in Figure 12, where "type 1" and "type 2" refer to two different elastomeric bearing designs. The beneficial effect of introducing flexible bearings on reducing natural frequency is apparent.

Macros were developed to automatically write the model file data for an individual pier, enabling further quick refinement of the design to be undertaken which was essential to the rapid re-design of sections of the alignment subject to change.

For the multimodal analyses, the seismic load effects from each mode were summed using the complete quadratic combination. These results were obtained directly from the *LUSAS* output and verified against simpler hand calculations based on the most dominant modes. Directional combination of effects was achieved using a simple combination rule whereby 100% of the response in any one direction was considered with 30% of the response in the other two orthogonal directions.

8. Fatigue of cranked reinforcement

Due to limitations on maximum pile sizes, reinforcement content was particularly high for the most heavily loaded monopiles. In several cases, two layers of longitudinal reinforcement were required. This called for an additional detail to cater for piling tolerances and led to the cranking of longitudinal bars (as indicated in Figure 9) in some instances. The high centrifugal loads on the tallest, slender piers resulted in particularly high fatigue stress ranges in the cranked bars.

It is commonly appreciated that bent bars have lower fatigue resistance than straight bars but design codes give limited guidance on their design. BS 5400:Part 4[9] covers fatigue of reinforcement inadequately by limiting stress ranges to 325 MPa and does not distinguish between bent and straight bars. The Highways Agency document BD249, as amended by Interim Advice Note (IAN) 5, also fails to distinguish between straight and bent bars and its limiting stress ranges are inappropriate for rail loading; they are based on calibration studies using static highway loading. As a result of these deficiencies, Eurocode EN1992-1-1[10] was used for the fatigue assessment because it contains rules for bent bars with different bend diameters. For the typical bar arrangements used for Dubai Metro, the fatigue stress range limits are 116.5 MPa for straight bars and around 65 MPa for the cranked bars, which constitutes a large reduction in effectiveness. This predicted magnitude of reduction in fatigue performance for bent bars has actually been shown to occur in tests[11]. These limits were used for all the substructure reinforcement throughout the project.

In order to comply with these relatively low permissible limits, the fatigue stress ranges were calculated from the likely mix of actual train loads rather than from the full characteristic loading used in the static design. For the predicted 20 million load cycles within the design life, the latter would be too onerous. Realistic operating train speeds were also used in the calculation of centrifugal forces, rather than the enhanced design speed used for ultimate limit state calculations.

The above approach was enough to justify the use of cranked bar details, apart from one or two worst cases where separate, bespoke details were developed without bending the bars.

9. Pier head design

The design and construction of the pier heads was subcontracted by JTMjv to ETiC, but Atkins retained the design responsibility and was instrumental in developing the final design solutions to satisfy the particularly onerous temporary load conditions. The design interfaces were complicated further with close coordination required between the designers, the bearing suppliers and VFR, who were responsible for the supply of gantry loads and erection sequences. Iteration with the bearing design was subsequently simplified by landing the decks on temporary supports before jacking onto permanent bearings after the gantries had moved on to erect other spans (see section 10 below).

The appearance of the pier heads was architecturally led (see Figure 13) and required precast outer shells to achieve the desired finish consistently. The outer shells were cast using steel moulds and stored with the deck segments in the dedicated casting yard at Jebel Ali. The outer shells comprised thin-walled, reinforced concrete sections which could be transported to site when required, before being infilled with in situ concrete and prestressed in controlled stages on site as the permanent load was added.

Designed initially for the permanent condition only, the pier heads had several shear and torsion deficiencies identified under construction load cases due to the high eccentricity of the temporary gantry loading. In this temporary condition, all the self weight of both spans is applied on one side of the pier head through temporary supports (Figure 14). In the final condition, the load from each span is more balanced.

For the temporary condition, the maximum shear and torsion could not be resisted by either the shell or the in situ infill concrete alone and thus the pier head had to be designed to mobilise a contribution from both the shell and the infill, despite minimal interface reinforcement being provided between the two.

To achieve an adequate design without modifying the pier head moulds, which had already been fabricated, several departures from BS 5400:Part 4 were required. In particular, the inclined components of prestressing force and compression chord force were included in the shear resistance, the tensile SLS stresses during construction were limited to Class 2 limits, rather than the.1.0 MPa limit stipulated, and stresses during bearing replacement were limited to Class 3 limits following agreement to this relaxation by the client.

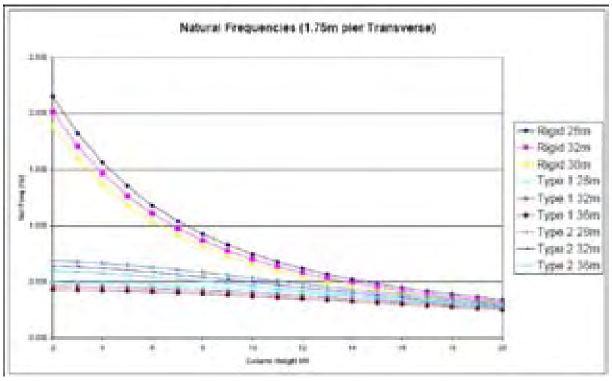

FIGURE 12
TYPICAL NATURAL FREQUENCY ANALYSIS RESULTS FOR A RANGE OF PIER HEIGHTS, BEARING TYPES AND SPAN ARRANGEMENTS

FIGURE 13
TYPICAL PIER HEAD ARRANGEMENT

FIGURE 14
HIGH ECCENTRICITY OF TEMPORARY LOADING FROM OVERHEAD GANTRIES

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10. Bearing design

The majority of the single-span decks are supported on elastomeric bearings. Pot bearings are used for all the continuous structures for both internal supports and end supports and therefore also the ends of the single spans sharing pier heads with continuous spans.

The philosophy for the design of the elastomeric bearings was complicated by two opposing design objectives. The bearings needed to be stiff enough to comply with the continuous rail deflection limits (of International Union of Railways technical standard UIC 776-3[12]) and strong enough to withstand the vertical and horizontal loads. However, they needed to be flexible enough to minimise the seismic forces attracted to the substructure since lower stiffness leads to reduced natural frequencies and reduced lateral accelerations. The range of adequate elastomeric bearings that could satisfy these requirements was particularly narrow and also very sensitive to span arrangements.

Typically, bearings of around 600mm by 650mm to 750mm by 750mm in plan and 150mm to 210mm tall were adopted. The elastomeric bearings were generally designed in accordance with BS 5400:Part 9[13], but this only gives guidance on design at the serviceability limit state and does not cover ultimate limit state design which was required for the usually critical seismic load cases. The similar design rules given in BS EN 1337-3[14] were therefore used to check the adequacy of the bearings at ULS.

For speed of construction and simplicity, the contractor was keen to avoid any mechanical fixing of the elastomeric bearings to the deck. Consequently the bearings were designed to be placed dry between the deck and the pier head plinths and to rely on friction as the sole means of lateral restraint. Where transverse forces on the elastomeric bearings exceeded 10% of the vertical load (as occurred for all locations), the bearings were fitted with an interfacing chequered plate to provide a minimum coefficient of friction of 0.5 between mating surfaces. This attachment was capable of carrying the entire transverse load.

Since reliance was placed on friction as the sole means of restraint against lateral seismic loads, a coexistent vertical acceleration was considered in the design of the elastomeric bearings, despite not being specified in the Employer's Requirements. This acceleration was derived from a response spectrum similar to that used for the horizontal accelerations, but with values equal to two-thirds of the horizontal ones.

Because the design of the elastomeric bearings was tailored to the permanent condition, there was little capacity left to accommodate deflections, rotations and translations that would be locked into the bearings due to the use of launching gantries and the construction sequences adopted. Consequently, the majority of the decks had to be landed on temporary supports during the initial erection by gantry before they could be subsequently jacked up onto their permanent bearings when the gantry had moved on. Although not ideal for construction, this solution had benefit in reducing the temporary load effects in the pier heads and was thus adopted as a compromise to overcome several problems.

11. Rail-structure interaction

As discussed above, the viaducts comprise a combination of simply supported spans on elastomeric bearings and continuous spans with fixed and free guided sliding pot bearings. The rails are continuously welded across all the decks and deck joints and are connected to the deck by regular track fixings, thus the rails interact with the deck causing relative movements to occur which induce stresses in the rails and forces in the deck.

The temperature range of the continuous welded rail (CWR) was considered relative to its neutral setting temperature of 40°C ±3°C and the maximum and minimum rail temperatures which were assumed to be +75°C and +3°C respectively. This gave CWR extreme ranges of +38°C and -40°C. The rail-structure interaction (RSI) temperature range is governed by the change of structure temperature relative to deck temperature at the time of installation of the rail. Based on the specified design temperature ranges, the maximum and minimum deck temperatures were assumed to be +55°C to +5°C. It was further assumed that the rails are fixed to the deck at deck temperatures between +20°C and +40°C which gave maximum and minimum temperature ranges of + 35°C and –35°C. This corresponds with the International Union of Railways technical standard UIC 774 3R[15] requirements of maximum and minimum bridge temperatures ranges of ±35°C. The build up of CWR forces in the rail was considered at points where rail breather joints are located and for the case of a rail break. For viaducts with horizontal curvature the effects of the radial forces arising from the full CWR forces were considered.

The rails were checked against the recommendations of UIC 774-3. These requirements limit the stress ranges due to this RSI behaviour to prescribed values. The checks are only necessary for in-service conditions, thus no analysis of rail forces was undertaken for seismic events.

Checks were also made on the additional stresses induced in the rails, due to the end rotations of the decks and the differential vertical and transverse movements permitted due to the flexibility of the bearings. The values of these additional forced rail displacements and rotations were considered by the trackwork designers in the final rail design.

The rail-structure interaction behaviour is non-linear because the track fixings allow slip to occur with an elasto-plastic relationship illustrated in Figure 15. The decks also interact with each other through the continuous rails and the rail connections and so the overall global behaviour is complex. The effects can be rationalised however, such that simplified approximate methods can be used initially to identify the most critical viaduct lengths which required further detailed analysis.

The global behaviour was analysed using 2D models of both the structure and rails to examine the longitudinal load distributions. The rails were modelled as beam elements with non-linear springs connecting them to the deck. The out of balance effects of different adjacent span lengths were analysed using a simple elastic 2D model of at least 5 spans either side of the design pier in order to quantify the forces on the bearings and piers. This approach was also used for vertical load effects, seismic loading and traction and braking.

A series of simplified spreadsheet approaches to estimating the rail-structure interaction forces and for checking compliance against the UIC code requirements was developed. Where the simplified checks showed borderline cases, the more sophisticated non-linear analysis was used to demonstrate adequacy. In some instances, this required the flexibility of the fixed piers and their foundations to be included in the non-linear models.

Changes to the overall viaduct setting-out continued throughout the construction process where previously undisclosed, existing utility locations were identified requiring span arrangements to be adjusted. The development of the simplified approaches to RSI analysis enabled a rapid decision to be obtained on the impact such setting-out variations had on the overall RSI conclusions, without the need to undertake more complex analysis every time the span arrangements altered.

In addition to the RSI analysis, absolute and relative displacement checks were carried out against the UIC 774-3 requirements for braking, acceleration and deck

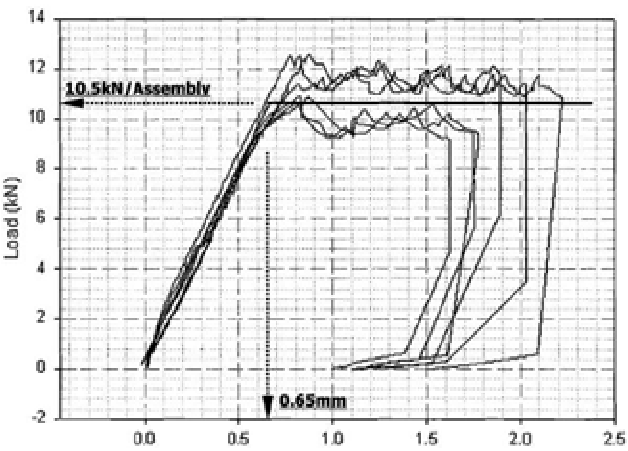

FIGURE 15
TRACK FIXINGS SLIP CHARACTERISTICS

end rotation due to vertical loading. The relative vertical deflection across adjacent decks at rail locations was limited to 3mm.

12. Acknowledgements

This paper is published with the permission of the Dubai RTA and Naramichi Oba san of the JTMjv. The authors would also like to acknowledge Atkins Project Director, John Newby, and his management team for their excellent coordination of the numerous design parties.

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The implications of the change to Eurocodes for bridge design

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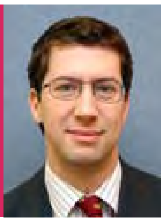

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Abstract

The use of Eurocodes on real projects in the UK highways sector is nearing reality. National Annexes for all the parts required for bridge design are nearing completion and should be ready by March 2008. The Highways Agency and BSI committees are completing production of BSI PD documents to give non-contradictory complementary information to help with the use of the Eurocodes and a variety of organisations are producing design guides.

Despite the cultural and technical changes that the Eurocodes bring, pilot studies have shown that designers adapt remarkably quickly. There are also incentives to embrace the Eurocodes. The less prescriptive approach and more up-to date rules provide the user with greater scope for innovation and economy. This paper identifies the various areas in steel and concrete design where there are technical differences between the new Eurocodes and BS 5400 and also where the new rules lead to the potential for greater structural economy.

1. Introduction

The use of Eurocodes on real projects in the UK highways sector is nearing reality. National Annexes for all the parts required for bridge design are nearing completion and should be ready by March 2008. The Highways Agency and BSI committees are completing production of BSI PD documents to give non-contradictory complementary information to help with the use of the Eurocodes and a variety of organisations are producing design guides.

There are 58 Eurocode parts. Steel-concrete composite bridge design is the activity requiring the largest fraction of these as shown in Figure 1. Additionally, the UK National Annexes will be required for each part. Whilst this might seem daunting to those with limited knowledge of the Eurocodes, pilot studies by consultants have shown that Eurocodes 2, 3 and 4 are simply different from BS 5400, not more difficult to use.

Eurocode 3 looks particularly daunting with its 17 parts, but Eurocode 4 has a more familiar feel to it while Eurocode 2 is a much more comprehensive document than BS 5400 Part 4.

The individual designer will notice some cultural differences when first using the Eurocodes. Engineers will need to make greater use of first principles, as fewer rules and formulae are given. In many cases, this will lead to greater use of finite element modelling. There is inevitably some new terminology and the term "Eurospeak" has been coined by some to describe this. However, initially perplexing requirements such as "verify web breathing under the characteristic combination of actions" will quickly become second nature if the results of pilot studies are anything to go by. The remainder of this paper describes some of the differences between the requirements of the steel and concrete Eurocodes and existing UK practice.

2. Steel and steel-concrete composite design to EN 1993-2 and EN 1994-2

Material properties

Unlike EN 1992, where designers need to get used to using cylinder strength rather than cube strength in calculations, there is little of any great difference to contend with in EN 1993 with respect to material properties. Yield strength varies with plate thickness in the same way as it does when using BS 5400:Part 3, and limiting plate thicknesses to prevent brittle fracture come out very similar to those obtained from BS 5400:Part 3, although the calculation route is a little different.

Some additional guidance is provided in EN 1993-1-10 as to when to specify steel with improved through thickness properties (Z quality steel to BS EN 10164) in situations where a plate can be loaded through its thickness.

These rules have however been universally panned by the UK steel industry; not for being incorrect, but because they tend to encourage specification of expensive steel, rather than consideration of better detailing. This is unusual for the Eurocodes, which usually promote greater thinking in design. Clearly if the bridge is of concrete-composite construction, then there is the need to get to grips with cylinder strengths in calculation.

Global analysis

There has been concern expressed about the increase in complexity of global analysis to EN 1993 and EN 1994. It is true that the default analysis in the Eurocodes is second order, considering *P*-Delta effects. However, the exceptions where second order analysis need not be used and where first order analysis will suffice are such that in almost all situations where first order analysis was used in previous practice to BS 5400, it can still be used.

FIGURE 1
EUROCODE PARTS NEEDED IN THE COURSE OF THE DESIGN OF A STEEL-CONCRETE COMPOSITE BRIDGE AND THE NEAREST EQUIVALENT BRITISH STANDARDS

EUROCODE PART	EQUIVALENT BS 5400 PART
EN 1990 Basis of structural design	**BS 5400 Part 1 and 2**
ACTIONS EN 1991-1-1 Densities, self weight and imposed loads EN 1991-1-4 Wind loads EN 1991-1-5 Thermal loads EN 1991-1-6 Actions during execution EN 1991-1-7 Accidental actions EN 1991-2 Traffic loads on bridges	**BS 5400 Part 2**
CONCRETE EN 1992-1-1 General rules and rules for buildings EN 1992-2 Bridges	**BS 5400 Part 4**
STEEL EN 1993-1-1 General rules and rules for buildings EN 1993-1-5 Plated structural elements EN 1993-1-8 Design of joints EN 1993-1-9 Fatigue EN 1993-1-10 Brittle fracture EN 1993-2 Bridges	**BS 5400 Part 3**
STEEL-CONCRETE COMPOSITE EN 1994-2 General rules and rules for bridges	**BS 5400 Part 5**

The effects of local plate buckling and joint flexibility theoretically need consideration in global analysis but, once again, the effects have to be so severe before they require inclusion that generally they can be ignored. Similarly, shear lag needs to be considered but frequently will not lead to any actual reduction in acting flange width. For composite bridges, the effects of cracking on stiffnesss in global analysis are treated in almost exactly the same way as in BS 5400:Part 5.

Section classification

The section classes employed in EN 1993 will look familiar to UK building designers but less so to bridge designers in that there are four section Classes:

- Class 1 cross sections can mobilise a plastic bending resistance and have enough rotation capacity to permit plastic global analysis;
- Class 2 cross sections can mobilise a plastic bending resistance but have insufficient rotation capacity to permit plastic global analysis;
- Class 3 cross sections can achieve a bending resistance corresponding to first yield;
- Class 4 cross sections can achieve a bending resistance corresponding to plate buckling in compression somewhere at a stress below that of yield.

Class 1 and 2 cross sections were previously referred to as "compact" in BS 5400:Part 3. The distinction between them is lost somewhat for bridge design to EN 1993-2 as plastic global analysis is not allowed, other than for accidental combinations of actions. Class 3 and 4 were previously referred to as "non compact" in BS 5400:Part 3, although a similar distinction within "non-compact" existed but was hidden; the equivalent to a Class 4 cross section was essentially one where a reduction was needed to the web thickness in stress analysis. The boundary between compact and non-compact behaviour remains very similar to that in previous UK practice.

Shear lag in cross section design

One significant conceptual difference between EN 1993 and BS 5400 is that shear lag must be considered at the ultimate limit state. Previous UK practice was to ignore this at ULS on the basis that the occurrence of plasticity led to redistribution of stresses across the cross section. The approach in EN 1993-1-5 is to require consideration of shear lag at ULS, but to make explicit allowance in calculation for this plasticity. Consequently, different effective widths are used for calculations at ULS and SLS. Initially, those keeping an eye on the drafting of EN 1993 in the UK objected to the possible loss of economy implicit in using a reduced flange width at ULS, but trial calculations showed that most typical bridge geometries would achieve a fully effective flange at ULS in any case. The difference between effective widths to be used at SLS and ULS is illustrated in Figure 2 for the fraction of flange width acting at an internal support of a continuous steel beam with span lengths L and flange width bo available each side of the web.

Shear lag must also be considered for concrete flanges of steel-concrete composite beams, but the same effective width is used for SLS and ULS for simplicity. This has little implication for bending resistance as there is usually an excess of available compression concrete. There can however be benefits in using a smaller effective width in shear connection design as it can lead to fewer shear connectors being required.

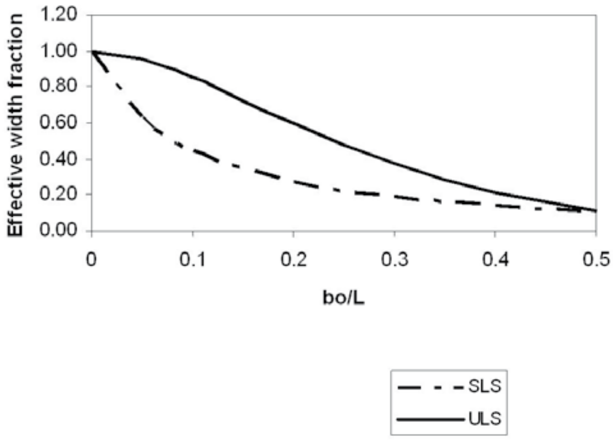


FIGURE 2
EFFECTIVE WIDTH OF UN-STIFFENED STEEL FLANGE AT AN INTERNAL SUPPORT

Cross section resistance for bending

For Class 1 to 3 cross sections, the calculation procedure for bending resistance will be found to be very similar to the design of compact and non-compact beams to BS 5400. One small difference for Class 3 cross section design is that where the bending resistance is based on first yield at an extreme fibre, EN 1993 defines an extreme fibre as the mid-thickness of the flange, rather than the outer surface. This gives a small increase in economy over BS 5400, particularly for shallow beams with thick flanges.

EN 1994 employs a similar rectangular stress block for concrete for plastic design as is used in BS 5400:Part 5, although the resisting compressive stress is slightly higher. The treatment of Class 4 cross sections and beams with longitudinal stiffeners (which are treated as Class 4 cross sections in EN 1993) differs significantly, however, from that in BS 5400. Class 4 beams without stiffeners are treated by making reductions to the compression areas and then checking stresses against yield when calculated on the resulting reduced cross section. The procedure for composite beams is first to calculate accumulated stresses on the gross cross section, following the construction sequence, and then to determine the effective areas of the compression elements based on this stress distribution. Finally, the accumulated stresses are recalculated using the reduced effective steel cross section at all stages of construction. This is illustrated in Figure 3.

Class 4 beams with longitudinal stiffeners are treated in the same way as beams without longitudinal stiffeners in EN 1993, unlike in BS 5400:Part 3 where a completely different approach to calculation was employed. In BS 5400:Part 3, individual panels and stiffeners are checked for buckling once stresses are determined in them, generally using gross cross sections other than for flange plates as shown in Figure 4. There is therefore limited load shedding between components and a single overstressed component can govern the design of the whole cross section.

In EN 1993-1-5, effective widths are again used to allow for buckling of web and flange elements, as for unstiffened Class 4 cross sections, but the same approach is also used for stiffeners. This effectively allows load shedding between all the various elements such that their combined strengths are optimally used. This represents a significant change from previous UK practice and can give rise to an increase in economy in the design of longitudinally stiffened cross sections. A typical resulting effective cross section is shown in Figure 5. Reference 1 gives greater background to these differences.

Shear buckling resistance

The rules for shear buckling in EN 1993 and BS 5400 are based on quite different theories but produce similar results. In the early 1970s, two contemporaries worked on the problem; Rockey in the UK and Höglund in Sweden. Rockey's theory was adopted in BS 5400:Part 3 but now, thirty years on, Höglund's theory is being used in EN 1993-1-5. The implications are, as previously stated, not great for the design of webs themselves, but are significant in the design of transverse stiffeners as Höglund's theory places less demand on their strength. This is reflected in EN 1993-1-5, which allows lighter transverse shear stiffeners to be designed than would be permitted to BS 5400:Part 3.

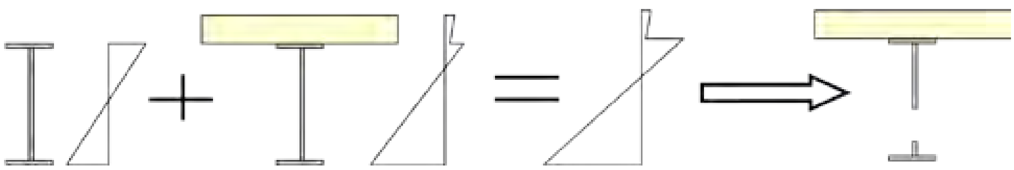
Shear – moment interaction

EN 1993 produces a more economic check of shear and moment interaction than does BS 5400 and consequently has been used in assessment to justify not strengthening existing bridges. It is more economic for three reasons:

- Shear does not interact with lateral torsional buckling resistance. It only interacts with cross section resistance;
- The interaction diagram is a continuous curve, rather than a series of straight lines as was the case in BS 5400:Part 3, as shown in Figure 6;
- Even if the cross section is in Class 3 or 4 (so that the bending resistance is limited to first yield), the interaction is performed using the plastic bending resistance. The interaction is truncated by the requirement to limit the moment to the elastic moment. This has the effect of permitting almost full web shear resistance with full bending resistance, as shown in Figure 6, which reflects the findings of recent non-linear parametric finite element studies.

Longitudinally stiffened cross sections are treated in essentially the same way in EN 1993-1-5 as for unstiffened cross sections, so the same economic benefits can be obtained. To BS 5400:Part 3, the check of the cross section would have to be performed on a panel by panel basis in such a way that any shear stress at all has the effect of reducing bending strength.

FIGURE 3
ILLUSTRATIVE PROCEDURE FOR DETERMINING EFFECTIVE CROSS SECTION IN CLASS 4 COMPOSITE BEAMS



(i) Accumulated stresses determined on gross cross section

(ii) Accumulated stresses used to determine an effective steel cross section for re-calculation of accumulated stresses

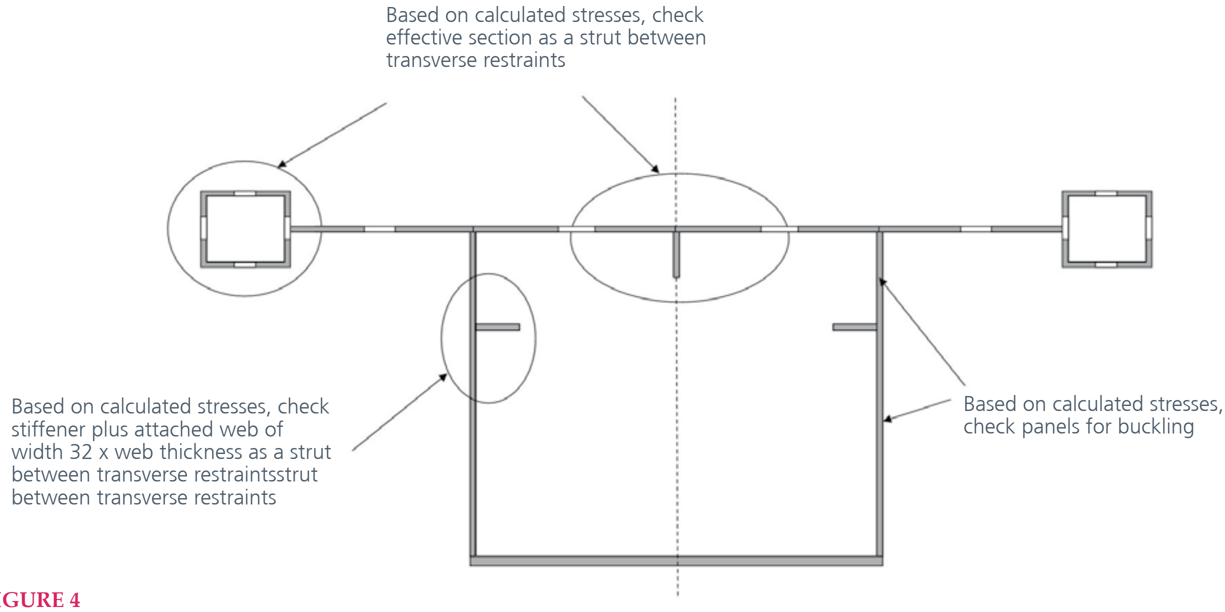


FIGURE 4
TYPICAL DESIGN APPROACH FOR
LONGITUDINALLY STIFFENED
BEAM IN BS 5400:PART 3

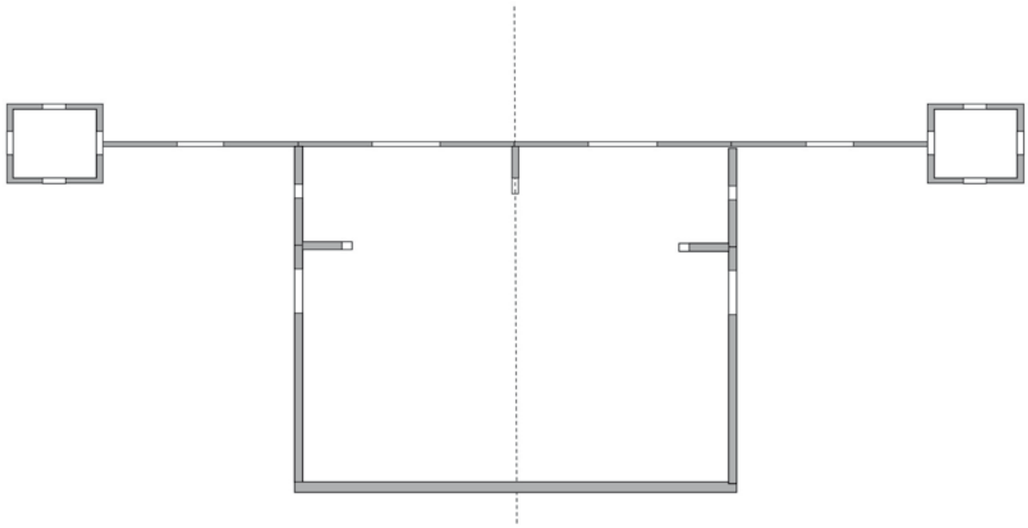


FIGURE 5
TYPICAL EFFECTIVE CROSS SECTION
FOR DESIGN TO EN 1993-1-5

Lateral torsional buckling and distortional buckling

Whilst BS 5400:Part 3 gave extensive empirical guidance on lateral torsional buckling, EN 1993 takes a more theoretical approach. EN 1993, as a general approach, gives only an expression for slenderness,

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

where M_{cr} is the elastic critical buckling moment. No guidance is given on the calculation of M_{cr} which tends to lead the designer towards performing a computer elastic critical buckling analysis for its determination. It is not, however, always necessary to do this. For steel-concrete composite members with the deck slab on top of the beams, where buckling is by lateral buckling of the compression flange, a simpler method is provided that avoids the need for this calculation. It is actually simpler to perform than any equivalent check in BS 5400 where the moment does not reverse between restraints. Where there is moment reversal, some interpretation is required, such as that provided in reference 2.

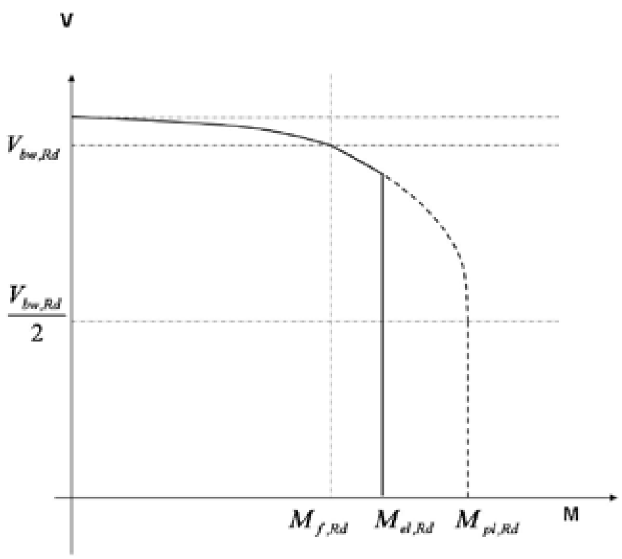


FIGURE 6
TYPICAL INTERACTION DIAGRAM FOR SHEAR AND MOMENT TO EN 1993-1-5

No hand calculation method is provided for the case of paired beams during concreting, prior to the deck slab providing restraint in plan to the beams. In this situation, recourse can be made to guidance documents, based on old BS 5400 practice, or finite element analysis undertaken. The latter will bring rewards, as the bending resistance so derived in accordance with the EN 1993 will be significantly greater than that to BS 5400:Part 3 2000. A typical global buckling mode derived from a finite element model is shown in Figure 7. With a bit of experience of creating such models, it can be quicker to perform than the calculation to BS 5400:Part 3, which requires a mixture of hand calculations and plane frame analysis.

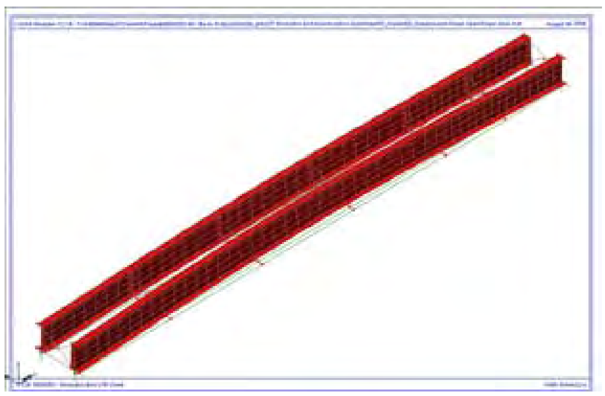


FIGURE 7
GLOBAL BUCKLING MODE FOR PAIRED STEEL BEAMS DURING POURING OF DECK SLAB

Web transverse stiffeners

The design of web transverse stiffeners provided to enhance shear resistance has provoked some debate in the UK. The original Eurocode proposal was little more than a stiffness requirement. This was augmented, following comments by the UK, to include a strength requirement similar to that provided in BS 5950, but not going as far as the similar strength requirement in BS 5400:Part 3, which was more conservative for asymmetric beams and beams carrying compression.

Even with this less conservative proposal adopted in EN 1993-1-5, the Eurocode still provides an effective way of assessing existing structures to demonstrate adequacy where BS 5400:Part 3 suggests inadequacy. Work commissioned by the Highways Agency, comprising some 40 non-linear analyses of stiffened plate girders, did not find any cases where the Eurocode was unsafe. The UK National Annex however still provides some caveats to its use.

3. Concrete design to EN 1992-2

Non linear analysis

More advanced calculation methods are allowed and codified in EN 1993. Non-linear analysis is one example and its use can lead to significant refinement of designs. The model in Figure 8 was one of many set up by Atkins on behalf of the Highways Agency to investigate the Eurocode rules for transverse stiffeners resisting shear. The model was set up in accordance with the requirements of EN 1993-1-5 and modelled the geometry of an actual physical test specimen, tested in the 1980s[3]. Not only did the non-linear FE model give results almost identical to the physical test, but it also showed the EN 1993-1-5 rules for stiffeners to be very conservative for this particular beam and the BS 5400:Part 3 predictions even more so.

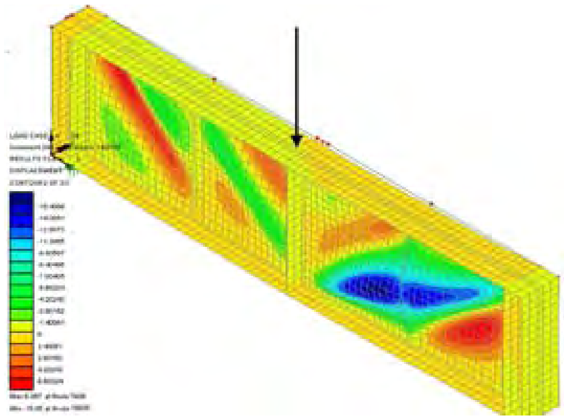


FIGURE 8
NON-LINEAR MODELLING OF A PLATE GIRDER IN SHEAR

Serviceability

Checks of steel beams at the serviceability limit state are very similar to those currently employed. A new check on web breathing has, however, been introduced to control fatigue in welds at panel boundaries but this check never seems to be a governing one.

For composite bridges, the check of crack widths is significantly different in both approach and outcome and is in many ways more straight-forward than in previous practice. Crack widths are calculated under "quasi-permanent" actions. These are essentially permanent loads and the effects of temperature. Having calculated the stress in the reinforcement under these actions, using cracked section properties and adding a correction term to allow for tension stiffening, the resulting stress in the reinforcement is compared against one of two limits; one based on bar spacing and another based on bar diameter, both for a given design crack width, usually 0.3 mm. It is only necessary to satisfy one of the two limits. As a consequence, whereas crack width was often a governing check to BS 5400, it will rarely govern the Eurocodes.

Materials

It is essential to note when using EN 1992 that all the formulae use cylinder strength rather than cube strength. Since cylinder strength is approximately 80% of the cube strength, this is an important distinction. The formulae are also generally applicable up to much greater concrete strengths than in previous UK practice; C70/85 for bridges and C90/105 for buildings. The UK National Annex however places a limit on cylinder strength in calculations of 50 MPa for shear. This is due to concerns over the validity of the equations with high strength concrete, particularly those with limestone aggregates[4].

The approach to shrinkage and creep calculation is more rigorous. Shrinkage strain is calculated taking into account the concrete composition, the environmental humidity and the concrete section dimensions. Total creep strain additionally depends on the concrete age at first loading. The concrete Young's modulus for permanent loads is calculated from considerations of creep factor and is not taken simply as half the short term modulus as was previous UK practice. The EN 1992 approach typically leads to lower values of the long term modulus.

Global analysis

As with steel design, the default analysis in EN 1992 is second order, but first order analysis will suffice in almost all situations where first order analysis was used in BS 5400. Similarly, shear lag needs to be considered but frequently will not lead to any actual reduction in acting flange width. Geometric imperfections (e.g. lack of verticality in columns) also need to be considered in analysis. Uncracked linear elastic analysis may always be used for all limit states and a certain amount of moment redistribution is permitted at ULS, depending on the depth of the compression zone at each cross section. Non-linear global analysis may also be used for SLS and ULS, while rigid plastic global analysis is permitted only for ULS combinations where there is adequate rotation capacity and where permitted by the National Authority. In the UK, this means that it will only be permitted for accidental combinations of actions.

Resistance to bending and axial force

For design of reinforced concrete sections in bending and axial force, there is a choice of stress-strain curve for reinforcement as shown in Figure 9. The curve with a plateau at design yield may be used without any limit on reinforcement strain, while the curve with the branch rising towards design ultimate tensile strength may only be used if a limit is placed on the strain in the reinforcement to prevent fracture. Use of the latter curve with under-reinforced sections and ductility Class B reinforcement can give around 7% greater bending resistance than is obtained with the idealisation with the yield plateau.

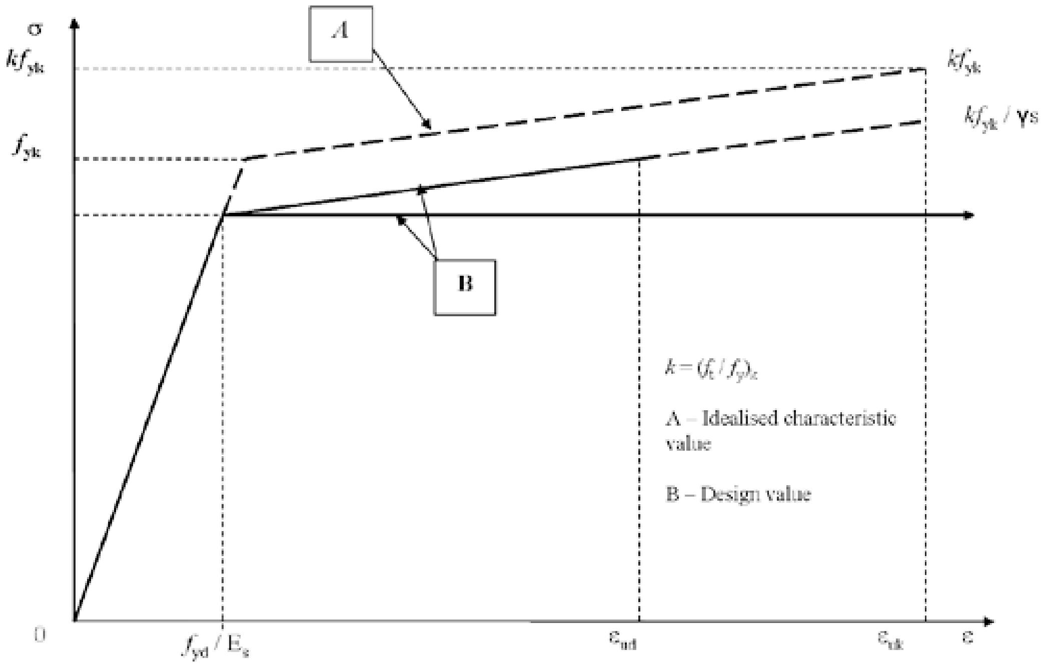


FIGURE 9
ALTERNATIVE REINFORCEMENT
STRESS-STRAIN DIAGRAMS

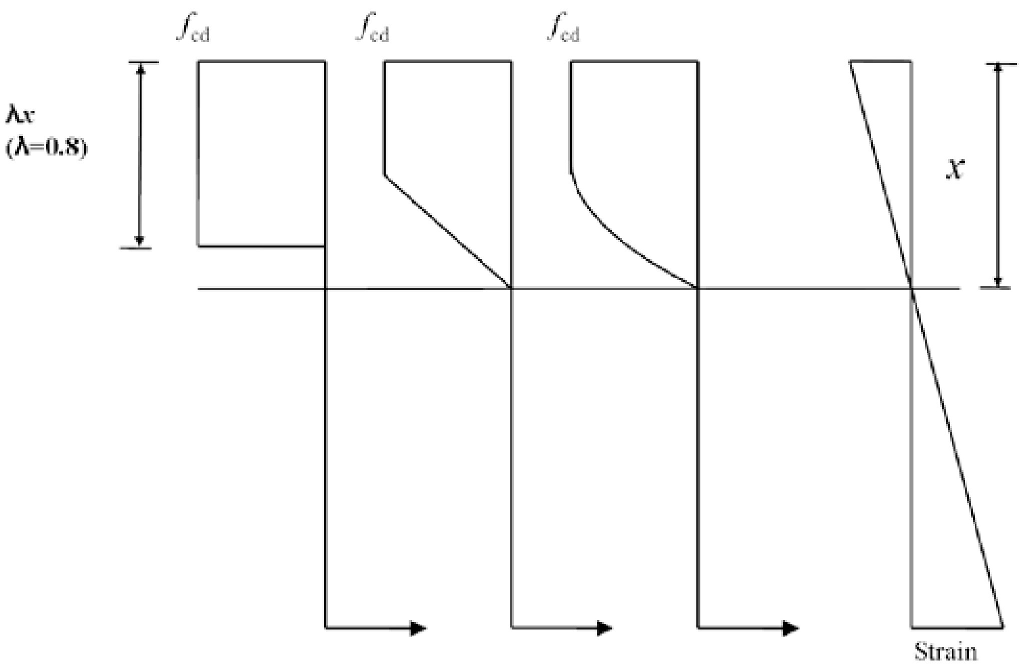


FIGURE 10
ALTERNATIVE CONCRETE STRESS
BLOCKS FOR FLEXURAL DESIGN
(SHOWN FOR FCK ≤ 50 MPA)

STRUCTURES

Around 10% increase in bending resistance can be obtained compared to the resistance from BS 5400:Part 4, which is also due in part to the lack of a limit on lever arm in EN 1992 equivalent to the of 0.95d limit in BS 5400:Part 4.

There is also a choice of concrete compression block to use in the calculation as shown in Figure 10. The rectangular block differs from that in BS 5400:Part 4 as it does not extend all the way down to the neutral axis. The rectangle-parabola curve is very similar to that in BS 5400:Part 4. For $f_{ck} \leq 50$ MPa, the rectangular block, which is the simplest to use in calculation, always gives the greatest bending resistance.

This is a consequence of the fact that all three stress blocks have the same limiting stress f_{cd}, unlike BS 5400:Part 4 where the rectangle-parabola block had a greater limiting stress than the rectangular block.

No formulae are given in EN 1992-2 for flexural resistance of reinforced concrete sections as it was felt by the drafters that computer programmes made such material redundant and it was, in any case, text book material. Design guides[5] are available to provide simple formulae. Shear lag must be considered at ULS.

Shear resistance

Where there are no designed links, as in the majority of slabs, the formula for shear resistance of reinforced concrete is very similar to that in BS 5400:Part 4. The resistance is a function of reinforcement percentage, concrete strength and a depth factor. Additionally, where there is axial force, the shear strength is enhanced if the force is compressive, and reduced if it is tensile. The resistances produced for reinforced concrete without axial force are similar to those from BS 5400 Part 4, but slightly lower.

Where there are designed links, however, the approach is quite different to that in BS 5400:Part 4 and leads to a potential large increase in economy. The resistance is based on a truss model, as in BS 5400, but unlike BS 5400 the angle of the web compression struts can be varied by the designer between 45° and 21.8° to the horizontal; Figure 11 illustrates the truss model. A design with flat struts minimises the amount of shear reinforcement required (the struts cross more links) but also minimises the maximum shear force that can be put on the cross section, irrespective of shear link quantities, based on crushing of the concrete.

This latter is not usually of great consequence as the maximum shear stresses permitted by EN 1992-2 are generally considerably higher than those allowed in BS 5400:Part 4. A flat angle also maximises the length that the longitudinal reinforcement must continue beyond the point where it is no longer required for bending. This means that shear design can be a balancing act between the different failure mechanisms. It should also be noted that, unlike in BS 5400:Part 4, a concrete contribution cannot be added to the link resistance. This means that if a designer opts for a truss angle of 45° for consistency with previous practice, more links will be required in the EN 1992-2 design because of the absence of this concrete term.

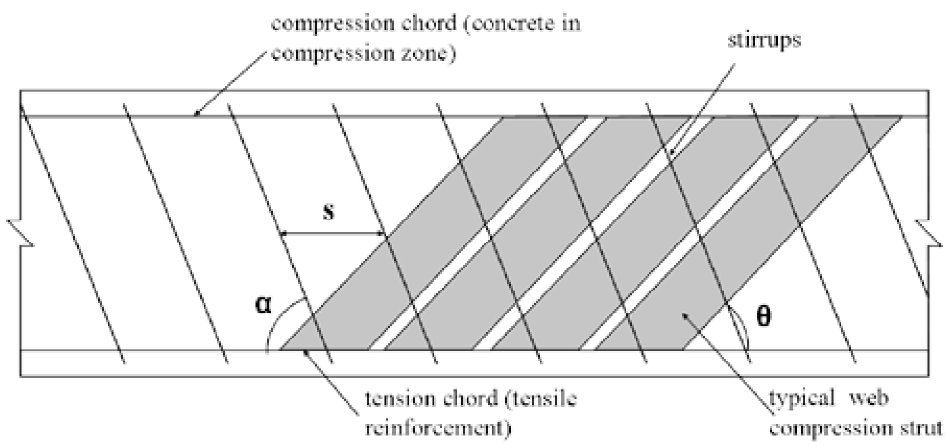


FIGURE 11
SHEAR TRUSS MODEL
TO EN 1992

Punching shear

Checks on punching shear are carried out in a similar way to the procedure in BS5400:Part 4, but there are some cosmetic differences. The basic punching perimeter is at 2*d* from the edge of the loaded area and the perimeter edges are rounded off as shown in Figure 12. In the absence of loads opposing the punching load within the basic perimeter, it is usually sufficient to check this perimeter and another at the face of the load (for crushing resistance).

A more significant difference is that the calculation of punching shear stress makes allowance for any moment transmitted at the same time as the shear load. The effect of moment is to increase the shear stress on one face of the punching perimeter and reduce it on the other as shown in Figure 13. This will affect the design of pile caps, for example, where significant pile axial forces and moments can co-exist at the connection with the pile cap.

Non-linear analysis

Non-linear analysis has always been allowable in the UK, but has required a Departure from Standards. It is now codified in EN 1992-2, although the guidance is not comprehensive. An analysis based on design material properties may usually be carried out, subject to certain provisos about indirect actions, or an analysis with realistic properties in conjunction with a complex and poorly-explained safety format may be used. The preference in the UK is for the former.

Non-linear analysis can be particularly beneficial for the analysis of slender piers, where simplified alternative hand calculation methods can be very conservative. The New Medway Bridge piers (Figure 14) were designed using non-linear analysis in accordance with the draft version of EN 1992-2 at the time.

This showed that whilst 32 mm diameter bars were required when a non-linear analysis was used, BS 5400:Part 4 would have led to in excess of 40mm diameter bars at the same centres.

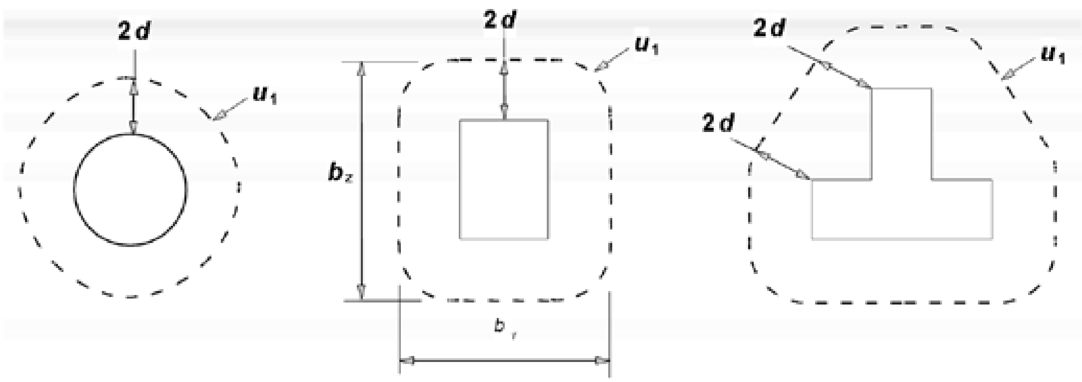


FIGURE 12
TYPICAL BASIC PUNCHING PERIMETERS

$$v_{Ed} = \beta \frac{V_{Ed}}{u_i d}$$

$$\beta = 1 + k \frac{M_{Ed}}{V_{Ed}} \frac{u_1}{W_1}$$

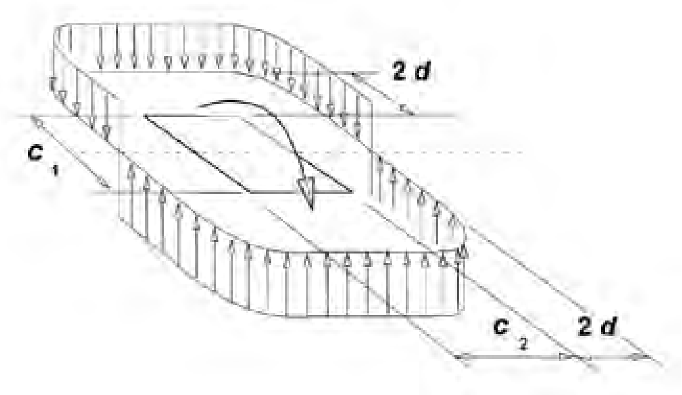


FIGURE 13
EFFECT OF MOMENT ON PUNCHING SHEAR STRESS DISTRIBUTION

FIGURE 14
SLENDER PIERS ON NEW
MEDWAY BRIDGE

Serviceability

The philosophy for stress checks and crack control differs from previous UK practice. Different serviceability combinations apply to different checks. For example, it would be undesirable for reinforcement ever to yield in service as this would potentially lead to irreversible deformation. Consequently, reinforcement stress checks are done using the "characteristic" combination of actions, which represents actions that are unlikely to be exceeded in the design life of the structure. However if a large crack opens under live load and then closes again, it is unlikely that this would compromise durability. Consequently, crack width checks are performed in the "quasi-permanent" combination, which represent actions which will occur 50% of the time. This is because the time-averaged crack width is most relevant to durability and not the largest crack ever experienced. This change in philosophy means that crack width checks rarely will govern the design of reinforced concrete when EN 1992 is used, whereas cracking normally governs flexural reinforcement provision to BS 5400:Part 4.

4. Pilot studies

Trial calculations conducted on existing concrete and steel-concrete composite bridges, carried out for the Highways Agency and other clients, have indicated that the Eurocodes give a small increase in economy in the design, on average, when the basic application rules are applied. For steel design, there is more economy to be obtained from using the Eurocodes for stiffened structures, which reflects a greater confidence in behaviour as a result of recent testing and non-linear parametric studies. For concrete structures, there is a systematic saving in flexural reinforcement and shear reinforcement for reinforced concrete structures, but generally little difference for prestressed structures. Economy can, however, be improved further when the more complex methods of analysis permitted are employed. This is all part of the general conclusion that the Eurocodes give greater scope for innovation and often reward more complex analysis.

One other significant conclusion from the pilot studies was that the engineers best able to cope with the change were those who were taught with an emphasis on structural theory, rather than on the use of specific design codes. This may well influence where the emphasis lies in teaching in the future.

5. Concluding remarks

There are cultural and technical differences in the Eurocodes that designers will have to get used to and extensive training and guidance documents will be needed in the transition. However, the pilot studies also reveal that designers adapt quickly and there are many similarities to current UK practice. Further motivation for the change should be that the less prescriptive approach and more up-to-date rules provide the designer with greater scope for innovation and economy.

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Conceptual design of cable stayed pedestrian bridge at Taunton, Somerset

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Abstract

This paper intends to describe the conceptual design of a Cable Stayed Bridge at Taunton, Somerset, United Kingdom. The development of the detailed design and the critical issues associated with the aerodynamic effects, fatigue effect on cable stays, layout of cable stays, deck and pylon of the cable stayed bridge have been briefly discussed. The design aimed to meet the Client's requirement of minimising the aerodynamic effect on the structure and optimise the ratio of back span to main span and pylon inclination in order to minimise the uplift effect at supports.

1. Introduction

Project brief

Taunton (Somerset, UK) desires to develop a vibrant mix of employment, retail, housing, cultural and leisure facilities by 2021. It is in the process of developing its transport infrastructure to meet the needs and demand of its growing population whilst retaining its distinctive and valued market town character.

The key infrastructure needed in the short-medium term has been identified as the Taunton 'Third Way' which has evolved from the proposals for an Inner Relief Road, and the Taunton Northern Inner Distribution Road (NIDR).

The NIDR provides strategic access to Firepool and in conjunction with the Third Way would enable regeneration of the large areas of derelict brown field land adjacent to the River Tone which runs through the centre of Taunton.

Firepool is the industrial heart of Taunton and provides a strategic employment site within the town centre. It has the potential to attract a range of new businesses including government departments/relocations by virtue of its strategic transport links, and the quality of its riverfront environment.

The NIDR scheme includes a new bridge link over the River Tone and the Taunton and Bridgewater Canal. The bridge will mark the transition point into the new and expanded Taunton town centre. The bridge caters to the requirement of motorists, pedestrians and cyclists. This bridge is henceforth referred as the 'Main Bridge' in this paper.

As part of the above bridge scheme, it is also proposed to construct a landmark structure in the form of a pedestrian cable stayed footbridge along the river/canal connected to the Main Bridge.

Scope of Work

The scope of the project included the Preliminary Design of Pedestrian Footbridge. Specific requirements of geometric and structural design were obtained from the client in the form of a design basis note.

The construction of the proposed bridge is at an early stage of development and needs to be taken through a number of Statutory Procedures, which include Planning Consent and Land Acquisition, before construction works can commence. It is envisaged that construction work will commence by mid 2009 and will be completed by the end of 2010. The development of the scheme will be undertaken by Somerset Highways and the construction works will be the subject of a competitive tendering process, which will involve national road building contractors.

2. Design specifications

Location of the bridge

The proposed footbridge is located at Firepool, Taunton, Somerset, UK. It is located between the Rive Tone and Taunton/Bridgewater Canal, as shown in the schematic location plan in Figure 1.

The footbridge connects to a vehicular-cum-pedestrian river bridge (Main Bridge) at an elevated level and gradually descends to meet an existing road at grade, spanning over a virgin territory. The conceptual plan is shown below in Figure 2.

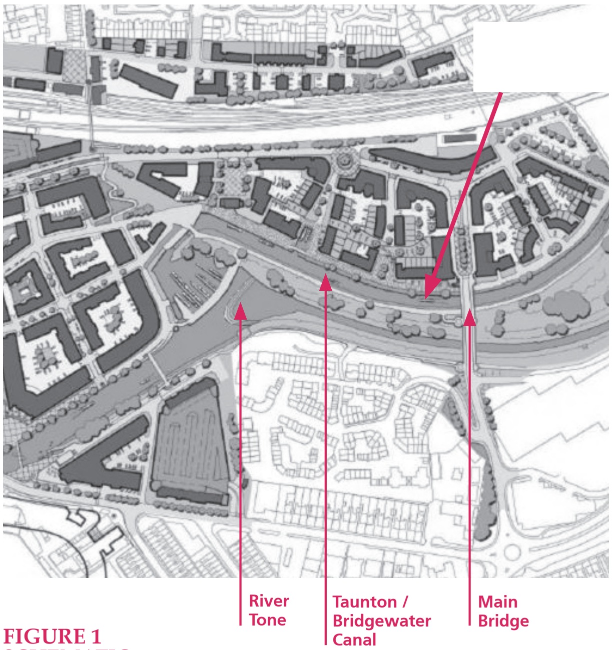

FIGURE 1
SCHEMATIC LOCATION PLAN

FIGURE 2
CONCEPTUAL PLAN FOR THE MAIN BRIDGE AND THE PEDESTRIAN BRIDGE

Design Standards

The preliminary design was based on the British Standards and Design Manual for Roads and Bridges (DMRB).

The footbridge was designed in accordance with BS 5400:Part 3 2000. The loading was taken from BS 5400:Part 2 2006, Highways Agency standard, BD 37/01 (DMRB Volume 1, Section 3, Part 14). The bridge was designed for footways and cycle track loadings.

"Aerodynamic Susceptibility Parameter" was determined in accordance with the Bridge Directives and Advises, BD 49/01 (DMRB Volume 1, Section 3, Part 3). Technical Guidance Notes on assessment of vibration behaviour of footbridges under pedestrian loading published by SETRA, France, were also referred to during the design process.

Relevant data and specifications

Geometric design
The terrain profile at the proposed site is flat with minimal variations in the ground levels. The longitudinal gradient of the structure was restricted to 5% as per Clause 6.4 of BD 29/04 (DMRB 2.2.8). The minimum headroom was restricted to 2.4m as per Clause 8.5 of BD 29/04 (DMRB 2.2.8).

The proposed minimum clearance under the structure was restricted to 2.4m.

Structural design
The span of the proposed bridge is 60m and it consists of a single deck with carriageway width of 3.5m.

The preliminary design aimed to achieve maximum ratio between the main span and the back span whist ensuring minimum or no uplift in the backstay anchorage and satisfying the aerodynamic stability requirements of BD 49/01 (DMRB 1.3.3).

The design evaluated the aesthetic and the structural feasibility for the following pylon heights - Option 1: 14m and Option 2: 24m. The heights of the pylon were specified by the client. Option 1 was rejected based on aesthetic considerations.

Aesthetic appearance

To reinforce the distinctive character of the proposed footbridge location and its future role, the County Council desired to construct a landmark structure in form of a cable stayed footbridge, which aesthetically blends with the surroundings.

Pile Diameter	Socket Length	Negative Skin Friction	Allowable Working Load	Allowable Net Working Load
m	m	kN	kN	kN
0.75	4	245	1470	1225
0.75	6	245	2206	1960
0.75	8	245	2940	2695

TABLE 1
GEOTECHNICAL RECOMMENDATIONS (PILE CAPACITY)

Wind and temperature

The preliminary design was based on a basic hourly mean wind speed of 22m/sec (Vb). This value was obtained from Figure 2 of BD 37/01 (DMRB 1.3.14). The temperature loading was calculated based on maximum and minimum shade air temperature of 34° C and -18° C for a group two type superstructure. As per BS 5400:Part 2, group two structures include steel deck on steel truss or plate girder structures.

Soil condition

Soil investigations revealed the presence of a significantly thick made ground and variability of the superficial sand and gravel deposit. Based on the preliminary estimated design loads it was recommended that the bridge be supported on pile foundations. The recommended pile capacities for different socket lengths are enumerated in Table 1 of this paper.

Analysis methods

The static, dynamic and non-linear analysis of the structure was carried out using LUSAS 14.1 software. The structural model is shown in Figure 3. Natural frequency corresponding to the sum of the mass participation factor greater than 90% was evaluated to determine the aerodynamic stability parameters.

The cable elements were modelled as bar elements with rotational releases at each end. The modulus of elasticity was modified manually based on the Ernst Equation, taking into effect the centenary profile of the cable.

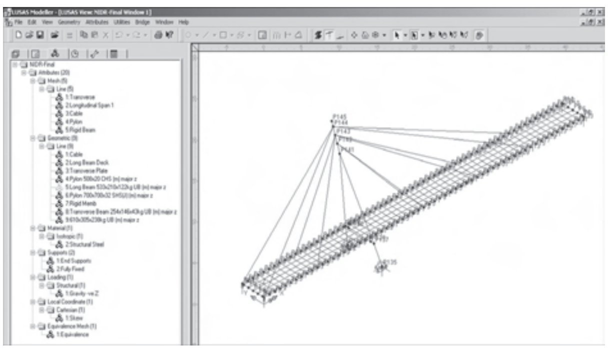

FIGURE 3
LUSAS MODEL

The aerodynamic susceptibility parameter (P_b) was calculated as per BD 49 using the following equation:

$$P_b = \left(\frac{\rho b^2}{m}\right)\left(\frac{16V_r^2}{bLf_b^2}\right)$$

Where:

- ρ = Density of air
- b = Total width of the structure
- m = Mass per unit length of the bridge
- V_r = Hourly mean wind speed
- L = Length of relevant maximum span of the bridge
- f_b = Natural frequency of the bridge

Based on the value of aerodynamic susceptibility parameter (P_b), BD 49 classifies bridge structures in the following categories:

(a) Bridges designed to carry the loadings specified in BD 37 (DMRB 1.3), built of normal construction, are considered to be subject to insignificant effects in respect of all forms of aerodynamic excitation when $P_b < 0.04$.

(b) Bridges having $0.04 \leq P_b \leq 1.00$ shall be considered to be within the scope of the rules specified in BD 49, and shall be considered adequate with regard to each potential type of excitation if they satisfy the relevant criteria given in the code.

(c) Bridges with $P_b > 1.00$ shall be considered to be potentially very susceptible to aerodynamic excitation, and shall be verified by means of further studies or through wind tunnel tests on scaled models.

Aerodynamic Susceptibility Parameter (P_b) was calculated based on the natural frequency derived from LUSAS analysis and was found to be less than the limiting value of 0.04 and hence the structure was considered to be subject to insignificant aerodynamic effects.

The analysis evaluated the load effects on the various elements of the structure during the Service and the Construction Stage. The procedures for replacement of cable and bearings were proposed to be considered at the detail design stage.

3. Preliminary design considerations

Introduction

The footbridge was planned as an asymmetrical cable stayed bridge with span of 60m and a steel girder deck of 550mm deep. The deck is supported from a 24m steel pylon by 5 pairs of high tensile galvanized cable system. Three pairs of cable supported the main span and two pairs of cable supported the back span.

The superstructure deck is continuous over pylon and simple supported over two abutments. The end stays on the main span are anchored in the abutment at the north end, whereas those on the back span are anchored in the ground at the south end to eliminate the uplift forces on abutment. The structure is restrained in the transverse direction at the pylon.

Steel was selected as the material for deck and pylon because of the benefit derived in terms of the ease in construction.

The following section aims to outline the various options considered in the conceptual design of the various key structural elements of the cable stayed bridge. The choice was made after careful deliberation on their relative merits and demerits.

Structural interaction between cables, deck and pylon

The basic load bearing elements of the cable stayed structure are the cables, deck and the pylon. The relative stiffness of the deck and the pylon and the number of cables determine the behaviour of the structure. The following options in Table 2 were considered in this regard.

Options	Deck Stiffness	Pylon Stiffness	Cable Spacing	Remarks
Option - 1	Very Stiff	Slender	Relatively Large	Bridges built in the recent past with this option, have had higher construction cost, which makes them economically non feasible.
Option – 2	Slender	Stiff	Relatively small	This option is usually more feasible for multi-span bridges.
Option – 3	Slender	Slender	In this option the cables acts as the determining stabilising element of the structure.	This is the recommended option for the following reasons:
1.	Acceptable structural behaviour.			
2.	It leads to a relatively slender deck and pylon.			
3.	The slender deck and pylon improve the aesthetic value of the structure.			

TABLE 2
EVALUATION OF OPTIONS: STRUCTURAL BEHAVIOUR

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Superstructure

Transverse cable layout
The central suspension and the lateral suspension options were considered for the transverse cable layout. The deliberation on their relative merits and demerits is set out in Table 3.

Longitudinal cable layout
The Harp, Fan and Semi Harp layout options were considered for evaluation. The deliberation on their relative merits and demerits is set out in Table 4.

Cable spacing
Multiple cables are proposed to be used with the following advantages:

- The cables act as an elastic support for the deck, hence multiple cable increases the number of elastic supports leading to moderate longitudinal bending in the deck.
- Multiple cables result in lesser forces on the cables.
- Replacement of cable is relatively simple.

Deck
The following deck cross sections were evaluated to determine the most aerodynamically stable structure. The selection was based on reaching the fine balance between the stiffness and the weight of the superstructure.

- Steel plate box girder
- Steel I - beam with steel plate deck
- Steel I - beam with thin RCC deck slab

The proposed deck consists of steel plate resting on two universal I-beams (Figure 4). The steel plate deck was overlaid by 5 mm thick anti-skid and anti slip system. A universal steel beam (UB) was used as cross girders at 2m intervals, which contributes to the transverse rigidity of the structure. The transverse rigidity of the structure influences the vibration behaviour of the structure due to the wind loads.

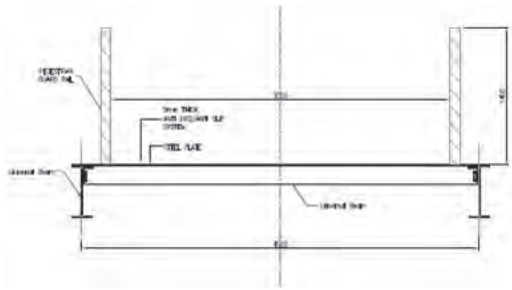


FIGURE 4
DECK CROSS SECTION

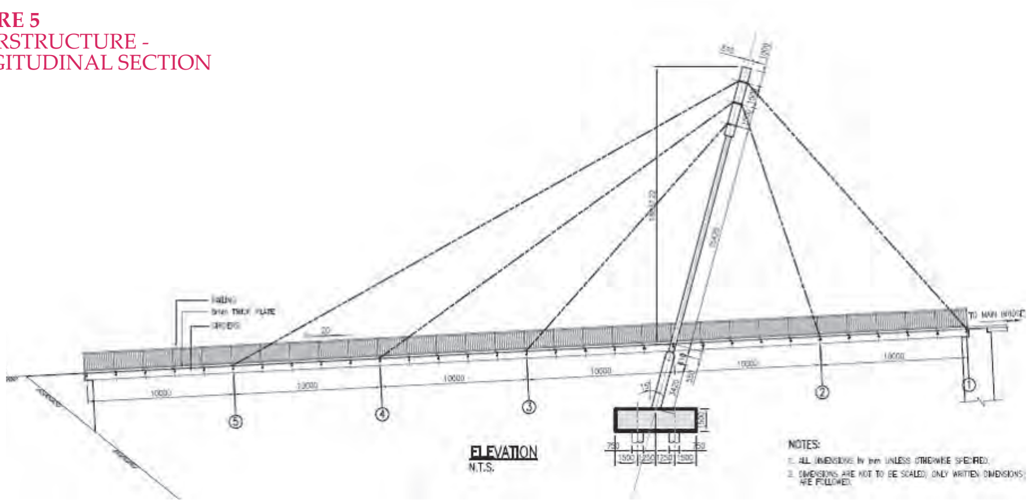
Options	Remarks
Option – 1 Central Suspension	**Merits:** 1. This option is aesthetically superior. **Demerits:** 1. The deck requires higher torsional rigidity and the bending stiffness of the deck is not exploited to its capacity.
Option – 2 Lateral Suspension (Recommended option)	**Merits:** 1. This option provides improved stiffness and stability of the deck, when used with A-frame pylon. 2. This option provides greater aerodynamic stability to the structure. **Demerits:** 1. Head room clearance may be restricted. 2. Erecting A-frame pylon is generally more complicated.

TABLE 3
EVALUATION OF OPTIONS: TRANSVERSE CABLE LAYOUT

Options	Remarks
Option – 1 Harp	**Merits:** 1. This option is aesthetically superior. **Demerits:** 1. This layout is not the best from the static or economic point of view.
Option – 2 Fan	**Merits:** 1. This layout is advantageous from the static point of view. **Demerits:** 1. This option is aesthetically poor.
Option – 3 Semi Harp	Semi Harp is an intermediate solution, between extremes of Harp and Fan patterns. This pattern combines the advantages of both these systems, whilst avoiding their disadvantages. Hence Semi Harp layout is recommended to be used.

TABLE 4
EVALUATION OF OPTIONS: LONGITUDINAL CABLE LAYOUT

FIGURE 5
SUPERSTRUCTURE - LONGITUDINAL SECTION



The geometric proportioning of the deck also ensured that it met the criteria laid down in the British Standards, as well as the Client's directive, that no wind tunnel tests should be required for evaluation of the aerodynamic parameters at the preliminary design stage.

The serviceability requirement for the superstructure has been met by ensuring that the fundamental natural frequency of vibration exceeds 5Hz in the vertical direction for the bridge, without any live load and 1.5Hz in the horizontal direction for the bridge with live loading. The structure meets the requirements of vortex shedding, classical flutter and divergent amplitude in accordance with the British Standards and Bridge Directives of British Highways Agency.

The longitudinal view of the superstructure is shown in Figure 5.

Cable stays

Full locked coil strands made from hot dip galvanized high strength steel wires are proposed to be used for cable stays. The strand comprises an inner core of round wires made of one or more external layers of Z shaped wires. The Z shape of the wires is specially prepared in a self locking formation to give a compact section as shown below.

The typical properties of the wires are:

Tensile strength:	1570 to 1600 MPa
Proof stress:	1180 to 1245 MPa
Elongation at breaks:	4% minimum on 250mm gauge length

The critical aspect of the performance of cable stays is their behaviour under fluctuating loads. In the proposed Firepool Bridge, the cable stay anchorages are attached to the web of the universal beam with a pinned connection, as shown in Figure 6.

FIGURE 6
CABLE STAY ANCHORAGE

Locked coil tendons have a fatigue performance in excess of 2 million cycles at a stress range of 150 N/mm^2 and with a maximum stress of 45% of the ultimate tensile stress. The stress levels in the cable were restricted to meet the above fatigue criteria.

Substructure

Pylon

The form and properties of the pylon in a cable stayed bridge are very important because it provides the main vertical resistance to the bridge. The appropriate form of the pylon depends on balancing the structural, maintenance, geometrical and aesthetic factors. Hollow pylon sections provide the opportunity for fixed access to the top from inside the pylon, but for smaller bridges providing sufficient room for access makes the external dimensions of the pylon out of proportion with the rest of the bridge. In this case the external dimensions of

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the pylon have been evaluated based on the structural requirement.

The 24m height of the pylon was primarily based on aesthetic reasons. An inverted Y-shaped pylon was proposed to enhance the torsion stiffness provided by the cable system. Details of the pylon are shown in Figure 7.

The forces in the pylon due to other load effects were calculated by performing a plane frame analysis using LUSAS 14.1.

Foundation

The superstructure is supported on electrometric bearing resting on bank seat abutment. The pylon is supported by 750mm pile foundation.

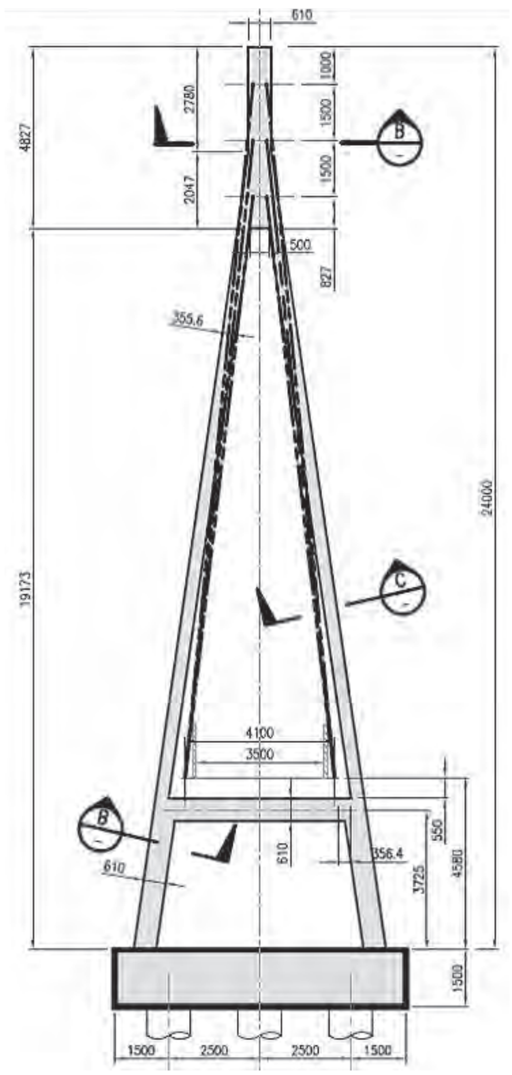

FIGURE 7
PYLON

4. Method of construction

The erection procedure has a very strong influence on the design of the cable stayed bridge. However in this case the proposed bridge site has a uniform elevation with minimal ground obstructions; hence the construction is proposed on a continuous staging over the length of the bridge.

Construction on continuous staging eliminates the requirement for construction stage analysis and hence the same has not been considered in the preliminary design.

5. Health & Safety and Environment

Construction Design and Management Regulation (2007), requires integrating health & safety issues into the design and management of projects. The focus is on the actions necessary to reduce and manage risks associated with health & safety and environment issues connected with construction and usage of structures. A preliminary designer risk assessment was prepared for the proposed bridge. The key aim of the report was to:

- Improve the planning and management from the very start of the project
- Identify hazards early on so that they can be eliminated or reduced at the design or planning stage and the remaining risks can be properly managed
- Target effort where it can do the most good in terms of health & safety

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The influence of design and detailing on the life and maintenance of a bridge

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Abstract

Each new structure is designed in accordance with the standards and details which are codified at the time of its design and conception, and with the requirements of the owner of the bridge who will be responsible for its maintenance in the long term. The codes and standards are guidelines, but are not generally definitive, which gives the design engineer scope to use best practice and his own experience when designing an individual structure. The main enemy of a bridge, in the UK at least, is uncontrolled water, but careful attention to the use of materials and the shaping of the vulnerable areas to shed water and take it away in a controlled manner can ensure that the maintenance requirements of a structure can be minimised.

1. Introduction

Many ancient structures standing today are assumed to have been well built, and remain standing because they were well built, but this is not necessarily the case.

To extend the idea to nature, even the most durable rocks are attacked by the elements and gradually deteriorate, so it is unreasonable to assume that a structure made from the same materials will last without any maintenance. In fact, we tend to use the weaker, more workable material in our structures, which makes the problem worse.

Any structures which are out in the open in all weathers will be affected by wind and rain, and alternating heat and cold, in the same way as cliffs or crags. The normal process leading to damage of a structure built of stone is that wind-driven rain penetrates the stone or the joints which are in any way porous, then freezes and breaks up the stone. An example of this can be found in Durham Cathedral, built from 1092 AD onwards; this has had scaffolding somewhere about it to allow the repair of stonework for as long as I have known it, which is over 50 years, and this is probably the case with most large and ancient structures. Those which have not been constantly maintained have fallen into disrepair and in some cases are now ruins.

The performance of the foundations of a structure must also be considered as one of the defining factors in the life of a structure. Foundation failure is normally avoided by the robustness of the design, be it spread footing or piled foundation, but the ground conditions themselves can lead to material failure. Before the application of soil mechanics theory to foundations, whether a structure stood up or not was a matter of trial and error. There was an understanding of soft ground and rock, and where there was a need for piles, but much of this was based on local knowledge, not theory. Where piles were used, which was usually in a marshy situation, timber was the preferred material, because that was all that was available. Its properties were such that being kept wet preserved it. The most vulnerable part of a pile was the short length near the surface which was subject to wetting and drying, so as long as this was protected, the life of the pile could be limitless.

Other natural materials used in the past were "withy mats and sheep's wool". During the railway building era in Britain there was a need for large numbers of viaducts, which were usually built in stone or brick, or even a mixture of both. In marshy ground it was not uncommon to found the arch structures on mats made from willow staves woven together, with sheep's wool layered into it, which acted as a spreader. Durham Viaduct (Figure 1), on the main line from London to Edinburgh, is reputed to be built on such a foundation, and has been there from 1856. At the time the local pundits said that it would not last, but it is still there, still taking mainline trains, after more than 150 years.

FIGURE 1
DURHAM VIADUCT

2. Bridges

If we consider bridges, the oldest structures from modern times are railway bridges. These are usually either arch structures in brick or stone, beam bridges in wrought or cast iron, or trusses in the same material.

2.1 The life of arch bridges

The life of arch bridges has been shown to be variable, and they fail in different ways.

The essence of an arch bridge is that the foundations are sound, and that the arch barrel itself is supported by a firm backing. It also is of benefit if the span of the arch is square and the backing to the arch is drained.

It is unfortunate that not all of these conditions were met in the railway boom in Britain, nor in other parts of the world.

The railways were built largely between the 1840s and the start of the 20th century to link industrial centres, over routes where the founding material could be variable, so problems arose through settlement of the foundations leading to arch distortion, which reduced the arch's capacity and led to failure.

But arches were constructed by artisans and were largely undesigned in modern engineering terms. Their sizing was estimated by the masons who worked to drawings prepared by draughtsmen with little theoretical basis.

By and large, however, masonry arches and modern concrete developments have been found to be durable and reliable in terms of performance.

2.2 Metal bridges

With the advent of cast and wrought iron, and latterly of steel, which were capable of withstanding tensile forces, structures spanning as beams became possible.

These took various forms according to the spans involved, but it can be presumed that none of these were designed with a finite life in mind. They were designed to perform a function, and not fall down, and that was as far as it went.

2.3 The failure of metal bridges

As rail loading changed, various bridges which had never been designed to modern levels of sophistication were assessed to modern codes, and were found in many cases to be below standard. Part of the assessment was to inspect the bridges for condition, and these inspections found that bridges were often corroded at similar details.

It was found that the detailing of beams and trusses almost invariably resulted in corners where water and leaves or other detritus could be retained.

Typically this would be at the junction of vertical stiffeners and bottom flanges, or at a point where truss members came together. The most serious corrosion of this kind would be at the bearing seating, where the shear stress would be at its highest, or else at mid-span at the point of maximum flange tension, which was equally undesirable. The message from this is that if these places had been regularly and frequently cleaned the life of the members could have been prolonged.

That having been said, some of these bridges are still in service after a hundred years, so good detailing is not necessarily the most important factor; making members bigger than strictly necessary for strength reasons can be beneficial in terms of sacrificial material being available. The better the detailing however, the smaller the sacrificial steel thickness required and this is especially true of unprotected weathering steel bridges today.

2.4 Definition of the end of the life of a bridge

As long as the repair or maintenance of a bridge is economically viable, and its design or assessed capacity is adequate for the duty it is being asked to perform, then it can be considered to be within its design life.

Once the cost of replacement is less than the cost of repair over a defined period then a structure can be considered to have reached the end of its life.

An alternative to replacement is to downgrade the duty of the bridge and re-route whatever it is carrying. In this way a highway bridge can become a footbridge, while the road it used to carry is diverted onto a new structure. Thus the old bridge lives on.

In the case of metal bridges, if the deck is in good condition, but the details are not in accordance with current practice, then the choice is between strengthening or replacement, which comes down to a matter of cost. This is not a simple choice based on the extent of strengthening; the cost of disruption to rail timetables or road closures must be added into the works cost, which can make replacement almost as economical as adding on strengthening material.

The decision on the end of life of a metal bridge is easiest when corrosion is considered. The loss of section in critical areas is usually difficult to repair because by definition they are highly stressed, and relieving the stress is not generally easy. In economic terms, the cost of relieving stress by jacking or other means can exceed the cost of the repair, making deck replacement a more attractive option, since all of the deck will be renewed, giving a more predictable future to the bridge.

3. Modern bridges

The foregoing has been related to historic bridges, so now attention will be turned to modern structures. In the same way that rapid expansion of the railways led to a boom in construction of bridges to carry them, the spread of the motor car and expansion of road transport led to construction of motorways and other major trunk roads. In Britain this was largely from the 1960s onwards, suggesting a period of some 40 years during which the defects arising in modern bridges have been able to develop.

However, the design and detailing of structures in modern materials was carried out in the early years of the motorway expansion, the 1960s and 1970s, with limited knowledge of their long-term properties, and with no clear idea of how they would deteriorate, other than knowing that steel rusted and would need painting.

Concrete, once believed to be permanent, has been subject to alkali-silica reaction, carbonation, chloride attack, sulphate attack and thaumasite attack, to name the common ones.

Structural steelwork fails in a number of ways which are not always immediately detected by visual methods. Fatigue and buckling failures are both sudden, but cannot normally be seen in advance of the event. Fatigue failure is avoided by careful detailing and buckling, including lateral torsional buckling, is avoided by design, but standards can change with increasing traffic loading and with continuing research into structural behaviour. "Steady-state" assessment can lead to the conclusion that the bridge could fail, but there is no visual evidence, so in such cases the bridge must be monitored using instruments. Strain gauges can detect the stress regime, dye penetration can detect cracks in welds, and the advance of fatigue cracks can be sensed by the use of Acoustic Emission Monitoring (AEM), where each reversal of stress producing an advance in the crack can be "heard" by continuous monitors.

The condition of closed structures where access is difficult can be ascertained by use of air sampling, which can indicate an atmosphere conducive to corrosion, or by viewing through an aperture using remote optical or video recording instruments.

Most modern bridges incorporate concrete within their structure as supports, with decks being of reinforced or prestressed concrete, or steel and composite concrete. Larger bridges may be of suspension form or, more recently, cable stayed.

In the early days of the design and detailing of concrete bridges it was in the hands of named individuals, such as Messrs Freysinnet or Hennebique, who personally supervised the construction of their designs. But then, when there was a huge expansion of the roads programme, design moved into the hands of ordinary engineers working to the then current codes and standards and their own experience. At the time it was believed that deck joints could be sealed adequately, so no attempt was made to protect the concrete of the beam seatings or the bearing shelves.

At around the same time, because of the increase in motor traffic and the fact that more modern, faster cars were becoming available, the practice of salting roads to prevent icing in winter became more widespread, to prevent accidents.

The bearing shelves were almost always detailed as being flat, so water was able to pond and soak into the concrete. The chloride ions in the salt migrated to the reinforcement and changed the chemical environment, causing the steel to corrode and break off the concrete cover.

An example is a cross-head of an elevated road in the Midlands. As a trial, an existing flat-topped cross-head was modified by the addition of a screed on the top surface which fell in such a way as to shed water.

The costing showed this to be very expensive in terms of access and man-hours, but the result was that the condition of this cross-head remained good, while others which were left with a flat top suffered all of the effects of chloride penetration. Had the top been shaped to shed water from the beginning then the expenditure could have been avoided.

4. The importance of sealing deck joints

Before the boom in motorway construction little attention had been paid to the sealing of deck joints. Long span railway bridges were generally of arch form, which did not have joints, and long metal railway bridges had open deck joints, because the running rails spanned them and they were not detected by the rail vehicles. Any water going through the joints was picked up in gutters, or launders as the railwaymen call them, and taken away to a drainage system. Other than possible frost damage, this was never a problem, since the water from the permanent way did not contain salts, and the engineering brickwork commonly used for the abutment was not particularly susceptible to chemical attack. However, open joints are not desirable in road bridges because of the noise of tyres when they cross them and the potential for joints to be prevented from closing by any debris that may fall down them.

Deck seals were intended to reduce noise and prevent the ingress of debris, but were also intended to prevent surface water from reaching the structure beneath. (Figure 2).

The pursuit of a joint which does not leak has led to many designs. The design depends on the expected movement of the deck. In Britain this is covered by a Departmental Standard BD 33/94 (1994), which recommends buried joints for less than 20mm, asphaltic plug joints for the range 5mm to 40mm, nosings with a poured sealant 5mm to 12mm, which is less than a buried joint, nosing with a preformed compression seal 5mm to 40mm, and proprietary joints above that, according to their design. Over the years, however, there have been varying degrees of success.

Various methods have been devised by design engineers using the materials available at the time. In the early days of motorway bridges, steel angles to retain the surfacing were bolted to the deck and a sealant put into the gap; epoxy concrete was tried as a nosing to retain the surfacing, again with a sealant in the gap, as was fibre reinforced concrete but these were largely unsuccessful.

A successful development in the late 1970s was the "asphaltic plug joint" (APJ), which by the addition of a polymer to bitumen binder was rendered stable in summer but flexible in winter, so is ideal for use as a deck joint.

There are drawbacks to APJs, so there have been further developments.

"Elastomeric in metal runner" deck joints were developed for larger deck joint movements but the weak point of these is that the bolts which hold the frame to the deck eventually work loose.

Combining the idea of "Elastomeric in metal runner" with polymer concrete nosings has largely been successful, but performance does depend on the workmanship of the nosing installation. (Figure 3).

However, the maintenance of a deck joint, particularly the removal of grit which accumulates with time, is crucial to its life, since any trapped grit compacts and prevents the joint from acting properly when it closes with the seasonal increase in ambient temperature, and in many cases causes damage to it.

Joints for very large movements of decks are a subject of their own.

Their purpose is to provide a smooth surface for the passage of traffic, while controlling any water passing through it such that it is channelled away. Various different designs are available from a number of manufacturers, and involve various principles including beams which span the gap supporting plates, or lazy tong arrangements which again support plates, or a number of transverse plates which are guided in rails to beneath the deck as the bridge moves.

FIGURE 2
RESULT OF LEAKING DECK JOINT.

FIGURE 3
FAILURE OF METAL RUNNER FIXING IN NOSING.

STRUCTURES

5. The life of bridge bearings

In the early days of motorway bridge design short span structures were given a free and a fixed end, to cater for temperature expansion and contraction in the longitudinal direction. But often the fact that the bridge was wider than its span was overlooked, and failure or displacement of elastomeric bearings occurred because the design movement was exceeded, so either the bearing split, or it moved bodily from its original position. 30 or 40 years on, the restoration of the bearings to their original position is a major exercise frequently involving lifting of the entire bridge deck.

Sliding bearings usually consist of a layer of PTFE bonded to a steel substrate, which slides against a polished stainless steel plate that is also bonded to a steel substrate. Such bearings have a measurably limited life, which can be derived from the number of cycles they undergo, and the distance they travel. The life can be extended by greasing the bearings, but wear is inevitable, and the bearings must be replaced, which involves lifting the bridge. The cost of lifting the bridge can far outweigh the cost of the bearing itself, therefore the easier it is made at the design stage, the less it will cost in the future.

6. Avoidance of difficult details which can be badly constructed

There is often a conflict between the needs of the long-term design and the method of construction. It is not uncommon for a contractor to ask for design details to be altered or simplified to suit his methods, therefore, if the designer uses his experience to detail his structure to be easily constructed he will reduce the need to change it under the pressure of having to do a redesign within a construction programme.

The precasting of parts of a bridge should be considered. Unless it is a very complicated structure which cannot be simplified, a significant proportion of a bridge deck can be precast. It is normal to pre-cast beams, but, if properly detailed, diaphragms can also be precast, the main problem then being to connect them to the main structure in such a way that they become an integral part of the bridge. The advantage of this is reduced dependency on workmanship and reduced time on site. Eurocode EN4 (2005) allows shear connectors to be clustered, which opens the door to precasting composite concrete decks over their whole width.

7. The comparison of initial cost versus long-term maintenance

It must be assumed that no structure can be left for its design life without any maintenance being required. Steelwork must be painted, deck joints fail, bearings wear out and despite the designer's best efforts, concrete may deteriorate. Comparative maintenance costs for steel and concrete structures are evaluated in the British Departmental Standard BD 36/92 (1992).

Where access to a bridge is easy, and jacking up the deck is comparatively simple, bearing replacement is not a problem, particularly since it has long been the case in Britain that provision should be made in the original design for the future replacement of bearings. However, costs escalate when access is difficult.

As an example, a major viaduct over a river estuary in the south of England was built in the 1970s with PTPE on stainless steel sliding bearings.

Because the bridge as constructed did not have provision for bearing replacement, the estimated cost of the exercise of replacing the bearings is over £2m, of which just £20,000 is the cost of the bearings.

In designing a bridge to last 120 years, which is the case for structures in Britain, it is necessary to acknowledge that at some time in its life either the bridge must be lifted to allow replacement, or the bearings must be designed to last that length of time.

An approach to be considered is to change the philosophy of bearing design. Normally bearings are designed to the stresses allowed by the codes or standards, but this sets the stress level at the maximum for the material chosen; an alternative approach would be to overdesign the bearings, by making the contact areas bigger to reduce the stress. The effect of this would be to increase the cost of the bearings when first purchased, but this could be set against the savings in cost of replacement.

8. Preserving the life of bridges

With the experience built up over the last few decades, there is much that has been done in terms of specification to prolong the life of a bridge, but there is also much that can be done in terms of detail.

With regards to concrete bridges, the current specification for concrete attempts to eliminate corrosion of the steel due to carbonation by increasing the cover, the aggregates and cement are specified such as to avoid alkali-silica reaction, additives such as calcium chloride which are deleterious to the concrete are forbidden.

In addition to this, exposed areas of concrete which can be subject to chloride or other external attack are normally coated with a layer that prevents the ingress of chlorides or other unwanted material. These layers can either be barriers which are applied to the surface, or liquids which can be applied to the surface and penetrate the concrete to form a barrier beneath the surface.

Cladding the concrete in brickwork or other ceramic material which prevents penetration by rain or surface water has been shown in the past to preserve the structural concrete, although it can become a maintenance liability in its own right.

The deck edge beams can be clad with an environmental barrier, such as GRP panels, which protects them from chloride ingress, but the cladding should be easily replaceable or again it becomes a maintenance liability in its own right.

Decks should be made continuous over piers to avoid the need for deck joints, and abutments should be integral or semi-integral where appropriate, depending on span or skew.

Flat surfaces should be profiled to prevent water from ponding.

With regard to structural steelwork bridges, the concrete deck usually associated with composite bridges can be treated in all of the ways described in the section above, but the steelwork itself should be detailed to avoid the retention of water or debris, including nesting birds.

Corrosion protection is necessary, unless weathering steel is used, but certain areas are more vulnerable than others, and should receive special attention, such as additional coats of paint.

In hostile environments, or where access is difficult, which might be the same location, cladding the entire deck in GRP or similar material should be considered to provide both an environmental barrier and a permanent inspection access which, again, will require maintenance over its own design life.

8.1 The importance of detailing

There are certain rules in addition to those found in codes and standards which, if used for guidance, can go some way towards ensuring the long and trouble-free life of a bridge. These are:

- Assume that water will get in, and let it out
- Do not depend on one agent; assume that the first could fail, and have a back-up
- Assume sealants will fail, for instance, and provide a channel to pick up the water leaking through them
- Detail bearings which have sliding surfaces such that the surface more susceptible to damage from grit is uppermost
- Detail bearings for easy replacement
- Spend money on larger bearings at a lower stress to increase life between replacements
- Eliminate bearings altogether through integral construction where possible.

8.2. The importance of simple maintenance

Repeated simple minor maintenance can avoid major maintenance, or defer it for a long time. Such minor maintenance can be:

- Regular washing of bearing seatings
- Yearly cleaning of drains after the gritting season
- Yearly inspection of movement joints and removal of grit
- Providing drains in closed boxes to let the water out
- Washing down external stiffeners
- Local touching-up of paintwork
- Preventing birds from nesting, or removing their nests once finished with.

9. Conclusion

8.3 Plan for major maintenance

Major maintenance which can be expected is the replacement of bearings and deck joints, which must be planned for and funds set aside. The contribution which the designer can make to make this as infrequent as possible, and as easy as possible, is to:

- Design jacking points into the supports to allow the deck to be lifted to replace bearings
- Design the deck to be lifted from those points; this can induce reverse moments, which will be important in the case of prestressed beams
- Overdesign the bearings to reduce the stress and thus wear on the bearing material
- Use prefabricated deck joints where possible to reduce dependency on site workmanship, which is frequently subject to the extremes of weather
- Detail the deck ends for specific joints to ensure optimum performance

It is unrealistic to assume that once a structure has been completed it can be left unmaintained for the whole of its design life. However, the need for maintenance or repair can be reduced by careful detailing by the original designer and, where components have an identifiable design life, detailing the bridge to make their replacement as simple and convenient as possible.

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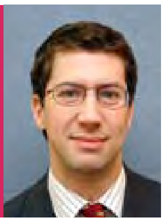
3. Eurocode EN4 2005 – Design of composite steel and concrete structures – Part 2: General rules and rules for bridges. London: BSI Publications, 389, Chiswick High Road, W4 4AL.

The strengthening of Irwell Valley Bridge



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Abstract

The Irwell Valley Bridge in the UK comprises 7 steel-concrete composite box girders with a simply supported span of 60.96 m, at a skew of 23°. It carries the M60 across the River Irwell adjacent to Junction 16 and was constructed in 1970. Structural assessment highlighted deficiencies in the shear studs connecting the concrete deck slabs to the steel boxes, steel diaphragms and end cross bracing. The former was addressed through a Departure from Standards, while the other two deficiencies required strengthening and modification. A means of jacking the boxes for future bearing replacement also needed to be provided.

This paper identifies the various ways in which a cost-efficient and buildable design was produced for the strengthening of the diaphragms and how detailing problems specific to strengthening a steel box with restricted access were overcome. Key to the project was the need to minimise disruption to the highway network. The motorway was therefore kept open during jacking operations and multiple boxes were strengthened simultaneously.

Due to the complexity of the construction sequence for the strengthening work, a virtual reality model of the bridge and all stages of strengthening was set up and distributed to all parties. The benefits of using this model to assist both design and work on site are discussed.

1. Introduction

The Irwell Valley Bridge carries the M60 across the River Irwell adjacent to Junction 16 and was constructed in 1970. The bridge comprises 7 steel-concrete composite box girders with a simply supported span of 60.96 m at a skew of 23° as shown in Figure 1. The boxes are supported on pairs of bearings on reinforced concrete skeletal abutments, comprising leaf piers and cross beams. A longitudinal deck joint separates the two carriageways and divides the deck such that the southbound carriageway is supported on three boxes, and the northbound is supported on four boxes. Each box is stiffened longitudinally by bulb flat stiffeners and transversely by angle stiffeners forming ring frames at 1.7 m centres as shown in Figure 2. The dimensions of a typical box are shown in Figure 3. Studs are provided across the full width of the box to prevent local buckling of the steel top flange but the effects of shear lag cause the studs nearest to the webs to be significantly more heavily loaded than the others under service loads. Longitudinal stiffeners are also provided on the top flange; this was to ensure its adequacy before it was made composite.

Access is provided to each box through manholes in the end diaphragms at each end of the bridge. The main load bearing diaphragms are square to the longitudinal axis of the boxes, but the box extends beyond these such that the boxes are closed off by non-load carrying end plates, also with access provision but parallel to the abutments and thus skewed to the main diaphragms by 23°.

The end plates also provide the location for connection of cross bracing between adjacent boxes. The diaphragms are stiffened by 12.7 mm thick flats, as shown in Figure 4.

The diaphragms at each end of the bridge were initially designed to be the same, even though one end has longitudinally fixed bearings while the other (the free end) permits longitudinal movement and thus eccentricity on the
diaphragm. As a result of the additional eccentricity, the free end diaphragm was strengthened in the early 1980s as a result of the Merrison[1] committee recommendations. This involved plating over the bottom part of the diaphragm (Figure 4) to carry some of the eccentric bearing reaction directly to the lower web panels.

FIGURE 1
LAYOUT OF BRIDGE.

FIGURE 2
INTERNAL STIFFENING
TO BOX

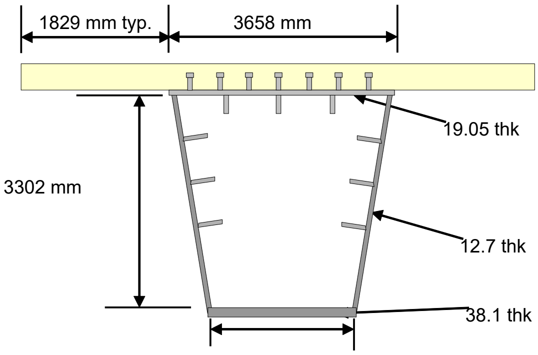


FIGURE 3
TYPICAL BOX GIRDER
DIMENSIONS

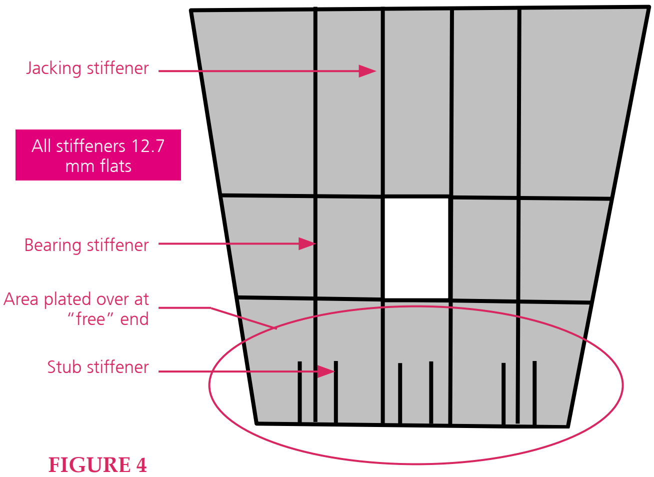


FIGURE 4
DIAPHRAGM
STIFFENING

2. History of assessment

Structural assessment of the bridge was originally carried out by Parkman over a number of years to BD 56[2] and BD 61[3]. The main boxes were generally found to be adequate in shear and flexure but deficiencies were found in the following areas:

- Shear studs connecting the concrete deck slabs to the steel boxes
- Steel diaphragms
- End cross bracing

The concrete abutments were not assessed, despite being significant spanning members in their own right. They were deemed to be satisfactory on the basis of visual inspection.

Shear studs

The shear studs were found to have inadequate fatigue life due to the uneven distribution of stud force across the box determined from BD 61 as indicated diagrammatically in Figure 5. This distribution depends on shear lag and stud stiffness. The formula in BD 61 is based on stud stiffness values assumed by Moffat and Dowling[4]. Finite element analysis with more realistic stud stiffnesses leads to a more uniform distribution of stud force (as given in Hendy and Johnson[5] for example). Parkman commissioned such an analysis which resulted in the studs being shown to be adequate. As a result of the theoretical overstress and to justify this Departure from Standards, acoustic monitoring equipment was installed on the bridge. After extensive monitoring and subsequent testing of core specimens, no fatigue damage to any studs was found.

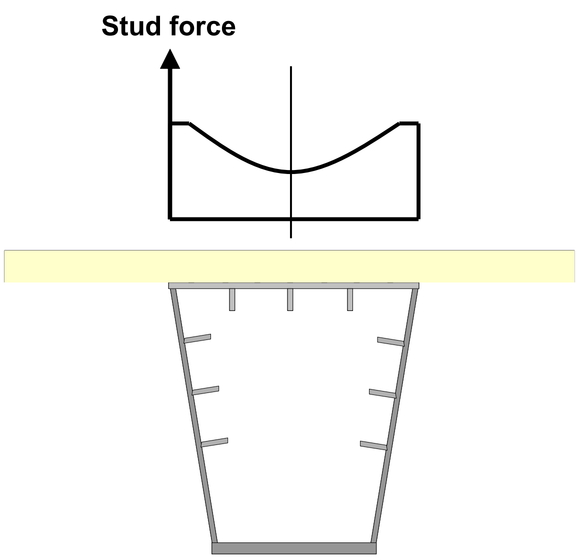


FIGURE 5
STUD FORCE
DISTRIBUTION

Of greater concern was the lack of any transverse reinforcement in the bottom layer of the deck slab across the box top flange. Reinforcement is typically needed to prevent splitting of the concrete ahead of the stud and to control the spread of force across the slab. This deficiency was accepted on the basis of inspections of the slab which found no longitudinal cracks and also a further assessment by Atkins which ignored the shear connection other than in its role of preventing top flange buckling. This indicated that the box would not collapse under live load (with unity partial material and load factors).

Diaphragm bearing stiffeners

The critical assessed elements on the bridge were found to be the diaphragm bearing stiffeners – see Figure 4. These were rated at "dead load only", governed by predicted failures in both buckling and yielding. The central stiffeners adjacent to the access hole were also considerably overstressed if assumed to form "jacking stiffeners" in the event of needing to replace the bridge's bearings. The diaphragm assessment was initially based on BS 5400:Part 3[6] Clause 9.17, but subsequent assessment used elastic FE analysis. Interim measures were taken to keep the bridge open. Parkman installed temporary props inside the box and stools beneath the webs outside the box (Figure 6) as an interim measure.

Bracing

The cross bracing at the ends of the deck are unnecessary for the adequacy of the bridge because the boxes are themselves torsionally restrained by their seating on pairs of bearings. The skew of the bracing relative to the axes of the boxes meant they attracted significant forces due to rotation of the boxes about a horizontal axis, thus overstressing them

3. The strengthening

Global analysis

In July 2002, Atkins took over as the Managing Agent in Highways Agency Area 10, also taking over responsibility for strengthening the structure. The first tasks of the new team were to:

- Re-assess the diaphragms, based on elastic global analysis, to confirm the findings of the assessment
- Attempt to refine the assessment to eliminate the need for strengthening.

Grillage analysis
Initially the bridge was modelled as an elastic grillage, with idealisation of elements as shown in Figure 7, following the recommendations in the SCI Guide 7. A shortcoming of this idealisation is that the single spine member, to which the box longitudinal properties are all attributed, does not model box distortion and so is torsionally too stiff. The excessive torsional stiffness leads to overestimates of torque in the boxes and therefore also to overestimates of the bearing reactions. This was clearly undesirable as it was the large reactions that were leading to the overstress in the bearing stiffeners. It is possible to "soften" the torsion constant to allow for distortion and to reduce the bearing reactions, but unique values of torsion constant would be required for every individual load case. A shell FE model was therefore used to refine the bearing reactions produced (Figure 8).

FIGURE 6
TEMPORARY PROPS INSIDE
THE BOX AND STOOLS
BENEATH WEBS

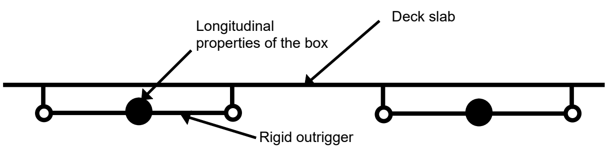
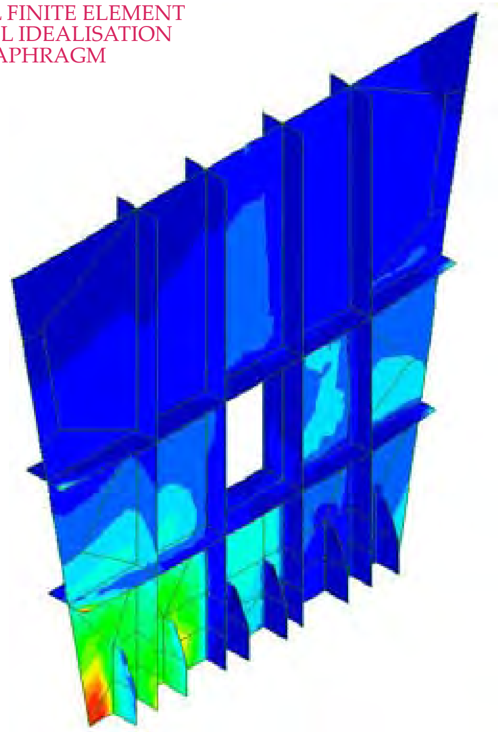


FIGURE 7
TEMPORARY PROPS INSIDE
THE BOX AND STOOLS
BENEATH WEBS

FIGURE 8
SHELL FINITE ELEMENT
MODEL IDEALISATION
OF DIAPHRAGM



STRUCTURES

FE analysis

The shell finite element model produced reasonable agreement on reactions with the grillage model with appropriately softened torsion constants. Subsequent assessment of the diaphragm using these reactions and BD 562 confirmed the diaphragms to be significantly overstressed. The results of the elastic FE analysis suggested further overstresses but these were generally as a result of the fact that BD 56 makes some allowance for plastic redistribution while no such redistribution occurs with an elastic analysis. In particular, the shear stress distribution between the diaphragm and web was found to be distributed very unevenly up the web, with a concentration of the load being transmitted over the lower part of the diaphragm in a classic "shear boot" distribution (illustrated in Figure 9). While this shear can normally redistribute up the diaphragm with the onset of a little plasticity, there was concern that this could not happen in the case of the free end diaphragm because the over-plating at the bottom of the diaphragm was discontinuous. As a result, each lower web panel at the free expansion end of the girders was strengthened by the addition of a longitudinal stiffener adjacent to the diaphragm.

Non-linear analysis is often the next course for such structures as the reserves of strength available with a little plastic redistribution are often considerable. In the case of Irwell Valley Bridge however, it quickly became apparent that there was little redundancy in the diaphragm. Failure followed shortly after initial buckling of the main load bearing diaphragms and non-linear analysis was therefore not able to demonstrate any significant increase in load rating.

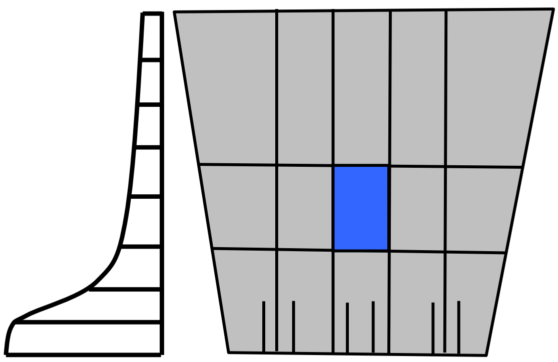


FIGURE 9
ELASTIC SHEAR STRESS DISTRIBUTION AT WEB/ DIAPHRAGM JUNCTION

Strengthening design

Fixed end

A permanent solution for strengthening the main bearing stiffeners was needed. Since the jacking stiffeners were also shown to be overstressed, a means of facilitating bearing replacement in the future was also required. Prolonged closure of the motorway, other than single lane closures at night, was not considered acceptable because of the disruption to the network. Therefore load needed to be jacked out of the bearing stiffeners before any modification to them could take place; both welding and drilling holes for bolting to them would temporarily weaken them further.

The initial concept for strengthening was first to strengthen the existing jacking stiffeners and then use these to jack under the middle of the box to unload the permanent bearing stiffeners. However, further investigation of this option revealed a number of difficulties:

- Very heavy additional stiffening was required to the existing jacking stiffeners each side of the access hole. This would have restricted further the already tight access hole
- The large size of the necessary steel plates would have been very difficult to erect within the confined space
- The jack size required for a single point lift was very large and could not readily be accommodated in the restricted space available between underside of box and top of abutment
- Single point jacking beneath the centre of the diaphragm overstressed the diaphragm plate panels in hogging bending
- Single point jacking would have required the boxes to be lifted well clear of the bearings or, under traffic loading, torsional rotation of the boxes would have caused the box to impact the bearings. It was not desirable to jack the boxes much clear of the bearings because of the risk of damaging the movement joint above.

The chosen solution was therefore to jack under the centre of the box and also at the webs via new inclined external jacking stiffeners. The central jacking stiffeners would carry most of the dead load while the outer jacking stiffeners would carry most of the live load. The central stiffeners at the access hole still needed strengthening first even to carry part of the bearing loads. Bolting was chosen to make as many of the new connections as possible, both because of the health & safety risks associated with welding inside a confined space and because of the potential for laminating the existing BS 15[8] steel. This steel is potentially more brittle than modern steels due to the absence of any specified minimum Charpy impact energy. Initial site surveys had already found evidence of laminations in some of the existing steel plates.
Most of the preparatory works for the strengthening could

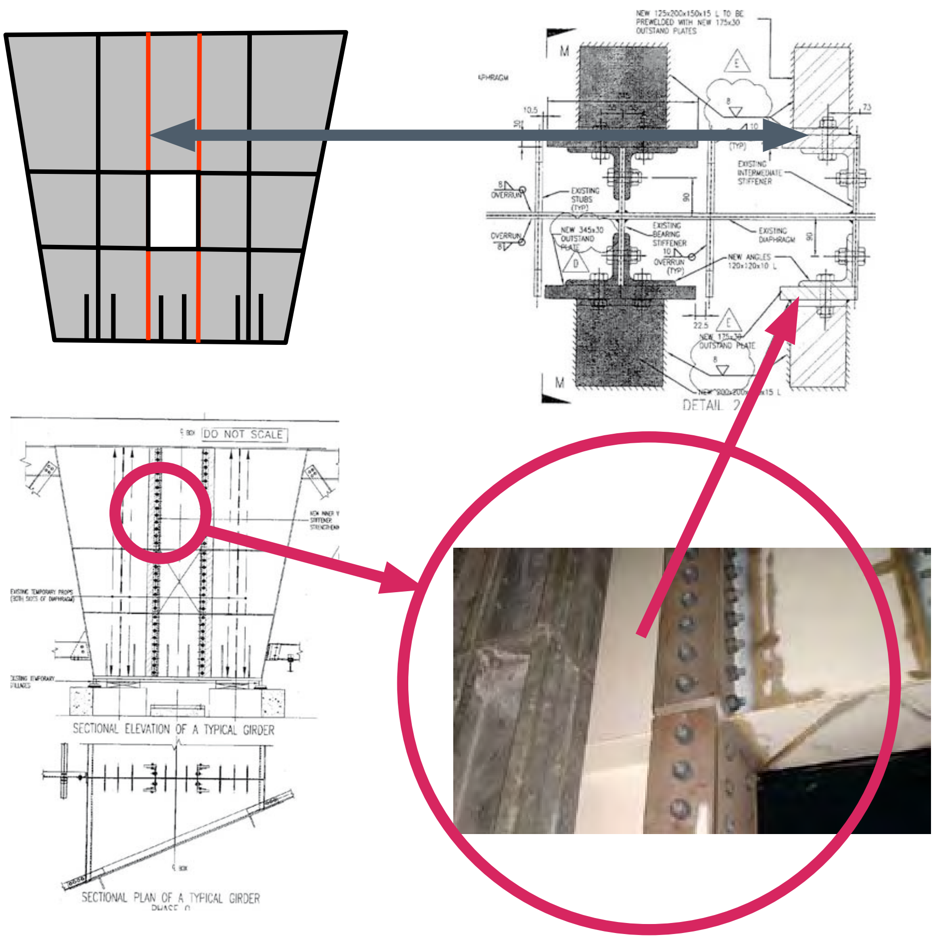


FIGURE 10
STRENGTHENING TO EXISTING JACKING STIFFENERS EITHER SIDE OF ACCESS HOLE

be done without any lane closures. First, the central jacking stiffeners were drilled ready to receive new outstand plates which were connected via angle cleats, as shown in Figure 10. Each new outstand plate was connected via cleats to the top and bottom flange to prevent lateral buckling. The plates were fitted to the bottom flange to allow the jacking load to be taken into the stiffener in bearing.

Beneath the centre of the bottom flange, an existing bearing plate was provided as shown in Figure 11 but, curiously, this was not wide enough to spread the load to the main jacking stiffeners each side of the access hole, only to the stub stiffeners directly beneath the access hole. The bearing area therefore needed to be enlarged. This was done by welding a plate with a central cut out around the existing plate. A capping plate was then placed beneath both plates and any small gaps filled with epoxy resin (Figure 12).

The temporary jacking system beneath the centre of the box needed to provide the same articulation as the existing bearings, because otherwise the fixity of the box would be lost once the existing bearings were unloaded.

This led to the need to provide the fixity shown in Figure 13. To achieve this, a temporary bearing with the same fixity as the permanent bearings at the particular location considered was mounted on a jack as shown in Figure 12. To lock the jack and bearings in position together, "keep" plates were provided as shown in Figure 14.

Before the new web stiffeners could be placed, it was necessary to enlarge the existing bearing plinths to provide sufficient area to place the jacks. This was done by breaking out the existing plinths to enable new reinforcement to be lapped in. This was then also drilled and resin-fixed into the abutment as shown in Figure 15.

Since the new outer jacks were loading the abutment cross beams at a new location where the beam was less deep, the cross beams themselves were assessed. This revealed an overstress in combined torsion and shear of the same magnitude as that of the diaphragm. Further investigation indicated a theoretical overstress in the permanent situation also. Many options were considered to solve the overstress in the temporary situation, such as propping the cross beam from the ground, but the solution needed to address both temporary and permanent problems.

FIGURE 11
EXISTING BEARING PLATE
BENEATH BOTTOM FLANGE

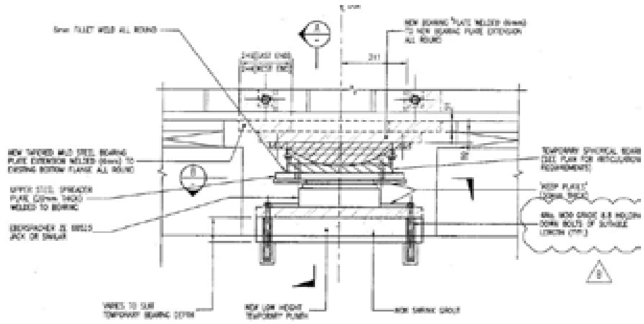


FIGURE 12
CENTRAL JACKING
ARRANGEMENT

The final solution was therefore to locally extend the cross beam soffit down to provide adequate resistance as shown in Figure 16. Reinforcement was drilled into the abutment wall and cross beam using automated mechanical drilling equipment to eliminate the risk of hand vibration injury. The drilled-in reinforcement was detailed to supplement the shear and torsion resistance of the existing cross beam. Anchorage lengths were determined using a draft HA Interim Advice Note[9].

Once the central jacking stiffeners had been strengthened and the bearing plinths had been widened, a nominal load of 1000 kN was applied to the central jack to relieve some load from the permanent bearing stiffeners. This additional redundant support allowed the temporary props and the temporary stools to be removed. The props needed to be removed at this stage to get access to the area where the new web stiffeners were to be attached. The end cross bracing was also removed permanently at this stage. It was not required structurally but its presence attracted forces which structural assessment indicated it could not carry. This was evident by visible bowing in some of the bracing members.

After the central jack preload was applied and locked off, the webs were drilled to receive the bolts to attach the new outer web box stiffeners as shown in Figure 17(a). The stiffener bolt holes for attachments were kept behind the diaphragm to avoid reducing the web shear resistance; the webs were already highly stressed.

The new web stiffeners were fabricated box sections comprising a channel section with a bolted closing flange plate. These were constructed in weathering steel so that the inaccessible internal surfaces of the box stiffener need not be painted or be made overly thick to make allowance for corrosion. First the fabricated channel was bolted up to the web as shown in Figure 17(b). The channel had a welded top plate already fixed for subsequent connection to the box top flange to give transverse restraint. Then the bottom base plate was welded to the channel section

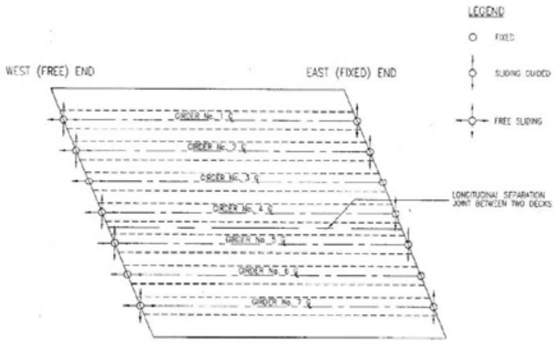


FIGURE 13
DECK ARTICULATION

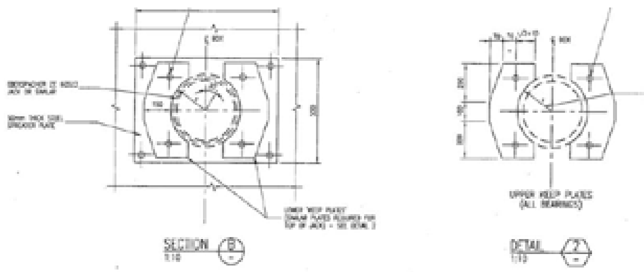


FIGURE 14
“KEEP” PLATES TO HOLD JACK IN
POSITION AND FIX BEARING TO JACK

FIGURE 15
EXTENSION OF BEARING PLINTHS

FIGURE 16
STRENGTHENING TO SUBSTRUCTURE

and the bottom flange of the box to provide transverse restraint to the stiffener and a defined flat bearing area, and the outer flange plate was bolted to the channel section via cleats – Figure 17(c). Finally, the top flange plate was welded to the top flange of the box, again to provide transverse restraint.

The design of the outer web jacking stiffeners was outside the scope of BS 5400:Part 36 because they were both single sided (and thus eccentric to the web) and also not vertical. Both of these unusual aspects gave rise to special design considerations:

(1) Single sided stiffener
The distribution of moment up the stiffener depends on how the shear is carried up the height of the stiffener. If the shear is carried uniformly up the height of the stiffener and the load acts at the centroid of the stiffener effective section (including attached web plate), no moment is produced in the stiffener as shown in Figure 18 (a). Any eccentricity from the centroid causes moments as in a normal symmetrical stiffener as shown in Figure 18 (b). However, if the shear is carried non-uniformly up the height of the stiffener (with more shear carried in the lower portion of the diaphragm) and the load acts at effective section centroid, some moment is developed as shown in Figure 18 (c). Variations in distribution of shear up the web do not normally give rise to stiffener moments where the stiffeners are symmetrical about the web. Any eccentricity from the effective section centroid causes additional moments as in a normal symmetrical stiffener as shown in Figure 18 (d). Allowance was therefore made for uneven shear distribution up the web in design. This was compared with the design stresses based on the results of finite element analysis, which gave results similar to that shown in Figure 18 (d).

(2) Non-vertical stiffener
The non-verticality of the stiffeners leads to transverse compression in the bottom flange of the box which then needed to be checked for buckling. This check was easily accommodated because the bottom flange was relatively thick and the width of the flange plate transversely between diaphragm and end cover plate was small.

After the installation of the web jacking stiffeners and the central diaphragm stiffeners, the box was jacked under night-time lane closures. Lane closures were necessary to permit accurate monitoring of the jacking loads against anticipated dead loads. The central jack was loaded to 2500 kN first. The jack was then floated at this load and the outer jacks were incrementally loaded until both permanent bearing loads were reduced to zero. Lift off at the bearings was determined from load deflection plots and the outer jack loads were limited to a maximum of 500 kN, which was not exceeded. All three jacks were then locked off. The outer bearing stiffeners subsequently carried the majority of the live load when the lane closures were taken off and the central stiffeners carried only a small part of the live load.

(a) (b) (c)

FIGURE 17
ATTACHMENT OF OUTER BOX JACKING STIFFENERS

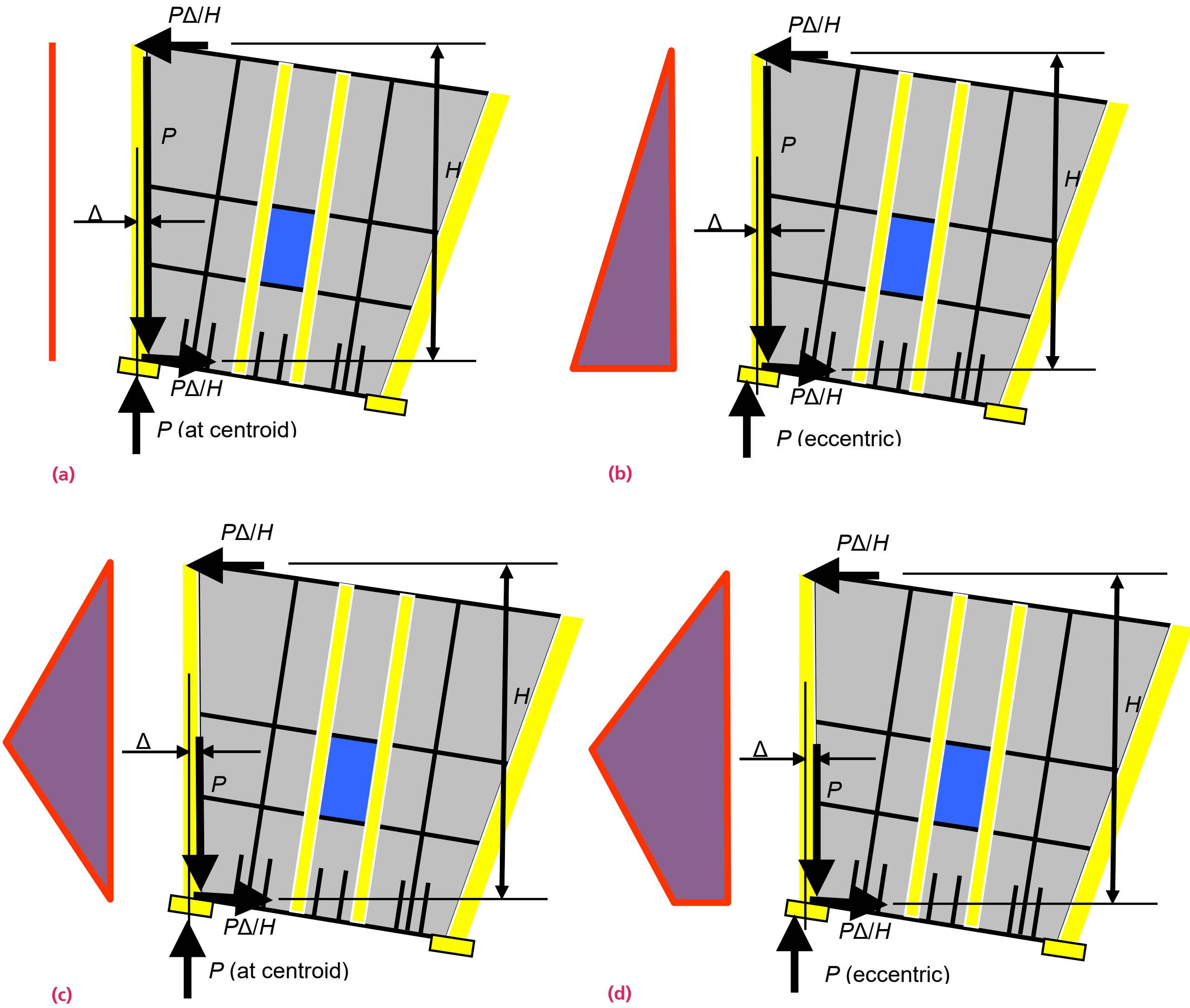


FIGURE 18
DISTRIBUTION OF MOMENT IN WEB JACKING STIFFENERS FOR DIFFERENT ECCENTRICITIES AND SHEAR DISTRIBUTIONS UP THE WEB

After the jacks were locked off, installation of the strengthening to the permanent bearing stiffeners could begin, as shown in Figure 19. It was necessary first to over-run the welds between the diaphragm and the flange and the stub stiffeners in order to give adequate fatigue performance. Holes were first drilled through the existing stiffeners to receive angle cleats and then the main stiffener flange plates were added by bolting to the cleats.

The top and bottom of each stiffener flange plate was also cleated to the top and bottom box flanges respectively to give transverse restraint. This was necessary because the stiffener flange plates were not welded to the bottom flange; it was not physically possible and large welds would have been undesirable. Instead, the stiffeners were closely fitted and all bearing loads were carried into the stiffener in bearing.

3.2.2 Free end strengthening sequence
The free end strengthening followed a very similar sequence and detailing but welding of stiffeners was needed in places due to the presence of previous strengthening in the 1980s which restricted the access for bolting. Welding needed to be carefully controlled, particularly for connecting web stiffeners, because the heat weakens the metal in the web and reduces the shear strength and stiffness. To minimise this effect, it was required for welding to be carried out in such a way that the height of web material experiencing a temperature increase in excess of 200˚ at any instant be less than 500 mm. The reduction in strength and stiffness associated with this temperature was determined in accordance with EN 1993-1-2[10]. Weld procedure trials were carried out to confirm this was possible and also to ensure that the electrodes were matched to the BS 15[8] steel.

3.2.3 Construction sequence for all boxes
For programme reasons, more than one box needed to be worked on at any one time. The system of replacing the permanent bearing restraints with equivalent temporary ones permitted all boxes to be worked on and jacked at the same time. Lane closures on the M60 motorway were required for any welding and jacking but these were limited to overnight closures only; the client (Highways Agency) placed great value on minimising network disruption. These lane closures could only be implemented between 9pm and 5am, including time for installation and removal of the Traffic Management system and the cooling of the welds. Effective programme management was therefore essential to the successful delivery of the scheme.

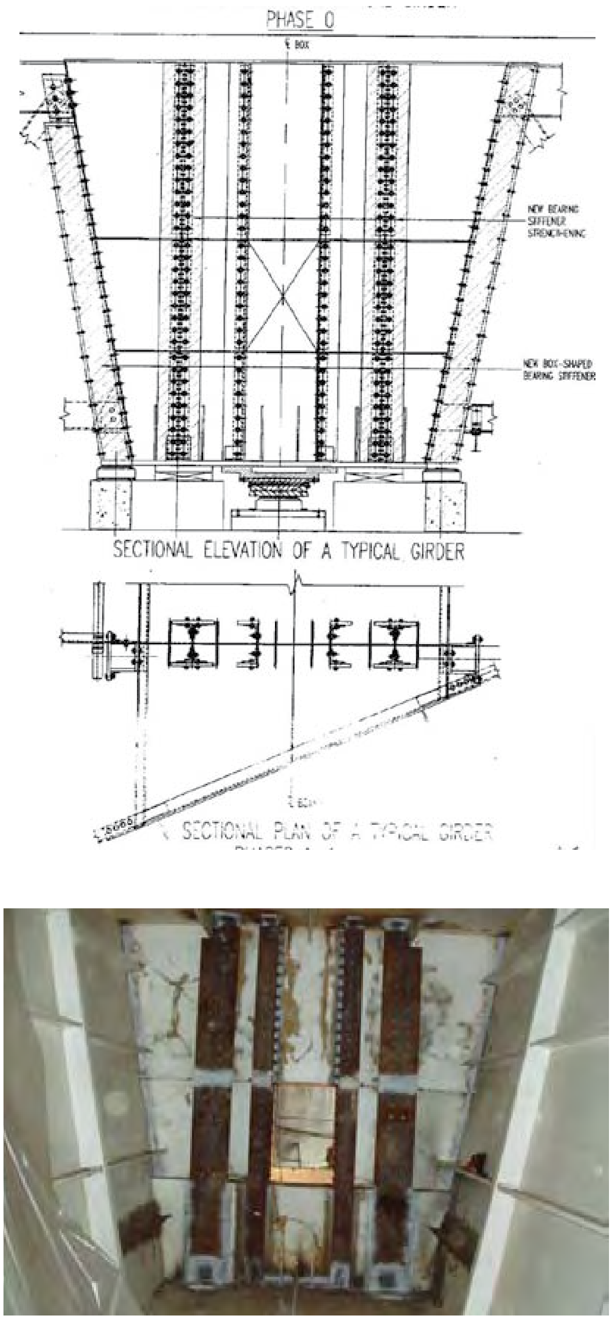


FIGURE 19
FINAL STRENGTHENING
ARRANGEMENT

4. Virtual Reality Model

The Designer's Risk Assessment identified a number of hazards in the design which were combated at source. These included welding in the confined space (bolting was generally used) and handling of excessively heavy steel plates (steelwork was split into smaller components). However, the complex construction sequence itself was highlighted as a significant risk because execution of certain activities in the wrong sequence had potentially very severe consequences. By way of mitigation, it was decided that a virtual reality (VR) model should be prepared. This would allow visualisation of the strengthening process and a means of stepping through the stages of construction in the correct sequence.

In the event, the VR model (Figure 20) proved to have far greater benefit than simply tracking the construction stages and reducing the risk of working out of sequence. This was formally acknowledged by the designer's site team, specialist contractors and their supply chains. The benefits included:

- Enhancing visualisation and understanding of the strengthening works – engineers new to the project, whether designers or contractors, were able to understand the job much more quickly by taking a virtual tour of the boxes and stepping through construction stages than by studying the drawings in isolation
- Facilitating quick location of elements referenced on the drawings – each element, whether jacks or strengthening, had a unique reference number which could easily be called up and located in the VR model
- Facilitating progress monitoring – progress could easily be measured against the construction stage numbers in the VR model
- Detection of clashes and fit problems – some of the strengthening details were modified at an early stage after clashes with existing steelwork were detected. These were obvious when viewed in three dimensions but had not been detected when the two dimensional drawings had been prepared
- Warning of highly stressed elements at each stage – colour coding was used in the VR model to indicate highly stressed areas. This acted as a deterrent to drilling through or welding to components at the wrong time
- Keeping track of traffic management requirements – at each stage of construction, the traffic management requirements were clearly indicated in a window of the VR modeller as shown in Figure 20.

As a result of the success of the VR model on this project, the authors are now instigating measures to make such models a standard deliverable for similar projects.

5. Acknowledgements

This paper is published with the permission of the Highways Agency and John Martin Construction. The authors would also like to acknowledge Gary Knowles, Atkins Area 10 Project Manager, for his effective coordination of the numerous parties while the strengthening work was being procured and on site.

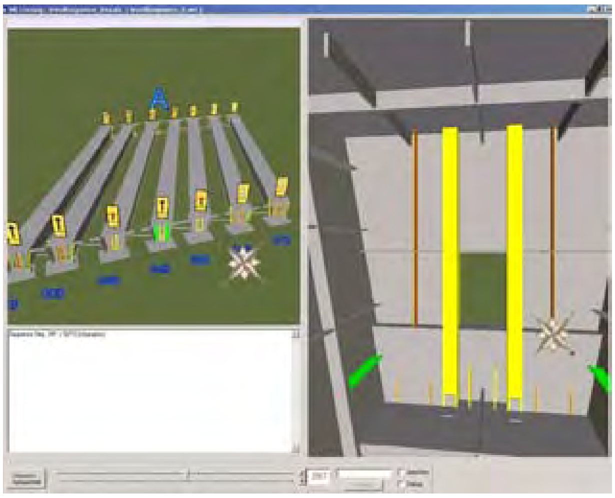


FIGURE 20
VIRTUAL REALITY MODEL FOR
THE STRENGTHENING SCHEME

STRUCTURES

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Critical infrastructure resilience

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Abstract

This paper discusses concepts for assuring and implementing a resilient infrastructure to support critical incident management activities. The paper draws upon Atkins' experience of planning, designing and enabling a range of transport systems solutions.

1. Network management services

Road operators, national or local, seek to proactively manage the road networks they are responsible for. Delivery requires that the operator proactively monitors and manages road space and incidents, undertakes effective incident management and takes measures to address causes of congestion and improve safety.

Dependent on the scope of service provided by the operator, delivery of such services may require involvement of numerous agencies such as police, maintenance agents and the like.

2. Infrastructure for service delivery

The framework for service delivery is an infrastructure that is not just limited to technology but extends to a number of areas:

- Monitoring, control and communications: traffic management and control systems CCTV, automatic incident detection, communications networks, telephony services, mobile and radio communications, operational staff
- Resourcing and management: command and control tools, general office systems, support functions
- Roadside assistance: vehicles, on-road patrols, training
- Processes: dealing with events and incidents, general behaviour, contingencies.

Systems and processes are integrally linked to facilitate service delivery. Unavailability of one part of this infrastructure affects others.

For instance, unavailability of the traffic control system may prevent a control centre from setting signals at the roadside. This, in turn, may make it unsafe for an on-road resource to enter a carriageway safely to deal with an incident.

Unavailability of traffic control systems could compromise not only the ability to deliver roadside information, but also the ability of a local resource to deal with an incident safely.

It is essential, in this and other similarly critical activities, that systems and processes are resilient, that probabilities of failure are diminished, and that business continuity processes exist to enable continuance in the circumstances of failure.

3. Resilience and business continuity

Resilience and business continuity are closely allied concepts as can be seen from the dictionary definition of "resilience":

- The power or ability to return to the original form, position, etc., after being bent, compressed, or stretched; elasticity
- Ability to recover readily from illness, depression, adversity, or the like; buoyancy.

Resilience and business continuity are of critical importance to service delivery and there is an emerging British Standard that addresses such issues; BS 25999-1 Code of Practice for business continuity management.

Whilst this standard addresses generic business operations, much of it equally applies to emergency or incident management operations.

Looking at resilience, all factors that influence successful delivery need to be addressed including technology, personnel issues, and contingent measures.

4. Technical considerations

Specifications for technology utilised within critical operational environments will undoubtedly stress the required availability of a system. In both specifying and implementing such systems, engineers have to consider carefully what is required and how to meet the requirement. Typical considerations might include:

- What is the consequence of system failures?
- How much down time can be tolerated?
- Is redundancy in systems required?If so how will this be achieved?
- Are Hot/Warm/Cold standby systems appropriate?
- Is geographical separation of servers needed?
- Are alternative communications paths available?
- Do standby paths have sufficient bandwidth to take over all operations?
- Should alternative suppliers be considered?

All of these questions and more are familiar to engineers. However, it is not just the equipment itself that needs to be considered in terms of resilience. The environment the equipment is located in also has to be considered. For instance:

- How is power being provided?
- Are the supplies clean?
- Are supplies secure?
- What UPS support is required?
- Is generator back-up needed?
- Is the building secure?
- Is there adequate cooling of the equipment?
- Is fire protection available?
- Were buildings designed to meet the requirements of a critical operation?

The latter question needs to be asked not just in terms of equipment, but personnel too. It should not be forgotten that a 24-hour operation increases the demands upon a building in terms of the facilities that need to be provided.

5. Personnel factors

In addition to engineering issues, it is necessary to extend thoughts to the personnel factors in providing resilience. Again this is illustrated by listing some of the questions that need to be considered:

- Illness: in the event of staff illness can operations continue? What can be automated?
- Staff retention: What can be done to retain trained, experienced staff? How does one avoid a specific individual having an irreplaceable role?
- Malicious activities: are adequate security processes in place to prevent malicious attack to buildings, staff or systems? What can be done to minimise the impact of such instances?

All of these factors may need to be considered by engineering teams as some, not all, may have a technology solution.

6. Assuring resilience

To assure resilience, all aspects of a critical operation need to be reviewed. The audit needs to ask key questions e.g.: What do users need to maintain continuity? What is the minimum acceptable service? What processes are in place if facilities are compromised? What scenarios are likely to occur?

Following an audit an evaluation of what is needed in terms of technical measures and processes needs to be undertaken. From this evaluation a suite of contingent measures and proposals will emerge. Subject to budgetary and operational constraints a programme of implementation needs to be put in place. This programme has a number of distinct phases:

- Development: both systems and processes to service gaps. Perfect solutions will not exist and the aim should be to ensure minimal impact of failure
- Implementation: priorities for roll-out should be set, often based upon likelihood of an event. It should be paramount that roll-out does not affect ongoing operations
- Test: measures need testing effectively. Testing should be progressive, starting with limited tests that minimise disruption to the ongoing business
- Awareness and training: all affected staff and third party stakeholders must be made aware of technical and procedural contingencies. Training exercises need to be in place
- Ongoing validity: resilience must be considered as part of change control processes to ensure technical and procedural measures continue to be valid. Furthermore an ongoing programme of test exercises needs to be developed with results audited to highlight and correct issues arising.

As part of this latter point it is worth undertaking, at intervals, exercises to review the resilience characteristics of all operations against user expectations to continually question whether actual provision meets ongoing needs of the service.

7. Surety, an enabling approach

Whilst resilient operations are essential, there are many other pressures on delivery of a mission critical service. As well as being resilient to failure, such services have to be delivered in a safe, secure and environmentally acceptable manner.

The identification, assessment, management and performance in these nominally diverse issues are synonymous and to this end Atkins has developed the concept of "Surety", an enabling approach to achieving and measuring such issues.

Surety has seven key principles that should be applied to service delivery

1. Prediction and proactivity
2. Prevention
3. Protection and containment
4. Preparedness and response
5. Recovery and restoration
6. Organisation and learning
7. Continual enhancement.

Immediately the parallels with resilience issues addressed in this paper can be seen. The holistic benefits of such an approach include:

- Comprehensive recognition, ownership, communication and mitigation of major adversities
- Cost effective management of key risks
- Reduction in accidents, incidents and consequent losses
- Effective communication of proactive performance indicators
- Effective management of safety, security and environmental performance
- Enhanced preparedness and response to adversities and improved recovery.

Surety has implemented a 'standard' range of modules to:

- Appraise, address and manage the issues
- Measure results
- Implement performance improvements
- An outcome of the task may be a range of performance indicators that stretch across a business and allow measured performance to evolve and improve.

8. Summary

A resilient operation is essential for delivering a successful incident management service. Service continuity requires a resilient supporting infrastructure. This is achieved through design, audit, evaluation and improvement. In reviewing resilience the same principles must apply to all aspects of a business. As such, a holistic, structured approach to service delivery can help deliver a resilient robust operation.

The future of traffic management on high speed roads

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Abstract

This paper provides a snapshot of the current state-of-the-art of traffic control on high speed roads and outlines some of the issues related to the future of traffic control on high speed roads.

The key to successful systems is a flexible approach to addressing particular problems at specific locations, at given times via combined and coherent use of available traffic control measures, based on Intelligent Transport Systems (ITS). The form of control will be based on the available physical infrastructure used in different ways, at different times. The traffic problem may change over time, which requires flexibility in the control algorithms to cope with changing circumstances.

1. Introduction

The design and operation of traffic control on high speed roads is based on a balance between three objectives namely:

- To improve efficiency – reducing congestion and journey times
- To improve safety – reduce the number and severity of accidents
- To produce environmental benefits – reduce emissions

A positive result in all three areas is the aim, however, under particular circumstances it may be sensible to prioritise one form of benefit over another (e.g. a reduction in speed to reduce accidents and emissions may result in increased journey times). The aim of this paper is to discuss the relationship between particular control interventions and the impact on efficiency, safety and environment. A number of high speed road control measures have been piloted recently. The next section reviews the outcome of those pilots.

2. Pilot results

Across the world a number of innovative control measures have been piloted recently, this section reviews the results of some of these pilots.

Variable speed limits

A number of projects have utilised variable mandatory speed limits. The M25 pilot began in 1995 and a Business Case for this system was completed in 2002 (Rees T[1]). The original aims of the scheme were to provide a smooth traffic flow, to improve journey times, journey time reliability and lane utilisation and to reduce the incidence of stop-start driving and the stress of driving. Subsequent notable achievements also included the development of proven technology and the reduction in environmental problems (i.e. noise and pollution).

During the monitoring of the variable mandatory speed limits (known as controlled motorways (CM)) there was an improvement in journey times on the clockwise carriageway in the morning peak period. The observed behaviour was attributed to smoother driving as drivers became familiar with the CM and drove more smoothly to prevent flow breakdown. Other areas of the M25 did not show as much improvement in journey time performance because congestion has increased, which was attributed to yearly increases in flow levels (more road users).

The number of shockwaves (a significant factor in the causation of accidents) decreased between 1995 and 2002, with a reduction from a typical 7 shockwaves per morning rush hour down to a typical 5. This observed behaviour could be attributed to the smooth driving behaviour or improvements to the control system over this period.

Evidence of safety improvements was demonstrated by the studies of injury-accidents. The CM has resulted in steadier and less stressful journeys and thus reduced the number and severity of accidents. Injury accidents decreased by 10% and damage only accidents by 30%.

Emissions have decreased overall by between 2% and 8%. The weekday traffic noise adjacent to the scheme has reduced by 0.7 decibels.

Lane utilisation and headway distribution have been improved. Lane utilisation became more balanced making better use of the road space, with a reduction in the number of very short headways (pre 1995).

The aim to reduce the stress of driving appears to have been achieved, with the primary road users indicating a positive response to the CM scheme. The driver opinion survey carried out in the first year of operation indicated that over half the respondents noticed an overall improvement; two thirds would like to see the scheme extended to other stretches of motorway. The survey indicated very clearly that the automatic speed cameras were essential to ensure drivers' compliance with speed limits.

Overall variable mandatory speed limits can provide benefits in terms of safety and environment and journey time reliability. The fine tuning of the set up of the control system is key to maximising the particular economy, safety or environmental benefit objectives.

Hardshoulder running

Pilots into the peak hour use of the hard shoulder have been undertaken in The Netherlands, Germany, France and most recently England. The evaluation of the first six months of the English pilot can be found in Highways Agency 2007[2]. The key findings from the study were as follows:

- Observed capacity increased by between 7 and 10%
- Journey time variability improved by between 27 and 34%
- Carbon Dioxide emissions reduced by 4%
- Oxides of Nitrogen emissions reduced by 5%.

Ramp Metering

Ramp Metering (RM), has been implemented in the United States and in Europe over a number of years. Most studies have shown an improvement in throughput and a reduction in journey time following implementation of RM.

The Highways Agency RM pilot project (Highways Agency 2005[3]) found that:

- Downstream flow increased by up to 5%
- Upstream speed increased by up to 18%
- Period of flow breakdown was shortened in the peak by up to 20 minutes
- Care must be taken over site selection as ramp metering did not always have a positive impact.

RM systems have operated in isolation from other control systems, the next section highlights the issues related to combining RM with other high speed road control systems.

3. Potential for combining systems

Over a number of years Traffic Wales has implemented an ITS programme capable of collecting, processing and delivering traffic information to road users. These current ITS methods help reduce congestion and improve journey times.

At present Wales contains two primary corridors, the M4 in the south and the A55 in the North. Both routes, including their associated trunk roads, are on the Trans-European Road Network (TERN) and are key strategic routes.

One potential enhancement is to combine the existing separate and distinct features into a coordinated approach. One element of this approach could be to combine ramp metering with the operation of variable mandatory speed limits.

A recent study investigated the implications of implementing ramp metering and variable mandatory speed limits on the same section of road (McCabe 2006[4]). A summary of the key issues is shown below.

The existing operation of RM and variable mandatory speed limits (or controlled motorway (CM)) could act in a complementary way. When deployed on the same section of motorway flow breakdown is delayed and platoon management can be achieved.

In the period when the flow is reaching capacity, the operation of RM delays flow breakdown allowing the upstream speeds to remain at 50 mph for longer. The benefits of increased throughput due to RM and higher harmonised upstream speeds due to CM are potentially enhanced by operating both control measures on the same section of motorway.

In the period after flow breakdown RM improves the merge behaviour by producing small platoons of traffic, at the same time, CM protects the back of any queue that has formed, the resultant impact on safety of both control measures operating on the same section has the potential to provide enhanced safety benefits.

Support of CM by RM

RM could assist the current operation of CM when the CM system is setting 60/50 mph in congested and heavily congested periods. RM could potentially control the flow to complement the setting of 50 mph and 60mph signals on the main line. This could result in an improved speed harmonisation allowing a higher level of service (higher main line speed/shorter journey time) to be achieved for longer, with only short delays to slip road traffic. This mode of operation would require sufficient acceleration distance from the stop line to the merge to enable the vehicle to reach merge speeds of between 50 and 60mph. In a number of circumstances this may require works to extend the length of the slip road.

Support of RM by CM

CM could assist the operation of RM when the slip road queue is reaching the slip road storage capacity. In theory setting a 30mph signal upstream of the merge area could reduce the mainline flow to a level that would allow more slip road traffic to join without exceeding the downstream capacity. This is currently a modelled theoretical concept (Hegyi 2004[4]) that in theory produces benefits in terms of allowing the downstream flow to run at capacity for longer without the bottleneck causing flow breakdown. When this has been modelled a significant reduction in the number of main line shockwaves has been noted, with resulting accident and journey time reliability benefits.

Overview of combined RM/CM operation

The conclusion of the study (McCabe 2006[4]) indicated that overall the benefits of CM and RM that were demonstrated in the pilots are achievable when the two control systems operate on the same section of motorway. The areas of concern that need to be examined are during flow recovery and when inappropriate speeds are set on the main line.

If the systems are linked in their operation the benefits in terms of speed and flow and accident reduction could be greater than that of the separate control systems.

4. Controlling emissions

Another reason for combining various control systems could be to reduce emissions locally (in the case of Particulates or Oxides of Nitrogen) or more broadly (in the case of Carbon Dioxide).

A recent study was undertaken to determine the potential impact of combining various control measures to reduce NO_x emissions (full results can be found in McCabe 2006[6]) on motorways.

Table 1 describes the scenarios tested. The model was run for a single day and the output was given in NO_x emissions per km based on modelled year of 2010.

Table 2 shows the reductions in emissions shown as the maximum hourly reduction and the total reduction for the day. The modelled results only include the reductions in emissions from the motorway main line. The increased entry slip emissions from ramp metering and traffic spill over into surrounding local road network were not part of the study.

Discussion on modelled results

With reference to Table 2, the operation of the fully integrated system and the access management systems resulted in the change in speed from 26.8 mph to 50 mph gave the largest hourly reduction in NO_x emissions of 17.3%. This change reflects the move from the congested through busy approaching free flow regime of the speed flow curve produces the largest reduction in emissions. The change in speed from 26.8mph to 50 mph should also produce benefits in terms of accident reduction and journey time savings.

In order to produce the change in speed from 26.8 mph to 50 mph an access management system would be needed because the 26.8 mph speeds occur in the middle of the peak period when a ramp metering system would normally have produced queues that fill the storage capacity of the slip road. Normally a ramp metering system would release additional traffic from the slip road when the ramp queue approaches the entry point which results in a drop in mainline speed. An access control system would relocate the queue to a suitable location away from the motorway network allowing the mainline speed to stay at 50mph.

The method normally used for access management would be to integrate the ramp metering queue management with the local road network traffic control system to facilitate queue relocation. A ramp metering system would still produce some improvement at this stage as the effect of managing the platoons of traffic and having a higher level of throughput on the motorway prior to reaching this stage in the peak, means the average speed may still be between 35 and 40 mph. The higher speed with ramp metering produces a maximum of 13.6% hourly reduction in NO_x. The overall daily impact of ramp metering at 2.2% and access management at 4.0% reflects the fact that the systems are only operational for a small proportion (7 hours) of the day, albeit at the busiest time when the highest levels of emissions are produced.

The emissions impact of reducing speed, when the flows go above a particular threshold (in the model 4000 vph and 5000 vph was used), was smaller in a given hour: 5.1% for 50 mph and 3.5% for 60mph. Also the speed reductions operate when the flows are lower than the flows for ramp metering/access management. During the busiest periods the actual speeds were less than the speed from the flow settings due to congestion and flow

No.	Name	Description	Impact on speed model
1	Set 60mph when flow > 5000 vph	This scenario set the speed to 60mph when the flow is greater than 5000 vph	This has the effect of reducing the speed to 60 mph when it is above 60mph with a flow greater than 5000 vph, has no effect when the speed is already lower than 60mph.
2	Set 50mph when flow > 5000 and sets 60mph when flow >4000vph	This scenario set the speed to 60mph when the flow is greater than 4000 vph and 50mph when the flow is greater than 5000vph	This has the effect of reducing the speed to 60mph when the flow is greater than 4000 vph and 50mph when the flow is greater than 5000vph, has no effect when the speed is lower than 60mph or 50mph being set.
3	Ramp metering	This increases the speed for the period when, in the base scenario, the motorway is running in the lower half of the speed flow curve	This increases the speeds by 10% to 15% above the base scenario for 7 hours in the model
4	Access management	This increases the speed from below 50mph to 50mph assuming that the access management system can manage the additional flow away from the motorway network	This increases the speeds from below 50mph to 50mph for 7 hours in the model
5	Fully integrated	This is a combination of scenario 2 (speed reduction) and 4 (access management)	Impact is a combination of 2 and 4
6	Fully integrated exceeding annual mean	This is as scenario 5 plus a 60mph is set when the hourly average NO2 concentration exceeds 40 ug/m3. (which is assumed to be 24h for this scenario)	Same as scenario 5 plus 60mph set for rest of hours when fully integrated not impacting on speed
7	Fully integrated exceeding hourly limit	Same as scenario 6 apart from a 50mph is set rather than a 60mph when the hourly NO2 concentration exceeds 200 ug/m3.. (which is assumed to be 24h for this scenario)	Same as scenario 5 plus 50mph set for rest of hours when fully integrated and not impacting on speed

TABLE 1
SCENARIO DESCRIPTION

Scenario number	1	2	3	4	5	6	7
Modelled reduction	Set 60 at 5000 vph	Set 50 at 5000 & 60 at 4000	Ramp Metering	Access management	Fully integrated	Fully integrated exceedance of hourly limit	Fully integrated exceedance of annual mean
Maximum hourly reduction	3.5%	5.1%	13.6%	17.3%	17.3%	17.3%	17.3%
Overall % reduction in 24 hour period	0.2%	0.8%	2.2%	4.0%	4.7%	5.6%	5.9%

TABLE 2
MODELLED REDUCTIONS IN NOX EMISSIONS RELATIVE TO BASELINE SCENARIO FOR DIFFERENT TYPE OF CONTROL INTERVENTION

breakdown. The overall impact of speed settings based on flow for the 24 hour period was also relatively small at 0.8% for 50mph and 0.2% for 60mph. While there may be a positive impact on accident reduction due to a reduction in speed, there is a negative impact on journey times due to slower speeds.

The impact of reducing speed, based on a threshold set from air quality monitoring produced higher benefits overall with improvements of 1.9% for 50mph and 1.6% for 60mph and hourly savings of a maximum of 8.6% for 50mph and 7.1% for 60mph (albeit during low flows at night). Linking the setting of speed limits to monitored thresholds close to annual mean or hourly standards can potentially bring improvements when they are most needed during times of poor air quality. There are benefits in terms of accident reduction of reducing speeds. There are dis-benefits in terms of journey time increases.

5. Conclusions

Studies into the operation of controlled motorways have shown that, potentially, benefits can be achieved with reduced congestion, reduced accidents and reduced emissions. However, it should be noted that a methodology of using speed and flow thresholds requires substantial fine tuning to achieve efficient results. A system that automatically fine tunes parameter settings potentially gives long term efficiency gains.

A fully integrated system using variable speed limits linked to ramp metering/access management potentially can manage a motorway to address a range of local priorities (congestion, safety or environmental). Reducing speeds to improve safety and produce environmental benefits can potentially increase journey times. However, the safety and environmental benefits can be enhanced by linking to ramp metering/access management to maintain constant speeds and improve journey time reliability.

Overall the principle of using variable speed limits and ramp metering to improve efficiency, safety and environmental impacts has been theoretically proven. To maximise the benefits of implementations the automation of parameter setting should result in more consistent outcomes.

The views expressed in this paper are those of the authors and do not necessarily reflect those of Atkins or any of Atkins' clients.

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Low Cost CCTV

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Abstract

Closed Circuit Television (CCTV) systems are a useful tool for managing the interurban highway network. They allow control room operators to provide appropriate response to incidents and events on the highway. CCTV infrastructure in the interurban environment has been expensive to provide adequate coverage of the whole network. This paper seeks to address this by investigating the possibilities of reducing the cost of implementing individual sites. The paper is based on work undertaken for the UK Highways Agency to improve the business case for CCTV. The output is guidance on how to identify and implement CCTV at lower cost.

1. Introduction

The Highways Agency (HA) is an Executive Agency of the Department for Transport (DfT) and is tasked primarily with the delivery of an efficient network of motorways and trunk roads across England.

In order to help achieve this objective, in recent years the HA has made a significant investment in information technology. This investment has included Variable Message Signs (VMS); Motorway Incident Detection and Automatic Signalling (MIDAS) systems; Closed Circuit Television systems (CCTV) and many others.

There has been an increasing demand for the delivery of CCTV images to operators to provide surveillance coverage of the motorway network and to enable the tailoring of responses to incidents. At present, there is less than 50% CCTV coverage of the motorway network, with some of this being limited to junctions and specific areas of interest. Discussions between interested parties concluded that there is a latent demand for camera images covering the motorway network.

In addition to the motorway situation, it is notable that the HA has very few legacy CCTV cameras installed on its trunk road network. Given that this network extends into some of the HA's harsher operational environments and that it represents a significant part of the HA's responsibility, this is an untenable long term position.

2. The issues

Whilst the HA recognises the value of images from the roadside, it is important to note that CCTV poses significant deployment problems for the Agency. Over many years, as a standards based organisation, the HA has built up a robust set of specifications which govern the deployment of CCTV on the roads. Whilst these standards have ensured that the HA has a high quality network, they have also mandated the use of equipment and techniques that have placed onerous constraints upon system deliverers.

These mandatory requirements of the organisation have driven the costs of deploying a camera to above £150K, per site, for areas that have existing suitable communications and power infrastructure. CCTV provision at sites where no infrastructure exists has typically been prohibitively expensive and is rarely installed.

It is clear, therefore, that if latent demand for CCTV based services exists, it is not being met.

An initiative to provide "Low Cost CCTV" has been born, then, out of a desire to achieve the deployment levels required by the HA, whilst achieving this within the organisation's limited technology budget.

As a first step towards achieving this challenging objective, the HA commissioned Atkins to evaluate the drivers behind the prohibitively high costs mentioned above. In addition the HA has also instructed Atkins to evaluate those costs and to provide advice on a suitable mechanism for reducing them on future deployments.

Proof of concept

The Atkins report identified that significant savings could be leveraged from the existing CCTV deployment costs, whilst maintaining a high level of service to the operators that utilise the images. Potential cost savings for deploying a camera are between £17K and £45K.

One of the constraints that was placed upon Atkins early in their task was the requirement to utilise only off-the-shelf or existing technologies. This was to ensure that pricing information contained within the report was viable against the need for a large scale rollout of Low Cost CCTV within the short to medium term.

Early implementation

Against the background of the report, Atkins has been commissioned to survey some likely locations for Low Cost CCTV and to manage the rollout of a significant number of sites within the cost constraints. Atkins has further been tasked to document this learning process in order to enable Low Cost CCTV to be rolled out across the network over the next 3 years.

In order to drive the early implementation of Low Cost CCTV sites before the end of the financial year, the HA has appointed Carillion as the contractor for the deployment of the first 100 sites.

3. Typical CCTV

CCTV infrastructure

The integral components of CCTV typically in use on the HA's network comprise of structural, communications, power, camera and enclosure infrastructure. The structural infrastructure consists of a 15 metre wind-down camera mast secured by concrete foundations. Communications infrastructure includes a local connection into the HA national communications network. Power is provided through an individual low voltage connection. The camera has pan, tilt and zoom control features and there are camera equipment housings and separate equipment, communications and power housings with surrounding hard-standing areas.

Design and deployment standards

Standards exist for the deployment and the design of sites for equipment and infrastructure to be installed on the highway. In the context of CCTV, appropriate standards exist for deployment, equipment functionality and the mounting of equipment onto structures. A design variation from these standards requires the appropriate technical approval in conjunction with mitigation for the variations.

The deployment criteria for CCTV are mainly concerned with traffic volume and specific geometric characteristics or predictable adverse weather conditions. Set traffic volume levels vehicles are per lane, per day, per carriageway. Geometric characteristics involve locations where average running speed is less than desirable. The all-purpose trunk roads are excluded, except near motorway junctions where the deployment criteria are met. There is limited flexibility in deployment even where sites would benefit.

Mounting of equipment onto a structure may be subject to Approval in Principle (AIP), and depending on the complexity of the problem, structural or design calculations would need to be undertaken and assessed by a competent structural engineer. This in conjunction with advice from the HA technical standards department would determine the viability of use.

4. Proof of concept

Determination of the viability of Low Cost implementations depends on the substitution of reduced cost system components. Analysis of previous schemes showed that of the five components, the 'Communications' and Structural' components accounted for the majority of cost differences. These are shown in Table 1 below.

TABLE 1
SUMMARY OF COMMUNICATIONS AND STRUCTURES COSTS

The difference in costs is not only attributed to the supply of necessary equipment, but the installation environment requiring additional traffic management and health & safety costs.

Clearly, the funding requirements to meet the latent demand vary significantly, depending on the components' availability. Solely identifying more cost effective installation practices and ensuring efficient procurement will not reduce cost significantly. A combination of elements was proposed, including utilisation of the existing structural assets on the highway network, trialling "fit-for-purpose" technology and challenging existing design and installation practices/standards

Innovatory schemes exist for both structural and installation practices, although these are somewhat sporadic in use. Combined structure usage has led to reduced costs; particularly the usage of CCTV enabled structures. There is a range of existing structures, including gantries, overbridges, lightweight superspan gantries, and post mounted textual and graphical VMS. Structural design standards determine the design and protect the integrity of a structure. Modifications by installation of equipment can require certified approval.

Approval is for an individual structure and is not flexible for generic use. The latest VMS have brackets to support camera housing included within the design and cable to provide both power and data.

Clearly, installing equipment without increasing the number of structures has aesthetic and environmental benefits. These have not been quantified within this scheme, however.

A comparison of typical system components with identified alternatives is shown in Table 2 below.

Substitution of standard CCTV site implementation with the technologies listed above is estimated to reduce the single site average cost to around £35,000.

TABLE 2
COMPARISON OF LOW COST CCTV OPTIONS

INTELLIGENT TRANSPORT SYSTEMS

5. Early implementation

The objectives for the Early Implementation are to prove the viability of a low cost solution applicable to the highway network. This is to include documenting revised design and implementation processes and to determine flexible and generic solutions for standards and actual costs. In addition, knowledge sharing information is to be provided.

To validate the actual costs, the implementation has to be of a sufficient number to encounter a variety of different site conditions. This was set at sixty sites.

During these works it has been important to ensure that selected sites are representative of the locations where the latent demand for CCTV images exists. In order to achieve this, sites have been selected from locations where a need for CCTV images has been locally identified.

Suitable structures within these areas were identified by querying of the Structures Management Information Systems (an asset register that contains construction and maintenance data for network structures). Each of these identified structures was then surveyed to capture physical characteristics, field of vision and suitability for low cost components. Their CCTV deployment suitability was recorded against a set of appropriate parameters including, ease of installation and maintenance, environmental impacts, cable access and power availability issues. Through a weighted scoring process, suitability of sites was ranked to provide a deployment suitability priority order. From this, the characteristics of suitable sites for lower cost installation have been identified.

The sites chosen are geographically spread across England requiring multiple regional agreements for installation access. To minimise construction delays a single contractor (Carillion) was chosen from the HA's select contractor list and through the usage of Early Contractor Involvement (ECI) was involved in the design build-ability process.

At the time of writing, the installation programme is scheduled to commence in October 06, with all sixty sites installed by April 07.

IUpon completion of the Early Implementation, guidelines for CCTV deployment and its implementation for the CCTV, a business case will be produced. Issues include the reduced costs for CCTV installation using alternatives to the civil engineering, structural and communication components of typical CCTV implementations.

6. The future of Low Cost CCTV and the HA

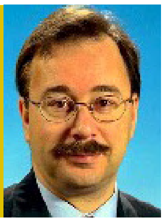
The work that the HA, Atkins and Carillion have undertaken to date demonstrates that by questioning the paradigm within which the HA operates, significant cost and time savings can be achieved. It is important to note that this project has only just begun to explore the range of possible technologies that could result in cost savings to the HA and as such it is expected that future works will be undertaken.

Some of these works will focus on the deployment of lower cost technologies and larger numbers of cameras. It is expected that depending on the desirable locations as much as 50% of the latent demand for cameras can be achieved using the techniques covered within the Early Implementation work.

Currently, about 50% of the latent demand for cameras cannot be serviced through the utilisation of the techniques described above. It is therefore the HA's intention to continue works in future years to investigate the use of new technologies in order to drive down costs further.

The work described in this paper is sponsored by the Highways Agency. The views expressed in this paper are those of the authors and do not necessarily reflect those of the sponsor.

Results from the Demonstration of Interoperable Road User End-to-End Charging and Telematics Systems (DIRECTS)



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Abstract

In 2001, the Fareway Alliance, comprising Kellogg Brown & Root (KBR), Atkins and Thales Telecommunication Services, was selected by the UK Department for Transport (DfT) to set up and operate DIRECTS, a fixed price research project to demonstrate the operation of Road User Charging (RUC) systems for open multilane trunk roads and urban roads. The project is aimed at preparing technical specifications and proving equipment interoperability, including Dedicated Short Range Communication (DSRC) and Mobile Positioning System (MPS) charging equipment, exception handling, enforcement and full Back Office functionality. Based on a business model proposed by the DfT following successful on-track testing completed at the TRL in Crowthorne, UK, the project moved to the demonstration area in the city of Leeds, UK, where on-road trials were completed in March 2006. This paper provides an insight into the practical experiences and results gained in integrating, testing and operating an end-to-end RUC system.

1. Introduction

The first paper[1] on this challenging project summarised the project objectives, the parties involved, the developing technical specifications and the aims of the demonstration in Leeds. In summary the relevant project objectives were to:-

- Develop, over the period of the contract, a suite of specifications known as the Open Preliminary Minimum Interoperability Specification Suite (OPMISS)
- Design, develop, test, install, operate (for a minimum of 12 months and 600,000 DSRC transactions from volunteer vehicles and 115,000 images of non-volunteer vehicles) and decommission a complete end-to-end demonstration system for RUC at a number of urban and motorway locations in the City of Leeds. Volunteers were to help test the system by going about their normal business. No volunteers were to be charged, and the project was not to affect other motorists using these roads
- Prove, via the demonstration, multi-lane free flow charging and enforcement capability, and the interoperability of systems and equipment provided by multiple suppliers
- Show, via the demonstration, the viability of the OPMISS and the DfT Business Model.

Fareway selected seven sites (comprising fourteen Charge Points) from the set of fifteen made available by the DfT. Over 500 vehicles from local volunteer organisations took part in the year long demonstration, and the RUC system gathered transaction data in accordance with the DfT's requirements.

Originally the volunteer strategy was based around recruiting participants from the general public with incentives for continued compliant participation in the trial. The DfT later restricted volunteers to commercial organisations only, however, with no incentives. All data captured underwent detailed analysis by Fareway, and the billing and customer care facilities supporting the volunteers provided valuable feedback for the operational procedures.

This paper provides an insight into the design and practical experiences gained in integrating and testing the end-to-end RUC system, and the performance results obtained. This paper presents further details on

- Overall architecture and business model
- Practical front-end system issues when operating with real users under actual traffic conditions, including interoperability, effective charging, capture of adequate and sufficient image data to handle exceptions and effectiveness of Automatic Number Plate Recognition (ANPR)
- End-to-end performance, Back Office billing, charge reconciliation between sub-system entities, and methods for minimizing front-end imperfections.

2. Aspects of design and operation

In this section, a short summary of the system architecture and business model is presented and then a number of design and operational issues, relating specifically to RUC, follow. These show the areas where interoperability is important and where technical choices may have to be made when procuring RUC system entities.

Architecture and business model

An open architecture was adopted, to demonstrate the ability of sub-system suppliers to deliver and integrate both front-end and back office entities, designed and built to comply with trial areas of OPMISS. This meant that any operator (a Local Authority or tolled crossing operator etc) wishing to introduce an electronic RUC scheme in the UK can use OPMISS for a competitive tendering process, knowing that, provided suppliers can comply, they will interoperate with all other sub-system entities that use these specifications as a basis for their systems.

The exact nature and form of these specifications is subject to further development and consideration within DfT [2]. The architecture proposed by the DfT is presented in [2] and will not be reiterated here.

The major OPMISS entities are:

- The user establishes a relationship with a Payment Services Provider (PSP) for the provision of services related to charged road use. This may also include the issue and fitting of the vehicle On-Board Unit (OBU) itself
- Users, as they drive on charged roads, will have their related Vehicle Passages logged by the On-Road Services Providers (ORSPs). These entities gather charges relating to road usage for each individual charging scheme (local authority, toll road, bridge, tunnel etc). ORSPs may capture data from the roadside via different technologies, for example DSRC or MPS based vehicle equipment. Any exception handling requiring manual checking of images is performed at the ORSP
- ORSPs will also capture evidential data (e.g. images) from the roadside of suspected non-payers either for the generation of Charge Records based upon images or for supporting the enforcement process through the issue of Penalty Charge Notices (PCNs)
- The Data Clearing Operator (DCO) provides both a 'bridge' and a 'block' between ORSPs and PSPs; a 'bridge' to route Charge Records from an ORSP to the correct PSP and a 'block' to provide users with privacy concerning their movements. (ORSPs do not need to know who is using their roads. Conversely, PSPs do not need to know where their clients have been unless a user requires an itemised bill)
- Each ORSP and PSP will establish a business relationship with the DCO
- The ORSP will establish Vehicle Registration Numbers (VRNs) from images of suspected non-payers. This information will be used by the DCO as the basis for determining whether the user's vehicle has an OBU and therefore the relevant PSP, or that there is no known OBU linked to the vehicle as the basis for initiating either an image based Charge Record or a PCN.

Each of the above sub-systems establishes a business model for effective charging for the individual services provided and for the agreement, reconciliation and passing on of charges to the next sub-system in the end-to-end RUC process.

In order to demonstrate every entity within the architecture and business models, as shown in Figure 1, an end-to-end system was installed at Leeds. This was designed to meet and trial elements of the open interface specifications defined within the OPMISS, and to demonstrate interoperability of equipments from seven different manufacturers.

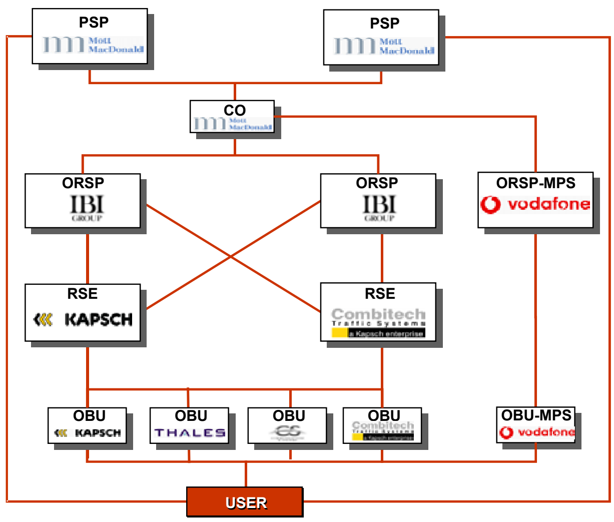

FIGURE 1
DIRECTS SYSTEM ARCHITECTURE

Practical front end systems

There are a number of ways in which charges can be levied for road use. These range from a fixed charge, through charges based on vehicle class, time of day, distance travelled or the number of times per day entering a cordon, area or road section, to a full demand control using one or more of the above. In addition, exception handling (a more general term for checking of potential violators of which enforcement is a subset) may be introduced at fixed or mobile positions within or on the perimeter of the scheme.

Results from the Demonstration of Interoperable Road User End-to-End Charging and Telematics Systems (DIRECTS)

Both DSRC and MPS based technologies were included in the DIRECTS project. These were employed to demonstrate cordon, area and distance (segment) based charging schemes. Each method has strengths and weaknesses, and the project has been used to investigate how the end-to-end system may be designed to minimise any weaknesses and thereby optimise performance. The following sub-sections present some of the challenges which have been observed and what measures were taken to address them.

DSRC performance

The DSRC technology is well developed and based on well proven International standards as described in[1]. The DIRECTS project exploited six different DSRC based OBU designs, (four off-board [central] account and two on-board account designs, based on the A1 and A1+ air transactions respectively[3,4,5]), and two Road Side Equipment (RSE) designs to test the level of interoperability achievable. Testing on TRL's test track, under controlled conditions, demonstrated that DSRC transaction success rates exceeding 99.9% are achievable. Even when attempting to stress the transaction process with, for example, unusual vehicle manoeuvres[6] results in excess of 99% are achievable. Similar results were obtained throughout the on-road trial in Leeds using a fleet of test vehicles.

Despite a number of attempts at the detailed technical level to maintain this level of accuracy over the entire volunteer fleet, however, DIRECTS only achieved an average performance of approximately 88.5% for raw DSRC transactions. (No DSRC accuracies were specified, they were driven by the end-to-end targets discussed below.)

This drop in performance was mainly attributable to two practical issues. The first relates to non-incentivised volunteers, who were neither rewarded for taking part nor penalised for "inadvertent" lapses in maintaining their vehicles' equipment status during the year long trial in which, overall, some 800,000 DSRC transactions were collected. The second reflects the change in make up of the test track fleet, which was predominantly private light goods vehicle based to that of the commercial volunteer fleet in Leeds. Additionally, there was an overall level of 0.6% incomplete transactions that were detected at the roadside but standing alone, contained insufficient DSRC data to charge the user.

This figure divides into incomplete transaction rates of 0.25% and 3.6% for off-board and on-board transactions respectively. The data is summarised in Table 1.

Although further analysis is ongoing, no interoperability issues between different manufacturer's OBUs and RSEs have been identified that would substantially alter these results. The Fareway end-to-end design mitigated this drop in performance within the DIRECTS trial system as discussed overleaf.

Image capture performance

Exception Handling within RUC systems is usually associated with the capture of an evidential image(s) which allows details such as VRN or vehicle class and trailer status to be assessed manually or automatically. This may be used for a number of reasons, including creation of an image based

Vehicle Class	**Supplier A Gantries**			**Supplier B Gantries**		
	Passages	Transactions	Performance	Passages	Transactions	Performance
PLG	155859	137462	88.20%	153986	141092	91.63%
PSV	151710	131743	86.84%	142682	125989	88.30%
VHGV	25970	22968	88.44%	25753	22911	88.96%
HGV	6099	5252	86.11%	5916	4930	83.33%
TOTALS	**339638**	**297425**	**87.57%**	**328337**	**294922**	**89.82%**
OBU Manufacturer						
Off-Board OBU #1	84315	71658	84.99%	81660	73378	89.86%
Off-Board OBU #2	101601	90676	89.25%	97939	90191	92.18%
Off-Board OBU #3	59020	51121	86.62%	56439	50206	88.96%
Off-Board OBU #4	42514	38017	89.42%	41235	37769	91.59%
On-Board OBU #1	11079	8196	73.98%	10753	7386	68.69%
On-Board OBU #2	41109	37757	91.85%	40411	35992	89.06%
TOTALS	**339638**	**297425**	**87.57%**	**328337**	**294922**	**89.82%**

TABLE 1
RAW DSRC INTEROPERABILITY PERFORMANCE BY MANUFACTURER AND VEHICLE CLASS

account for unequipped vehicles (i.e. those with no OBU), creating PCNs for unequipped or wrongly classified vehicles or recreating vehicle details when the DSRC transaction has failed.

In some cases, there is no option but to suffer an undercharge e.g. where an image is the only means of charging and the VRM cannot be identified from the image. In other cases, where for example, the trailer status cannot be determined, it may be more prudent to apply an undercharge to a user by assuming there is no trailer, rather than overcharge in error and risk a challenge that is indefensible.

This decision would be driven by an ORSP's business rules and over and undercharging targets. For RUC schemes where vehicle class and trailer status are used to vary tariffs, the camera system must be able to provide operators with images showing sufficient field of view and clarity at all times of day and night to determine such parameters unambiguously. Lane cameras (ideal for determining VRNs) and overview cameras (for vehicle class and contextual information) are therefore necessary and may have to be used in tandem. From the safety aspect, infra-red illumination and monochrome cameras are the best choice. The use of colour cameras to UK Home Office recommendations[7] is not essential for an RUC scheme, but can assist further in vehicle identification. The DIRECTS trial system provided Fareway with an opportunity to compare the performance of two designs of image capture system under a range of traffic density, weather and lighting conditions, to ensure the OPMISS contains best practice.

The average image capture accuracy was 83.9% and 91.7%[2] for the test-track and Leeds on-road testing respectively. Of the images captured, the usability rates were 98.3% and 84.6% respectively. Hence from a given traffic flow the number of usable images that would be captured were 83.2% and 77.6% respectively. The specified targets were 85% and 80% respectively. It should be noted that the average image capture accuracy between charge points for the Leeds on-road trial varied between 84.5% and 99.6%.

In addition, one of the suppliers significantly improved image capture performance between the test track and on-road tests and that it was always expected that image usability would decline in real open road traffic as opposed to that of the test track.

Classification system performance

Although there were no requirements for external, independent (measured) vehicle classification systems, they were installed as part of the DIRECTS system. The DIRECTS system utilised OBU declared class data and measured vehicle classification system based upon overall vehicle dimensions. Current European specifications are based upon axle, weight and environmental metrics. Given the limited space available here, these results have been omitted.

Manual operator checking

The process of manual checking of image material is costly, subject to human error and any methods for increasing efficiency are therefore beneficial. Sometimes the images are collected because the vehicle is unequipped and therefore the VRM is of prime importance. This would be used to identify the vehicle keeper so that charges due may be levied. For an equipped vehicle, an exception may be due to a class or trailer mismatch requiring validation of these parameters and the VRM is not necessarily required. The specified process for image analysis dictated that both the vehicle classification (including trailer) and VRM be entered. Over the period of the trial, 67000 images of volunteer vehicles were analysed with resulting average vehicle classification and VRM accuracies of 97.8% and 99.7% respectively. Additionally a one hour test was carried out where a target performance was set at analysing 150 images. Resultant speeds ranged from 96 to 190 images per hour, depending on the operator, with a non-corresponding accuracy range 90% to 98%. (A speed increase in manual checking could be achieved by indicating to the operator which parameters require checking and which are not necessary.)

Automatic Number Plate Recognition (ANPR) performance

ANPR systems potentially allow a percentage of image based exceptions to be processed without manual checking and both increase throughput and reduce processing costs. Although there were no requirements for ANPR,, it was installed as part of the DIRECTS system. Its performance was therefore not a primary concern of the trial. All DIRECTS RUC lane cameras were equipped with ANPR capability.

Some performance analysis of these ANPR systems was carried out, but it was found that the relationship between the confidence level and the probability of correct VRM interpretation was not monotonically rising to allow for its inclusion in any automatic process for reducing the operator workload. It was given to the operator as a prompt, however, for correction or confirmation. The maximum accuracy achieved by the ANPR systems barely matched the average accuracy of manual checkers as stated above. This reflects the fact there were no performance requirements for ANPR systems within the DIRECTS trial.

In addition to general levels of ANPR accuracy, there still remains the question of interoperability between ANPR systems from different suppliers. Unless the confidence level profiles and corresponding accuracy of VRM estimates are harmonised, ORSP image processing rules cannot be generic and ANPR supplier independent.

3. Summary and conclusions

End-to-end performance

The accuracy requirements of the DIRECTS contract were defined by means of end-to-end performance criteria for over and under charging of equipped vehicles, i.e. the volunteer fleet. These were set at six overcharges and thirty-six undercharges, at 95% confidence level, in 600,000 transactions. These targets were independently financially incentivised. Given the limitations of the Front End equipment Fareway took a commercial decision, given the fixed price nature of the DIRECTS contract, to concentrate on the overcharging target and not to actively pursue the undercharging target.

A number of processes have been developed to improve the Back Office functionality and implement a practical scheme. Each entity would expect to be paid for the value added by its functionality. The concept of charges passing between entities (ORSPs to the PSP via the DCO) allows each process to add a charge for their services. All charges from all entities result in a combined charge for road use which is levied at the roadside and appears on the user's monthly itemised or accumulated bill. If additional processing is performed by the ORSP or PSP in determining the owner of an unequipped vehicle or a classification or trailer violation, an additional charge is made. These additional charges, incurred for a violation, are agreed between entities at the time of establishing their business relationship and their combined result is published as a surcharge on the standard road use charge or as a penalty charge.

The design of end-to-end DIRECTS system attempted to mitigate the imperfections of the Front End systems through the ORSPs transaction processing rules for charging transactions These allowed charging where there was sufficient information to unambiguously charge as opposed to invalidating all incomplete transactions. It also recovered situations where the DSRC transaction proved inconclusive, by allowing vehicle passages to be charged based upon the VRN derived from the exception images captured. Consequently the raw DSRC accuracy of 88.5% quoted above was recovered to an end-to-end accuracy of 97.5% using these methods.

Additional Back Office algorithms were developed by Fareway to check for duplicated charges (e.g. a duplicated DSRC charge, a DSRC charge and a VRM based charge, a DSRC charge and an MPS charge) for the same vehicle passage to overcome or minimise overcharging and to provide early warning of any fraudulent use in the system. These algorithms resulted in the DIRECTS system producing an overcharging rate of approximately 0.0011% of equipped vehicle passages.

The DIRECTS project has demonstrated a practical open road RUC scheme using interoperable equipment and volunteer vehicles within a real motorway and urban environment. The knowledge and experience gained from this exercise has been captured in a suite of open and interoperable specifications which will allow any local authority or Back Office agent to procure, install and operate any RUC entity within an interoperable national framework, subject to their subsequent development and refinement within the DfT. This could then avoid potential non-interoperable deployment of RUC infrastructure in the UK, and the need for road users to fit multiple OBEs, or manage multiple contracts (in the longer term) before being able to travel country-wide

Additionally DIRECTS has demonstrated that:

- An end-to-end design approach should be adopted to counter Front End imperfections;
- Transaction accuracies for real users do not necessarily match those obtained using test fleets, even using the same set of "real" charge points
- Without volunteer incentives, the results from a road user trial cannot replicate the realistic situation where users are penalised for non-compliance with operational rules.

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Managing air quality by 'Data Mining'

Exploratory statistical analysis to determine causal factors affecting pollutant concentrations close to a motorway

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Abstract

Local air quality has taken on a higher importance in Europe with the introduction of mandatory limits from 2010. However, local air pollution problems can be subject to site-specific circumstances not always picked up by traditional modelling, especially in complex areas around large sources (e.g. motorways). The key to designing a successful ITS emissions reduction scheme is a detailed understanding of the site-specific problem to be addressed. Using work for the Highways Agency, this paper illustrates how a toolbox of statistical methods can be used to analyse local air quality problems.

1. Introduction and background

In the UK there is an obligation on local authorities to assess air quality in their area using a combination monitoring and modelling. In the UK this process has resulted in 192 local authorities declaring Air Quality Management Areas (AQMAs). 95% of these are mainly the consequence of road transport emissions, with 50% associated with Highways Agency roads. The majority of the AQMAs are for the annual NO_2 concentrations, with some expected to exceed the 2010 EU limit value for annual mean NO_2 if no action is taken to reduce emissions.

National and regional authorities routinely collect vast quantities of data (traffic speeds, flows, classifications, meteorological data and environmental data) relevant to addressing these issues.

However, in the past, the main difficulties have been timely collation of the data from different sources, enough storage/computing to analyse the data, and clarity on the relationships and interconnections between various data types before undertaking formal assessments. At the site-specific level the relationship between road use and measured air pollution is not a direct one, especially in physically and operationally complex areas.

This study demonstrates the power of "Data Mining" techniques to identify the causes and extent of a local air quality problem in the UK. This mainly relates to exceedances of an annual mean, however the techniques could be used as a decision support tool for traffic strategies (in particular atmospheric conditions) - in the absence of having detailed area wide traffic and environmental models available. In the case of this study, data mining was undertaken at an early stage to develop mitigation as the scheme was being developed – before the more usual modelling began.

2. Data Mining techniques – what and why?

This paper describes work undertaken for the Highways Agency to investigate the causes of a particular air quality problem and hence focus the evaluation of measures to reduce emissions. M Bell et al[1] identified the generic impact of implementing ITS measures (speed control and flow control) - this paper includes the influence and effect of site specific aspects when assessing the impact on emissions and air quality of a particular ITS measure.

The work uses the toolbox of statistical analysis techniques known as “Data Mining” to combine meteorology and activity data in time and space to extract key causal trends explaining air quality. Data mining can be used to answer questions such as:

- Is there an air quality problem – where and why?
- What are the dominant sources of the problem at each location of interest?
- Under what conditions do the sources contribute at that location?
- What are the current conditions that maximise and minimise air quality (the causal factors)?
- How can these conditions be influenced?
- What are the uncertainties?

Data Mining requires considerable data. Details of the data cannot be published here, but include: continuous air quality monitoring data; non-road emissions data; air quality background data; meteorological data; lane by lane detailed traffic data; previous air quality modelling; site details – terrain, plans, structures, land use, source and receptor dimensions.

The intention of the work was to systematically test the strength of causality of explanatory variables on resultant concentrations of Oxides of Nitrogen (NO_X), discount those disproved at this site as important, and to use the remainder to construct ‘causal profiles’ to explain the root of any found air quality problem, such that mitigation would be highly focused and effective

The paper focuses on Data Mining for defining the problem and root causes. It does not currently include mitigation tests for the study area in question as:

- The work was undertaken at an early stage of the project when several data were early unpublished drafts. For example using the morning peak traffic model only, as the evening peak and inter peak traffic models were not yet available - the final traffic model results could be very different.
- The mitigation tests themselves do not explicitly rely on Data Mining, but use standard emission modelling techniques – instead Data Mining has focused the actions tested and provided better quality data than normally available without detailed models of traffic and emissions.

A multitude of data mining techniques was used on the project in practice, including for example smoothing trends and controlled variable tests to identify any highly localised temporary effects influencing annual average summaries, pairs plots, correlations for filtered data (such as for specific met conditions), and relative source contributions by site type and location. This paper illustrates the advantages of the approach using just two:

- Generalised Additive Models (GAMs) – predictive statistical models of the causal factors in pollution at a site, accounting for non-linear relationships between variables
- Bivariate polar plots – combined meteorology and pollution roses used to define directional & conditional sources, and their proportional importance.

3. Bivariate polar plots

A useful exploratory technique for identifying candidate pollution sources is based on the idea of a pollution rose. A conventional pollution rose usually plots the wind direction dependence of the concentration of a pollutant in polar coordinates. The use of the polar coordinate system makes it straightforward to gain a rapid idea of the direction in which different pollution sources exist. The approach, which was originally developed for source apportionment analysis at Heathrow Airport (Carslaw et al,[2]) is briefly explained below. The usefulness of pollution roses can be greatly enhanced by including wind speed as a third variable, thus making a surface. Together, wind speed and direction yield important information about the influence of different source types. For example, as the wind speed increases concentrations due to a typical road source will also tend to decrease due to the increased dilution of the plume. By contrast, an elevated buoyant source (such as a chimney stack) has a different behaviour. As the wind speeds increase the plume can be brought down to ground level more rapidly and lead to higher concentrations than under lower wind speed conditions. These types of behaviour can be exploited in bivariate polar plots to highlight the potential presence of different source types. Furthermore, where sites measure more than one pollutant, the plots permit further source differentiation.

The technique has been further enhanced since its initial use at an airport setting. Rather than simply plotting a surface, the surface is ‘modelled’ using a two dimensional smooth function called a thin-plate regression spline (Wood[3]). The advantage of fitting a smooth function to the surface is that much of the noise present in the raw data can be removed to help identify the unique signal due to the source itself. The residuals from such a fit can also be tested using conventional methods to ensure that these are essentially random noise. Figure 1 shows a bivariate smooth for nitrogen oxides (NO_X) measured close to a major motorway site. The plot shows the clear presence of a source to the east (i.e. the motorway) and also shows that concentrations tend to decline with increasing wind speed - behaviour indicative of a ground-level source. The accompanying plot for carbon monoxide (CO) shares some of the characteristics of the NO_X plot - a clear indication of a source to the west i.e. in the direction of the motorway. However, the CO plot also indicates another source to the north east that does not seem to be a major source of NO_X.

This may well represent an elevated source, which the investigator can then try to identify from available inventories.

One of the useful aspects of bivariate polar plots is that they help in the selection of meteorological conditions that maximise the influence of a particular source. In the case of the motorway site shown in Figure 1, if one was interested in only processing data where the motorway had a strong influence on concentrations (and reduce the influence of other sources), wind speeds greater than 1 m s-1 and a wind direction range from ~ 200-300 degrees are clearly shown as the relevant focus in the plots.

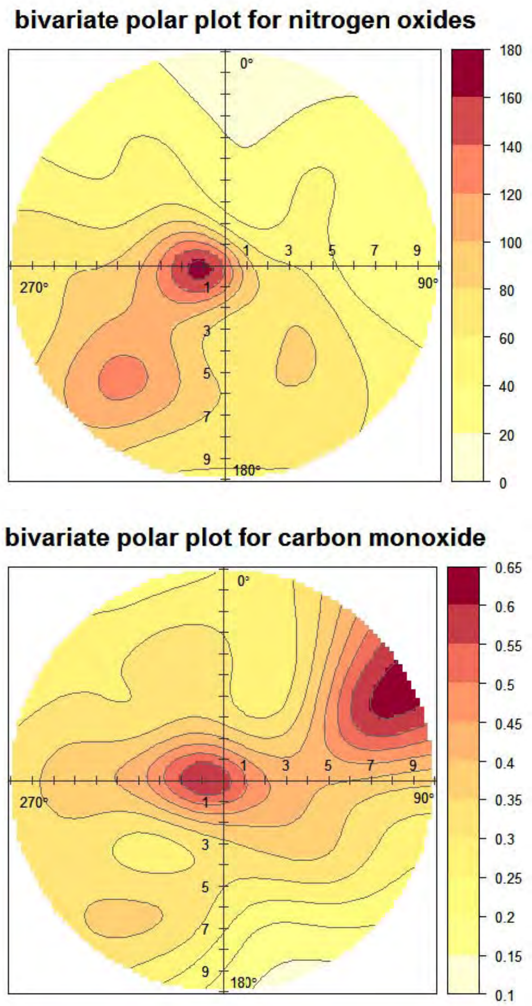

FIGURE 1
BIVARIATE POLAR PLOT SHOWING THE DISTRIBUTION OF NOX (PPB) AND CO (PPM) CONCENTRATIONS BY WIND SPEED AND WIND DIRECTION AT A MOTORWAY SITE.

4. Statistical modelling using detailed traffic and meteorological data

This section considers how a modern statistical technique based on Generalised Additive Models (GAMs) together with highly detailed traffic information can be used to model and predict air pollutant concentrations close to a motorway.

Linear regression is often used to describe the dependence of a variable on explanatory variables. However, for many applications, the linear model is too restrictive because the relationships between explanatory variables are too complex to be represented as linear functions. In these situations GAMs offer many advantages for the modeller by relaxing some of these restrictions. For the current study, where there are many non-linear relationships between variables, the GAM approach is particularly attractive.

The additive model in the context of a concentration time series is described as (Hastie and Tibshirani[4]):

$$C_i = \sum_{j=1}^{n} s_j(x_{ij}) + \varepsilon_i$$

where Ci is the ith concentration of the time series, sj(xj) is a smooth function of covariate p, n is the total number of covariates, ε_i is the ith residual and var(ε) = σ2, which is assumed to be normally distributed. A key part in developing a GAM is how to represent the smooth functions and how to control the degree of smoothness used. We adopt the GAM approach of Wood and Augustin[5], which integrates model selection and automatic smoothing parameter selection using 'penalised regression splines', which while optimising the fit, penalise roughness. A more detailed discussion of a recent application and development of GAMs for air pollution can be found in Carslaw et al [6]

For illustration the following outlines construction of a GAM to address the importance of vehicle type on end concentrations. Considerable traffic data were available in the form of MIDAS traffic data flow and speed data for 2005 on a 1-minute basis for each six lanes of the motorway.

The data were processed to yield values comparable to meteorology and air quality monitoring for the following variables: hourly mean flow of vehicles < 5.2 m long ('cat1'); hourly mean flow of vehicles 5.2 – 6.6 m long ('cat2'); hourly mean flow of vehicles 6.6 – 11.6 m long ('cat3'); hourly mean flow of vehicles > 11.6 m long ('cat4'); and mean vehicle speed (km/h).

Detailed vehicle emission estimates were then made using current UK emission factors used in the National Atmospheric Emissions Inventory. GAMs were constructed to attempt to explain measured hourly NO_x concentrations using a series of explanatory variables. The models took the form:

$$Log(NO_x) = s(u, v) + s(back.\ NO_x) + s(temp) + s(h/Lmo) + s(NO_x\ emission) + s(speed)$$

The r^2 for this model was 0.64, suggesting that most of the hourly variation in NO_x at these sites can be explained by the models. In the equation above, u and v are wind components (e.g. u = [wind speed].sin[π.[wind direction]/180)), back.NO_x is that background NO_x concentration, *temp* is the ambient temperature, *h/Lmo* is a stability parameter derived from the meteorological pre-processor from ADMS Urban, NO_x *emission* is the total hourly NO_x emission from vehicles using the motorway, and speed is the speed of vehicles in km/h. The logarithm of NO_x was required in order to provide residuals which met important diagnostic tests e.g. constant variance. Variables were also chosen that were not strongly correlated with one another as this could affect the inferences made from the model.

For all these relationships, the variation with NO_x was in keeping with prior knowledge of the likely physical relationships between variables, suggesting a well-formulated model. Various models were considered, but for all of them only flows of long vehicles (> 11.6 m) were the most statistically significant in explaining measured concentrations. These results suggest that flows of these long vehicles dominate measured concentrations of NO_x at site, despite only accounting for 11% of the total flow over this period.

This finding cross-checked with the findings of other techniques, and was used to focus vehicle-specific mitigation measures specific to the site, rather than the more common 'all traffic' orientated measures.

One interesting relationship revealed by the GAM modelling was that as vehicle speed increases there is a tendency for concentrations of NO_x to reduce. This behaviour is at odds with known speed-dependent emission factors where increasing vehicle speed is associated with increasing emission rates (at least over the range of speeds considered here). This behaviour is shown in Figure 2.

One possible explanation for this effect is due to the effects of vehicle induced turbulence (VIT). As vehicle speed increases then so does the turbulence generated in the wakes of the moving vehicles. VIT is most important for larger vehicle types where there is a large frontal surface area. Modelling VIT in a deterministic way is difficult and this is currently the source of uncertainty over modelling near-field concentrations close to motorways. Another interesting implication is related to traffic management.

For example, in the UK there has been some interest in enforcing speed restrictions on motorways as a means of reducing emissions. However, as vehicle speeds reduce then so will VIT and there will be two competing processes controlling the concentrations of pollutants close to motorways - dependent on the relative strength of these competing processes. Following on from this project, the Highways Agency is investigating aspects of improving VIT in available model approaches. Thus such data mining can assist in justifying further air quality model developments by focusing effort on truly causal issues.

The GAM models developed to explain concentrations at the motorway site were also subsequently used to test for trends in concentrations by removing the influence of meteorology. This was achieved by using the models to predict pollutant concentrations under constant meteorological conditions, thus removing the seasonal and inter-annual variation that can mask pollutant trends.

GAM models were also constructed in this study to investigate the importance of Primary NO2 at the specific sites as a causal factor. These were compared to the results of a chemical model of the estimated trend in primary NO2 from vehicle emissions on the motorway. Both models showed evidence of an increase in the ratio during 2005, although there was insufficient time series after the step change to statistically confirm this.

The same GAM approach used to define causal factors in explaining pollution concentrations can then also be used to predict future year concentrations. In the main project, GAMs were used to estimate future annual mean NO2 concentrations at key receptor sites in 2010 using detailed traffic data and speed-dependent emission factors. It was estimated that based on base year meteorology, and using only those variables with statistically significant causal links at each site, the annual mean NO2 concentration at these sites would be well below the Limit Value, without any further mitigation. These modelling results provide an alternative method of estimating future concentrations compared with conventional dispersion modelling, and in this case provided clear evidence for the Highways Agency on whether additional mitigation were needed or not in the intervening period before the major project was open for use.

A technique that we have begun to investigate is stochastic gradient boosting (or boosted regression trees, BRTs) - a technique that is gaining in prominence in the data mining community (Friedman[7]). BRTs have been shown to have excellent predictive capabilities and several characteristics that would be beneficial for analyses similar to those discussed in the current paper. There are several advantages of BRTs for modelling. They are able to handle a mix of variables e.g. continuous and categorical and missing data can be handled efficiently, which is a more general advantage of tree-based methods. Variables do not need to be transformed for model fitting purposes i.e. one can work in the response scale. Trees can model non-linear effects and interactions between variables. As noted by De'ath[8], non-linearities and interactions are often the norm for many problems and these are automatically included according to the size of trees used. While GAMs can handle variable interactions, it can be difficult to know which interactions are the most important and which should be included in the model. BRTs can also handle a very large number of explanatory variables, which is likely to be a useful characteristic for applications of ITS. Some limited use of BRT was included in the main project, but the authors have developed application of these concepts since then.

5. Conclusions and recommendations

Novel statistical techniques have been developed to consider the causes of measured pollutant concentrations at the near-motorway sites and to understand the factors controlling these concentrations. The study was able to identify the key sources of pollution in the area and gauge the importance of the various sources. This meant that any migration measures would focus on the key sources rather than all sources equally. Data mining was used at an early stage of the project, before traditional modelling was available – this gives decision makers statistical data to justify the development of mitigation measures at an earlier stage in the project programme.

If such methods are developed further, they have the potential to increase the understanding of the influences traffic on major roads has on local concentrations of pollutants. Such techniques can (and have) also been used to understand and constructively challenge the findings of traditional emission and dispersion modelling, especially to improve the understanding of any uncertainty in such outputs and its effect on decision making.

The methodology (rather than the findings) is transferable could be used to identify the relative effects of traffic and industrial sources of pollution in any other area with sufficient measurement data.

The technique facilitates the targeting of particular vehicle types on particular days for the developing of strategies to mitigate against regular pollution episodes. There is the potential to use these techniques as a decision support tool linked to multi-source data collection system to inform decision making for regular pollution episodes by analysing the impacts of previous strategies.

Acknowledgement

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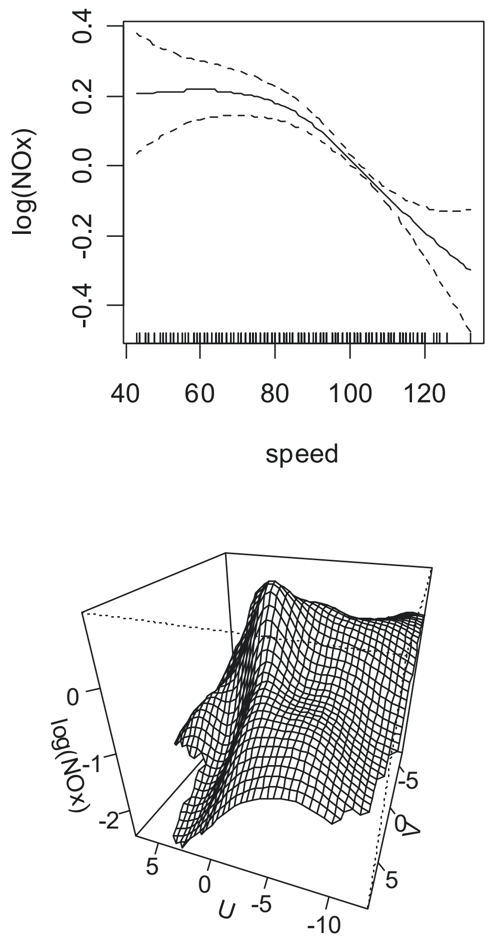

FIGURE 2
GAM RELATIONSHIPS BETWEEN MEASURED NOX CONCENTRATIONS AT THE SITE - WITH VEHICLE SPEED AND WIND COMPONENTS U AND V.

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Using asset management to deliver well-maintained highways

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Abstract

Bill Moss and Alan Taggart report on the progress being made in the use of asset management to deliver well maintained highways at both a national and local government level in the UK.

1. Background

The highway network is one of the largest and most visible publicly owned assets in the UK. It is used daily by the majority of the travelling public for commuting, business, social and leisure activities. As such it is fundamental to the economic, social and environmental wellbeing of local communities and the economic prosperity of the nation as a whole.

At a national level our economic prosperity relies on reliable movement of goods and people around the highway network. These national roads are part of the strategic road network. In England the Highways Agency is responsible for the management of the strategic road network which comprises motorways and trunk roads. The devolved administrations of Scotland, Wales, Northern Ireland and London have separate authorities responsible for strategic roads.

At a local level the highway network helps to shape the character and quality of the local areas that it serves and makes an important contribution to wider local authority priorities, including regeneration, social inclusion, community safety, education and health. Responsibility for the management of the local network rests with local highway authorities. In England alone there are over 100 local highway authorities. With Scotland, Wales, Northern Ireland and London included there are many more.

Increasing traffic growth has brought increasingly widespread recognition of the importance of highway maintenance at both a strategic and local level and the high value placed on it by users. However, this importance has not generally been reflected in levels of investment, and public concern has been increasing about the implications of this for safety and journey reliability. Recent increases in investment have been welcome but a sustained long-term programme of investment that delivers value for money is required. It is widely recognised by central government and highway authorities alike that effective asset management planning is essential to meeting these needs.

2. Guidance on Highway Asset Management

The benefits of asset management have been recognised by local authorities and in 2004 the County Surveyors Society published the Framework for Highway Asset Management. This provided basic guidance for local authorities on how to adopt and implement asset management. Further impetus was provided by the Department for Transport (DfT) who in their guidance on the second round of Local Transport Plans strongly recommended that all highway authorities in England, outside London, should develop asset management plans. In 2005 The Roads Liaison Group, representing all highway authorities in the UK, commissioned Atkins to produce the Codes of Practice for Highways and Structures which provide further practical guidance for the asset management these assets. The Codes are available for free download at www.roadscodes.org

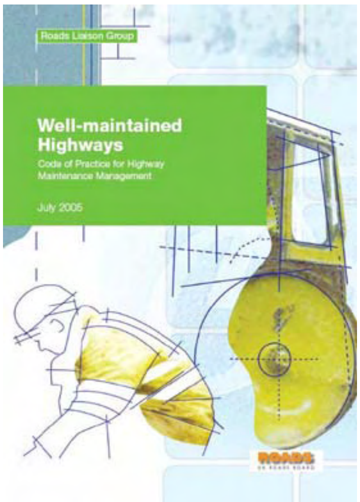


FIGURE 1:
CODE OF PRACTICE

The Code of Practice for Highways recommends that all local UK highway authorities should set out their longer term strategy for maintenance of the network in a Highway Asset Management Plan (HAMP). The objective of the HAMP is to substantiate investment in highway maintenance by demonstrating value for money in delivering the authority's social and economic aims over the life of the asset.

The HAMP should include appropriate policies and procedures for asset valuation, including an annual valuation report. This recommendation, although not mandatory builds on the guidance given by the DfT on delivering value for money through asset management planning.

The Highway Asset Management Plan, complemented by asset management plans for other asset groups such as structures, lighting, and other transport related assets owned by the authority provide the overall Transport Asset Management Plan (TAMP).

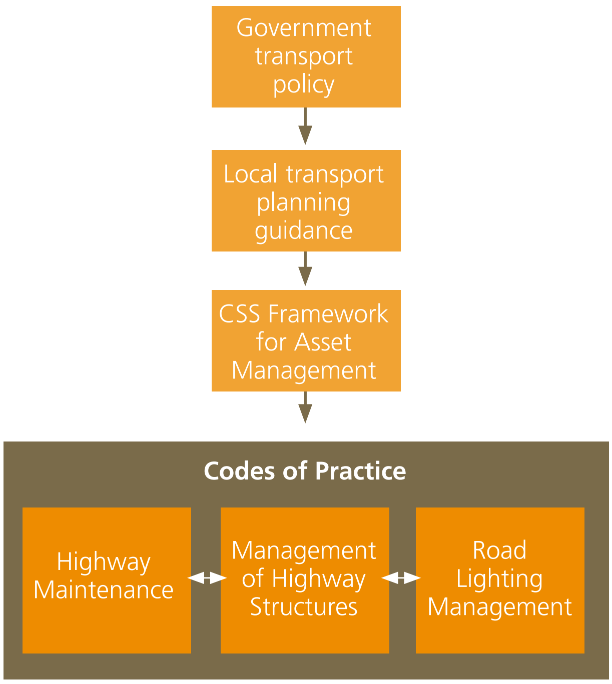
Progress is also being made on the development of asset management for strategic road networks in the UK. Atkins has supported this work for both the Highways Agency and Transport Scotland.

In England, the Highways Agency's strategic objectives are to deliver safe roads, reliable journeys and informed travellers. A well maintained highway asset makes a vital contribution in meeting these objectives. The Agency is recognised internationally as providing best practice in many key areas of asset management. However the need to continue to develop their approach is well understood.

The Agency's approach is set out in their asset management strategy which identifies targets for improvement and sets out how progress will be monitored against these targets.

In Scotland, the strategic road authority, Transport Scotland is embarking on their Asset Management Improvement Programme. This is an ambitious three year project that will deliver improvement to their asset management approach, processes and systems. The programme was developed as a result of a gap analysis benchmarking Transport Scotland against best practice highway authorities worldwide.

In the other devolved administrations of Wales, Northern Ireland and London the strategic road authorities have also been developing similar approaches to asset management.



The output from developing a lifecycle model is a tool that can assist authorities in answering:

- What funding is required for pavement maintenance and rehabilitation in the next 3 to 10 years?
- What condition distribution would be expected if the funding above were available?
- What are the consequences if this sum were not available?
- What are the benefits if the maintenance expenditure were increased?
- What are the optimum times for maintenance actions?
- What are the risks of delaying the necessary actions?

By answering these questions local authorities will be able to provide answers to substantiate future investment which will assist in their application for funding.

3. Life cycle management of road pavements

Lifecycle planning forms a key element of Highway Asset Management Plan.

Life cycle planning for pavements like other assets provides a method for objectively comparing alternative means of achieving the required service level; where the alternatives differ not only in their capital expenditure but also in their subsequent operational expenditure.

Through this approach alternative engineering solutions are assessed in order to provide value for money by determining whether a higher initial construction or rehabilitation cost is justified by a reduction in future maintenance costs. Delivering value for money is one of the key requirements of the second round of Local Transport Plans. For a pavement asset reduction in future maintenance costs are critically important in order to reduce congestion on the road network. The Traffic Management Act now requires such an approach.

A number of modelling methods have been adopted by Atkins to model pavements and these can be divided into two general categories:

- Probabilistic Models, ideal for long term (10+ years) planning which predict the distribution of the performance variables and provide future maintenance needs in terms of work volumes
- Deterministic Models, ideal for short term (0 to 10 years) planning which predict a single value for each performance variable and provide the location as well as volume of future maintenance.

To produce realistic results, both modelling methods require significant amounts of raw data. Atkins has adjusted the models to utilise the best available information at the time of evaluation, but at the same time recognising the risks associated with the uncertainty in data. Every highway authority must determine if they are collecting the correct data, in the right form and if they are utilising the appropriate system to manage the data.

Obtaining reliable data is a problem due to the high costs associated with large data collection programmes and the management systems. Recognising this, the Code of Practice for Highway Maintenance recommends that risk based approach to data collection should be adopted by authorities.

4. Development of Highway Asset Management Plans

For all highway authorities the key component of the Highway Asset Management Plan is the approach adopted for the management of the road pavement (running surface). This is what road users travel on every day and develop perceptions about how well roads are maintained or otherwise. The road pavement is also the most expensive part of the highway asset both in terms of capital replacement costs and costs to the economy resulting from delays to road users. Therefore in developing an approach for highway asset management road pavement often leads the way. As mentioned previously most local authorities are adopting asset management to enable them to substantiate investment in roads and demonstrate value for money.

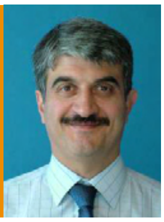
5. Looking to the future

Atkins has been commissioned by the DfT to report on the progress highway authorities are making in the development of their TAMPS and how asset management has been embedded in their respective organisations. The outcome from this work will provide a comprehensive picture on how asset management is being progressed by the majority of local authorities in the UK and will also identify areas for improvement.

Asset management can be used to achieve continuous improvement in providing a highway service that delivers value for money and thus provides a means for justifying investment in highways against the competing demands on local authorities.

Cathodic protection for reinforced concrete structures

A proven technique to stop corrosion



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Abstract

Corrosion is a worldwide problem and costs billions of pounds. Corrosion problem is not something new but the awareness of this problem associated with civil engineering structures, particularly the reinforced/pre-stressed concrete highway bridges, multi-story car-parks, buildings etc. is relatively new. Corrosion is insidious in nature. The corrosion of steel in concrete is only apparent when it is quite advanced and 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 large amounts of waste due to corrosion. Cathodic Protection is the only proven technique to stop corrosion of steel in chloride contaminated concrete.

1. Introduction

The problems of concrete deterioration due to corrosion of steel reinforcement and/or pre-stressed/post tensional systems in concrete structures are world wide, costing the nations billions of pounds equivalent to around 4 to 6% of GDP (Gross Domestic Product). A recent cost of corrosion study [1] 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 [2]. A study by Kei Wei in 1999 [3] put the annual cost of corrosion in China at more than $50 billion and with the fast growth in Chinese economy this figure must be escalating. There are no simplistic model(s) 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 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 and cathodic protection. The paper then outlines the methodology of evaluating concrete structures, in a real world, affected by corrosion utilising appropriate NDT techniques. This is a pre-requisite to identify and quantify the type and extent of distresses. This paper also 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.

2. 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 (e.g. $Fe = Fe^{++} + 2e^-$), and cathodic or reduction reactions (e.g. $O_2 + 2H_2O + 4e^- = 4OH^-$).

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:

- Loss of alkalinity in the concrete
- Penetration of aggressive ions to reinforcement depth
- A combination of both factors

The main offending ion for the break down 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 [4] it oxidizes 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 consume hydroxyl ions, as seen in the following reactions:

$2Fe + 6Cl^- = 2FeCl_3^- + 4e^-$

Followed by

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

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 [5].

Alternative reactions for complex formation are:

$Fe^{2+} + 6Cl^- = FeCl_6 + 4e^-$ or
$Fe^{3+} + 6Cl^- = FeCl_6 + 3e^-$

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

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

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.

Electrochemical principles of corrosion can be summarised diagrammatically with the aid of Pourbaix diagrams (pH-potential, based on thermodynamics) and Evan's diagrams (Polarisation diagrams, based on kinetics of the corrosion processes). An example of a common, although perhaps less well-known form of a potential-pH diagram for the iron-water system is given in Figure 1, which is based on various electrochemical equilibriums [6].

However, pH-potential diagrams do not indicate the magnitude or speed at which a corrosion reaction may proceed. The rate of corrosion is controlled by the kinetic factors and this can be graphically represented on a potential vs. current plot, commonly known as 'Evans' or 'Polarisation diagram'. An example of which is given in Figure 2.

From Figure 2, it can be seen that for any particular value of Ecorr (the measured electro-potential reading) the rate of corrosion could vary by several orders of magnitude due to the logarithmic relationship between corrosion potential and corrosion current.

Due to the inhomogeneous nature of concrete, steel reinforcement may develop electro-potential values over the full ranges for both active and passive states within the same structure. The range of potential values, both 'natural' and 'impressed' potentials of steel in concrete, is depicted in Figure 3 [7].

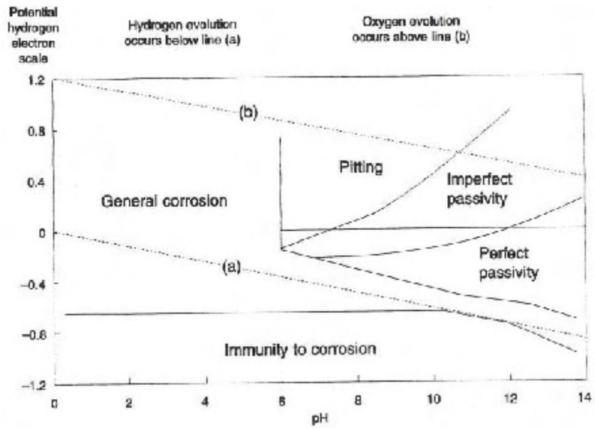

FIGURE 1
POTENTIAL-PH DIAGRAM SHOWING EXPERIMENTAL CONDITIONS OF IRON CORROSION IN SOLUTION CONTAINING CHLORIDE

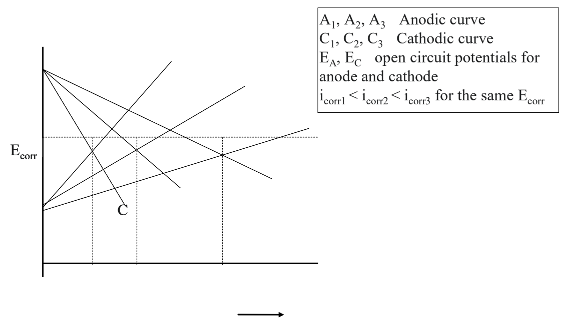

FIGURE 2
ELECTRO-POTENTIAL VERSES CORROSION CURRENT.

ASSET MANAGEMENT

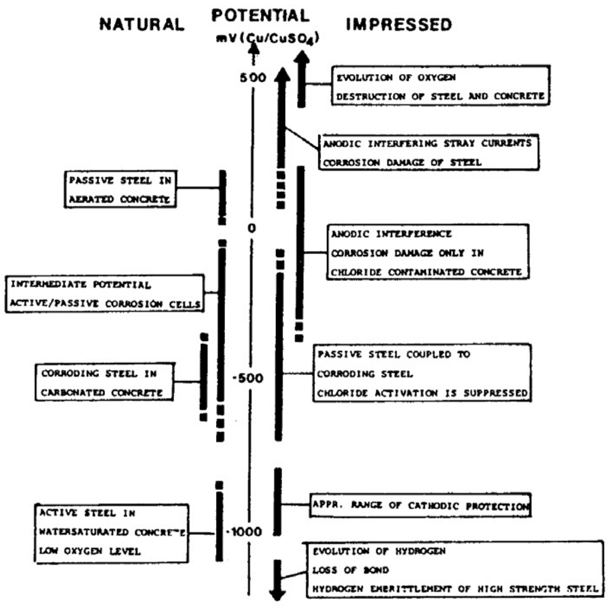

FIGURE 3
NATURAL AND IMPRESSED POTENTIALS OF STEEL IN CONCRETE.

The corrosion potential (Ecorr) of steel in concrete depends on many interacting factors such as:

a. Relative humidity of the pore system (i.e. moisture content)

b. Cement / aggregate ratio of the mix

c. Availability of oxygen, which in turn depends upon other factors including the permeability of concrete and relative humidity.

d. Distribution of active / passive areas

e. Presence and type of depassivating ions, e.g. chloride ions or CO3- ions

f. Environmental influences, such as seasonal variations of wetness / dryness, temperature, etc.

g. Concrete cover to steel reinforcement

Among the factors listed above, the availability of oxygen (which depends on the diffusion coefficient of oxygen through concrete cover) and the moisture content in concrete (or relative humidity inside concrete) have the most significant and measurable effects on measured potential.

Both entities influence the cathodic reactions of the corrosion process which becomes cathodically controlled, i.e. the electro-potential tends to be numerically more negative with increasing moisture content (i.e. relative humidity). In addition, electro-potential values tend to be numerically more negative with the depletion of oxygen. This oxygen depletion is caused by decreasing diffusion coefficient as a result of increasing water saturation of the concrete while the corrosion rate remains significantly low. Tuutti [8], reported that if the RH changes from 65% to 78% the oxygen diffusion coefficient decreases about four fold and at values of 90-95% RH the cathodic process, which consumes O_2, reaches a limiting situation. On the other hand, Gjorv et al [9] reported that the concrete resistivity changes by several orders of magnitude as the RH changes from 100% to 50% with a significant effect on corrosion rates. The overall effect of oxygen depletion and increasing water saturation on electro-potential values is shown in the Figure 4 [10].

Further detailed commentary on the corrosion mechanisms is outside the scope of this paper.

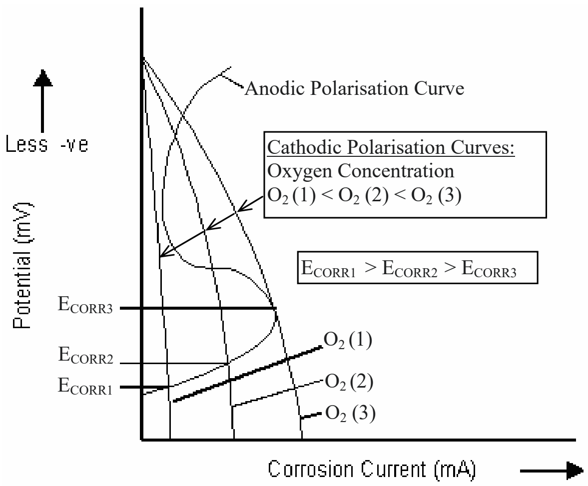

FIGURE 4
EFFECT OF OXYGEN CONCENTRATION ON ELECTRO-POTENTIALS.

3. Inspection and assessments

In the UK, the inspection of highway structures for assessment on a regular basis is a mandatory requirement by the authorities responsible for their upkeep and serviceability. The extent and the frequency of undertaking this assessment depend on the age of the structure and the condition existing at the previous inspection period. The current practice in the UK is mainly governed by the procedures laid down in the Department for Transport's Standards and advice notes, issued from time to time, together with various Codes of Practice for testing issued by the Standard Institutions. Broadly, the inspection of bridges and highway structures is carried out at various levels as a rolling programme.

The general practice in the UK is that all bridges and highway structures are inspected biennially (this is generically called General inspection) and all major bridges are subjected to a Principal inspection every six years. During the principal inspection phase it is required that the structure is inspected at close distance to identify all defects and incorporates NDT testing to establish the underlying cause(s). The main objectives for the inspection and NDT testing are:

1. To establish the existing state of deterioration, together with their cause(s)
2. To define the future rate of deterioration and their consequences from the data collated following an inspection and testing of the distressed structure
3. To quantify the extent and severity of defects for the formulation of the proper remedial solution(s) as required and to provide mitigation against the deterioration/failure due to incorrect diagnosis of defects and wrong specification of repair materials
4. To provide data for the management to institute a planned maintenance programme with due consideration to a whole-life costs analysis of various repair options.

In addition, Special Inspections are conducted on as required basis and are necessarily structure specific.

The common features of the investigation/testing regimes, generally comprise the following:

a. Visual Inspection/delamination survey
b. Half cell potential survey
c. Cover meter survey
d. Dust sampling for chloride content determination
e. Depth of carbonation
f. Concrete resistivity
g. Electrical continuities

And for pre-stressed/post tensioned structures additional tests to include:

- Post-tensioning Tendon Grout samples for chemical and petrographic analysis
- Tendon Inspection by intrusive methods
- Endoscope Inspection to examine the condition of the strands and the extent of voids by vacuum test.

Detailed visual inspection and delamination survey are of foremost importance to identify and quantify all visual structural defects and signs of distress within the structure. Structural distress can manifest itself in many forms – some quite apparent but others not so obvious. There are, however, always some "tell-tale" signs of distress and it will be the identification of these signs that will form the basis of the survey. A typical list of signs that would be appropriate for the survey of the structure and the likely cause(s) of the structural distress and their importance in relation to the application of cathodic protection are given in Table 1. The information gathered during the visual surveys will be recorded on a standard pro forma that will be specially developed for each project. Extensive photographing of the structure will also be carried out, supplemented by video recordings where these will be advantageous for future assessment works.

The visual surveys of the structures in totality will be vital to ensuring that all signs of distress are identified and recorded and that associated in situ testing and sampling works are refined to be of maximum value in the development of the structure repair and Cathodic Protection Design works.

Various in situ tests and the relevant testing procedures togtether with the purpose of the tests are summarised in Table 2.

The results of the inspection and in situ testing obtained from the Principal Inspection and/or Special Inspection is meticulously recorded and backed up by photographs.

The in situ and laboratory test results are then scrutinised and interpreted to draw definitive conclusions and recommendations based on the prognosis of the problems for the management's consideration. These procedures are equally applied whether the structure is just reinforced concrete structure, post-tensioned, or a combination of both. The structural assessment, (inspection and NDT testing as an integral part), is therefore crucial prior to developing technically correct and cost effective remedial measures for strengthening, repair and rehabilitation of ageing and/or damaged reinforced concrete structures.

It is important for the assessment (inspection and NDT testing) of highway and building structures that a very clear objectives are defined at the outset and plan the programme carefully, particularly with regard to the selection of the most appropriate NDT techniques to be used and then execute the programme methodically.

Distress Feature	Probable Cause(s)	Importance to Cathodic Protection Design Process
Cracking	Overload, Corrosion, Shrinkage	High Importance
Pattern Cracking	Differential volume change between internal and external concrete	Low Importance
Exudation	Alkali Aggregate Reaction	Low Importance
Incrustation	Leaching of alkaline material from concrete	Low Importance
Rust Staining	Corrosion of reinforcement, tying wire or other embedded metal items	High Importance
Dampness	Leakage, Poor Maintenance	High Importance
Pop-out	Development of local internal pressure	Important
Spalling	Corrosion of reinforcement, tying wire or other embedded metal items	High Importance
Delamination	Corrosion of reinforcement, tying wire or other embedded metal items	High Importance
Weathering	Environmental action causing loss of laitance and paste	Low Importance
Honeycombing	Lack of vibration	Important

TABLE 1
TYPICAL SIGNS AND CAUSES OF DISTRESS IN RC STRUCTURES

In situ Test	Frequency	Remarks
Delamination Survey	All accessible areas	To identify hidden defects leading to spalling of concrete cover to corroding steel reinforcement.
Half-cell Potential Measurement	To suit arrangement of the structure	The half cell potential of the reinforcement would be determined in accordance with ASTM C876-99 [11], using a nominal 500mm grid over an area of 2 square metres.
Cover-meter Survey	To suit arrangement of the structure	The concrete cover to reinforcement would be determined in accordance with BS 1881: Part 204: 1988 [12]. Three sets of minimum/maximum cover readings would be taken within the test areas.
Concrete Resistivity	Three measurements per test area	The concrete resistivity of various elements of the structure would be measured at selected locations, using the modified 'Wenner' method.
Dust Sampling	Two locations per test area	Concrete dust samples would be collected from selected locations for chloride content analysis. These sample holes would be drilled in 25mm increments to a depth of up to 105 mm (i.e. 4 No. sub-samples per sample location). The first 0-5mm of the sample from the first increment would be discarded in order to avoid surface contamination.
Depth of Carbonation	Two locations per test area	The depth of carbonation would be determined by spraying phenolphthalein solution onto the freshly fractured concrete surface in accordance with BRE Information paper IP 6/81[13].
Continuity Testing	One measurement within each test area	The electrical continuity between steel reinforcement at various concrete areas of the structure would be systematically tested.
Core Samples and Concrete Breakouts	To suit arrangement of the structure	A number of 100mm cores would be extracted from the structure to determine the strength of the concrete. All drill holes, core holes, breakouts and spalled areas would be rehabilitated using well acknowledged materials and procedures.

TABLE 2
COMMON IN SITU INSPECTION AND TESTING PROCEDURES FOR RC STRUCTURES

The results of the inspection and testing carried out during the Principal Inspection or Special Inspection of structures play their part in assisting responsible authorities to formulate technically correct options for repair and rehabilitation of the deteriorating structures. This is to institute a planned maintenance programme with due consideration to a whole-life costs analysis of various repair options.

4. Options for repair/rehabilitation

As stated in the introduction, highway and building structures worldwide are deteriorating at ever-increasing rate. The option for repair of the 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 for concrete deterioration is the 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. The strategies for repair/rehabilitation are discussed below:

4.1 Repairs of visible damages

Visibly damaged concrete areas (cracks, spalls, exposed and corroding reinforcement etc.) could be easily rehabilitated using traditional repair techniques. However this is a short-term solution only to cover up the unsightly condition of the structure without much consideration for identifying the cause of the problem.

4.2 Development of repair/rehabilitation strategy

The main objective here is the development of a repair/ rehabilitation strategy including a detailed repair scheme and preparation of manual for further inspection frequency and monitoring of the repairs. At this stage two main options are to be considered.

4.2.1 Conventional patch repair

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 be met easily. Furthermore, one of the potential problems with the 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.

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 in order 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 the assessment of strength, and are attended by the bridge engineer. These are not further discussed here.

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.

4.2.2 Cathodic protection

For longer-term solution of 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 the chloride-contaminated concrete. Alternative options are the replacement of a part of a structure, extensive concrete removal or a continuous programme of patch repairs throughout the life of the structure. Life-time cost analysis would suggest that the application of cathodic protection to stop incipient corrosion and concrete deterioration would be more cost effective than the alternatives. It is important to recognise that the application of catholic protection is not limited to highway structures and more recently this technique has successfully been used to protect buildings with significant residual life (greater than 10 years).

Anode Type	Long Term Current Density per m2 of anode	Long Term Current Density per m2 concrete	Typical Anode Life Estimate	Suitable for Wet Structures	Suitable for Running Surfaces	Dimensional & Weight Impact/ Installation	Performance Queries	Typical Anode Cost
Conductive Organic Coatings	20mA/m2	20mA/m2 Max	10-15 years	No	No	No Painted	Some unproven products	£20-40/m2
Sprayed zinc	20mA/m2	20mA/m2 max	10-15 years	Possibly	No	No Thermal Spray	Consumption rate Health & Safety	£60-100/m2
Mixed metal oxide coated (MMO) titanium mesh and grid in cementitious overlay	110-220mA/m2	15-110 mA/m2 varying grades	25-120 years	Yes	Yes	Yes In circa 25mm overlay	Overlay Quality Control	£60-£100/m2 including overlay
Discrete Pt/Ti or MMO Ti anodes, with carbonaceous surround	800mA/m2 from carbonaceous surround	Circa 10-110 mA/m2 subject to distribution	10-20 years	Yes, not tidal	Yes	No Placed in pre-drilled holes		£40-£100/m2
Discrete anodes in cementitious surround. MMO Ti or Conductive Ceramic	800mA/m2	Circa 10-110 mA/m2 subject to distribution	20-50 years	Yes	Yes	No Placed into holes or slots		£40-£100/m2
Cementitious overlay incorporating nickel plated carbon fibre strands	20mA/m2	20mA/m2 Max	10-20 years	Probably	No	Yes Sprayed, circa 8mm thick	Limited experience	£30-£60/m2

TABLE 3
IMPRESSED CURRENT ANODE TYPES AND CHARACTERISTICS [14]

Cathodic protection is an electrochemical technique to stop/mitigate corrosion by supplying 'current' from an external source in order to suppress the 'internally generated' current flow due to corrosion processes. The 'external' current source could be obtained simply by coupling the steel to another electrochemically more active metal, e.g. zinc; alternatively the 'external' current may be derived from a mains operated low voltage DC power source, viz. transformer/rectifier unit. These two different approaches to supply 'external' current to stop corrosion are generically termed as:

a. 'Sacrificial Anode' Cathodic Protection (SACP) system and

b. 'Impressed Current' Cathodic Protection (ICCP) system respectively.

Both approaches have proved to be feasible, but the impressed current CP system offers greater flexibility with regard to its ability to provide the necessary current in situations where concrete resistivity is relatively high and variable. The sacrificial anode system is most effective if the concrete resistivity is very low or the anode is placed in a very low resistivity environment such as soil with low resistivity, as the inherent driving voltage is low e.g. the potential difference between zinc and corroding steel in concrete is limited to approximately 0.7 volts.

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 development in anode materials and anode system design with real possibility of 'pick n mix' cathodic protection system(s) for above ground RC structures. Anode systems currently available are given in the Table 3.

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 onto the titanium coating.

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.

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 again to provide indefinite cathodic protection at a very low cost.

d. On the steel surface the coverage is approximately 4 - 5 square metres at 30 - 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 the '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. The electrical connections are achieved by attaching the wire ties, integral to the anode system tightly to steel reinforcement; and then the areas are instated using appropriate repair mortar.

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

4.2.2.1 Advantage 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 that the steel has sufficient reserves of strength then 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.

4.2.2.2 Conceptual CP design.

Designing an appropriate CP system is very much structure specific, although the design is to be in compliance with the recommendations of the latest edition of relevant Codes and Standards issued by the National Association of Corrosion Engineers (NACE); British/European Standard (BS/EN) institutes [15-16]. The main aspects of CP design considerations, utilising the 'impressed current' approach, are briefly described below.

i) Anode/Anode Zoning:
In designing the Cathodic Protection Systems, different elements or areas of the structure will need to be grouped into different zones for which different anodes may be provided. These zones may be determined by reference to environmental conditions, exposure conditions, extent of deterioration and so on. The zoning divisions for each element of the structure would need to be determined at an early stage of the design process so that the design of the Cathodic Protection System for the structure could be progressed in detail. Having selected the zoning appropriate for each element of the structure, the Cathodic Protection electrical current level for each zone would need to be determined taking account of various criteria unique to that zone.

ii) Transformer Rectifier:
Cathodic Protection Systems generally operate with a current density to the steel surface in the range 5-20mA/m2. The direct current is normally provided by an a.c. powered transformer-rectifier or equivalent power unit, taking single phase a.c., transforming it to lower voltage and rectifying it to d.c. and outputting it at typically 1-5 ampere, 2-24 volts to each independently operated anode zone. Thus the electrical consumption of an Impressed Current Cathodic Protection System is very modest. An anode zone is an electrically isolated area of anode. Each zone can receive different levels of current. Systems are divided into zones to account for different levels of reinforcement, different environments or different elements of the structure.

Cathodic prevention of new or not yet corroding reinforcement in concrete requires a current density to the steel in the range 0.5-2mA/m2. The lower current density results in good current distribution and safe application to prestressed structures.

Transformer-rectifiers may be manually controlled or controlled either locally or remotely via modem to a computer.

iii) Control and Monitoring System:
The control and monitoring system is usually closely integrated with the power supply. Probes are embedded in the concrete to verify that all of the zones are working and that sufficient current is passing to protect the reinforcement. Monitoring of steel/concrete potential is with respect to embedded reference electrodes. These are usually silver/silver chloride/potassium chloride or manganese/manganese dioxide/potassium hydroxide. There are placed in the concrete in the most anodic (actively corroding) areas prior to energising the system.

Additional reference electrodes are placed in the concrete at locations of particular structural sensitivity, at locations of high reinforcement complexity or at locations of sensitivity to excessive protection.

Steel/concrete/reference electrode (so called "half cell") potentials are recorded and the decay in potential from 'instant off' to a time of 4 to 24 hours or longer is used to determine the effectiveness of the system. A 100mV change in 24 hours or 150mV over a longer period is specified in BS EN 12696 [15], along with certain other criteria.

It is usual to embed several reference electrodes in each zone of the Cathodic Protection System. The cathodic protection current can be interrupted and steel/concrete/reference electrode potentials and potential decays measured.

Computerised monitoring systems allow the collection of decay data to be automated. Remote monitoring systems allow this to be done without a site visit.

4.2.3 Other special techniques

Other electrochemical techniques commercially available as repair systems for corrosion damaged concrete bridges, buildings and other structures are:

- Desalination process for the extraction of chloride from chloride-contaminated concrete; and
- Re-alkalisation process for reintroducing alkaline environment in concrete where carbonation is considered to be the main cause for concrete deterioration and damage.

However, the effectiveness of these two techniques is questionable when their application costs are considered with regards to improving long-term durability of the repaired structures.

5. 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. This paper highlights the importance of inspection and testing (using various NDT techniques for the identification and quantification of defects, together with the investigation into the cause and the consequence of the defects) as an integral part of a well organised programme for Highway Maintenance Plan. Without this information the management decision with regards 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 the cathodic protection is the most appropriate and proven technique to stop corrosion of steel in RC structures.

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

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Updating progress on Highway Asset Management

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Abstract

Asset Management is widely recognised as the best approach for identifying appropriate funding that can deliver value for money in highway maintenance. Lila Tachtsi provides this update on introduction of Asset Management.

1. Introduction

All local authorities are coming under increasing pressure to improve efficiency in the use of available resources for delivering services to the public. A well managed highway network is a core component of these services and vital for a successful local and national economy and society. This vital asset is often the most valuable that local authorities maintain. The management and preservation of the highway network is therefore a key priority.

2. Appropriate funding

The introduction of Whole of Government Accounts and the requirement for local authorities to deliver value for money has created the need for a more robust approach to justifying maintenance funding. Social and political pressures will always influence funding decisions, but all public services deserve their fair share of available funding. It is therefore the responsibility of highway managers to identify and justify appropriate levels of funding for the highway network using robust and defensible techniques that stand up to public and political scrutiny. To achieve this, highway managers must demonstrate the role of the physical highway network in delivering the authority's goals, objectives and agreed levels of service. Also, to ensure value for money, they must clearly present the planned construction and maintenance work required to deliver these targets.

FIGURE 1

3. A rational approach

Demonstrating the value and role of the highway network, and actually delivering planned construction and maintenance work, require effective and efficient management processes and systems that are transparent, defensible, auditable, practical and user-friendly. The onus is therefore on local authorities to develop and implement rational management approaches that are effective, customer focused and deliver value for money. Asset management is widely recognised as the rational approach to achieve these aspirations.

4. Definition and guidance

Asset management is a formal discipline with underlying principles and recognised guidance. The CSS (County Surveyors' Society) has defined Asset Management as:

'A strategic approach that identifies the optimal allocation of resources for the management, operation, preservation and enhancement of the highway infrastructure to meet the needs of current and future customers.'

An asset management approach therefore enables local authorities to link strategic goals and objectives (including customer requirements) to day-to-day operational activities by introducing tactical planning and decision making techniques. This link is illustrated by Figure 1.

Implementing asset management does not require a local authority to change all of its current highway management and engineering activities. The majority of existing activities are retained, but asset management introduces important new techniques that link activities together, improve management effectiveness, deliver better value for money and provide a customer focused service.

Guidance on what asset management means to local authorities and how it should be implemented is given in the following documents:

- Full Guidance on Local Transport Plans, London: Department for Transport, December 2004
- Framework for Highway Asset Management, CSS, April 2004
- International Infrastructure Manual, UK Edition, 2003
- PAS 55-1: Asset Management: Specification for the optimized management of physical infrastructure assets, IAM, 2005
- Well-maintained Highways, Department for Transport, July 2005
- Management of Highway Structures: A Code of Practice, Department for Transport, September 2005

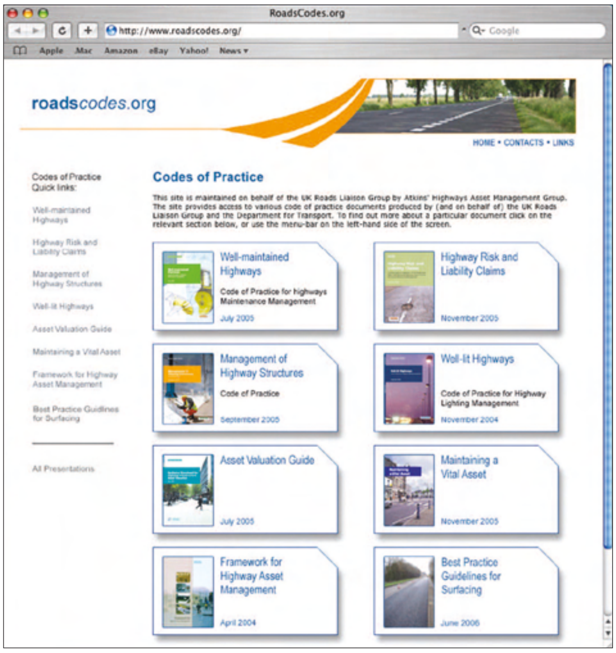


FIGURE 2
FURTHER INFORMATION ON HIGHWAY ASSET MANAGEMENT AND LINKS TO THE GUIDANCE DOCUMENTS CAN BE FOUND AT: WWW.ROADSCODES.ORG

5. Conclusion

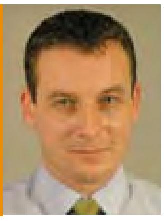
In England, local authorities have been using the available guidance and the services of consultants, particularly in the last two years, to develop and implement Transport Asset Management Plans. The activities this involves and the benefits it is considered to provide are well documented in the above guidance documents, including:

- Providing detailed information on assets, enabling better definition of longer-term corporate need;
- Establishing and communicating a clear relationship between the programme of work/expenditure set out in the TAMP and the LTP targets and objectives;
- Enabling value for money of local highway maintenance to be considered more effectively against other local transport spending.

Transport Scotland has carried out a gap analysis of its approach to asset management and has commissioned a three year project to improve its asset management practices. Scottish local authorities are currently considering the implementation of the Codes of Practice and how these affect the way in which the road network is managed and maintained.

It takes commitment, time and effort to develop and implement Asset Management practices that provide tangible benefits and are appropriate to the specific characteristics of a local authority. While this may appear a daunting task at the outset, it is essential that highway managers do not delay; otherwise they will fall behind their peers and be at a disadvantage compared to other sectors and departments within their own authority.

Identification of potential highways maintenance schemes using MCA



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Abstract

This paper presents a methodology for identifying potential pavement maintenance schemes based upon machine survey data. Highways authorities make maintenance decisions based on a number of factors; condition data is a common starting point. However there are currently no universally accepted techniques for ranking sub-sections by condition, based on the comparison of data from multiple condition assessment surveys. The methodology presented in this paper utilises condition data that has been captured from standard machine-based condition assessment surveys – specifically the TRACS survey, Deflectograph and the SCRIM survey. The resultant data is analysed using a Multi-Criteria Analysis (MCA) technique known as TOPSIS (the Technique for Order Preference by Similarity to the Ideal Solution). This technique is capable of investigating a number of alternative solutions. The data set is prioritised in terms of relative condition. The road sub-sections exhibiting the worst condition can then be highlighted, and potential schemes can be identified.

1. Introduction

Highway authorities develop short and long term road maintenance programmes based on various sets of pavement condition and performance data. Authorities must periodically highlight any condition hotspots and potential future maintenance schemes. This paper presents a methodology for assessing available condition data collected by machine surveys, to identify hotspots on a road network.

2. Condition assessment surveys and HAPMS

The measurement of road condition can involve either simple techniques, such as a visual survey of the road surface (Bandara, 2001), or more complex techniques, such as the analysis of laboratory samples. The condition of a road structure changes continuously over time, and therefore its condition data needs to be reviewed on a regular basis.

It is widely accepted that high-speed vehicle mounted apparatus provides the safest method of collecting condition data, when compared to inspectors performing walked-surveys in close proximity to live traffic. The Highways Agency actively promotes the development of machine-based survey systems for this very reason. Machine-based surveys also offer a quick and economical means of assessing the condition of a road network. These apparatus are capable of measuring a number of different pavement condition attributes.

There are a number of different defects that affect the condition of the road (Robinson, 1998). These include longitudinal-profile, transverse-profile, bearing capacity, surface friction, cracking, etc. Different condition survey machines capture different condition attributes and a consideration must be made as to which attributes are suitable to the identification of potential maintenance schemes.

In the UK, inventory data and condition data for the trunk road network is stored on a central database known as HAPMS (the Highways Agency Pavement Management System). HAPMS stores condition data collected by three machine-based condition surveys namely, Deflectograph, TRACS and SCRIM.

The Deflectograph survey measures the deflection values of a road pavement when subjected to a standard load by loading dual wheels on the rear axle of a suitable vehicle (HA, 2001). The Traffic-Speed Road Assessment Condition Survey (TRACS) records data that includes cracking and rutting (HA, 2005). Finally the Sideways Force Coefficient Routine Investigation Machine (SCRIM) measures low-speed skid resistance and assesses the condition of the microtexture of the surfacing (HA, 2004). The condition surveys mentioned here record a large number of attributes for each location along the survey route.

3. Pre-analysis

Appropriate condition parameters

The three condition assessment techniques described previously result in over 200 different attributes. After reviewing various HAPMS documents and following consultation with experienced pavement engineers, the following parameters were judged to be the most relevant, robust indicators of pavement condition:

Deflectograph

- Deflection 85th percentile
- Residual Life 15th percentile
- Required Overlay 85th percentile

TRACS

- Maximum Rut Category
- Texture Category
- Ride Quality (in terms of LPV 3m Category, LPV 10m Category and LPV 30m Category)

SCRIM

- SCRIM deficiency

These parameters were therefore used in the methodology outlined in this paper.

Sectioning

The three survey techniques employ two different sectioning techniques. The Deflectograph and TRACS surveys consider sections in 100m lengths whereas the SCRIM survey is based on varying lengths due to the assignment of risk values that relate to the road location and geometry.

It is therefore not possible to directly compare all SCRIM survey data with TRACS data. It was decided to consider the road data as homogenous sub-sections. This would divide the analysis into two parts and result in two outputs:

- A list of 100m road sub-sections prioritised in terms of condition with respect to Deflectograph and TRACS, and
- A list of road sub-sections of varying lengths prioritised in terms of condition with respect to SCRIM deficiency.

Inconsistent data

Due to the different scheduling intervals employed for the various surveys, network coverage is not complete. Although all sections should have TRACS and SCRIM data for Lane 1 and 2, the coverage is not total. An approach was developed to aid the analysis of non-existent data for those cases where surveys had not been conducted. This approach involved the rationalisation of the missing data.

The approach assumes that where data is missing, the missing survey data would have yielded average results. In other words, the average results for each criterion are calculated across the entire network and assigned to sections with missing survey data.

By rationalising the condition data, the data is in a form suitable to use as part of a methodology for identifying maintenance schemes.

4. Methodology

The methodology described in this section has been developed to utilise the data obtained from the steps described above.

The methodology employs a decision-making tool known as Multi-Criteria Analysis (MCA). There are a number of different MCA techniques. The Technique for Order Preference by Similarity to the Ideal Solution (TOPSIS) is one such methodology that was developed by Hwang and Yoon (Yoon, 1995). An MCA problem with m attributes and n alternatives can be visualised in terms of a geometric system with m points in n-dimensional space. The TOPSIS method is based upon the concept that the optimal solution is the solution that is closest to the positive-ideal whilst simultaneously being furthest away from the negative-ideal. Where no such solution exists, a trade-off between the two distances is calculated for each alternative.

The TOPSIS method is based upon the ranking of alternatives according to their score (Ci*). The alternative with the highest score is denoted as the optimal solution. The overall process is formulated by the following equation:

$$C_i^* = \frac{\sqrt{\sum_{j=1}^{n}\left(\frac{w_j\left(x_{ij}-\min\limits_{i=1,\ldots,n} x_{ij}\right)}{\sum_{i=1}^{m} x_{ij}}\right)^2}}{\left[\sqrt{\sum_{j=1}^{n}\left(\frac{w_j(x_{ij}-\max\limits_{i=1,\ldots,n} x_{ij})}{\sum_{i=1}^{m} x_{ij}}\right)^2}+\sqrt{\sum_{j=1}^{n}\left(\frac{w_j(x_{ij}-\min\limits_{i=1,\ldots,n} x_{ij}}{\sum_{i=1}^{m} x_{ij}}\right)^2}\right]}$$

EQUATION 1
OVERALL TOPSIS METHOD

Where:

x_{ij} = the score x for the alternative i under the attribute j

(i = 1,2,..,m), (j = 1,2,…,n)

w_j = the weight of attribute j

In MCA, the attribute weights (which represent the relative importance or value trade-offs of the attributes), are normally determined in accordance with the subjective preference of the decision makers or stakeholders. There are a number of different methods for determining weightings. However no method can guarantee a more accurate result than any other method, and the same decision maker may obtain different weightings using different methods.

With the diversity of attributes in the condition analysis problem, in conjunction with the multiplicity of stakeholders involved in road maintenance management decision making, it is difficult for an agreement to be reached on the relative importance via a subjective weighting process.

By recognising that no reliable subjective weightings can be obtained for the problem, the condition attributes are constructed in such a way that equal weightings can be applied; that is, there is no need for assigning attribute weights. The use of equal attribute weightings is in line with the principle of insufficient reason (Starr, 1977), which states that the use of equal weightings should be considered if the decision maker has no reason to prefer one attribute to another or when reliable subjective weightings are not obtainable.

In the methodology described in this paper, the TOPSIS model will consider each road sub-section and rank it in terms of its relative condition. The output will be a list of sections, ranked in terms of condition.

As the SCRIM data is expressed in terms of a single attribute, it is unnecessary to perform a MCA analysis; the list can simply be ranked in terms of SCRIM deficiency.

5. Analysis of Highway Agency Area 10 trunk road network

The methodology was applied to a road network within the UK. The UK's Highways Agency Area 10 road network covers the trunk roads in Cheshire, Merseyside, Greater Manchester and South Lancashire. In total the carriageway length of Area 10 is 1,500km. Area 10 condition data was extracted from HAPMS, the pre-analysis described above was applied, and the data subjected to the TOPSIS model.

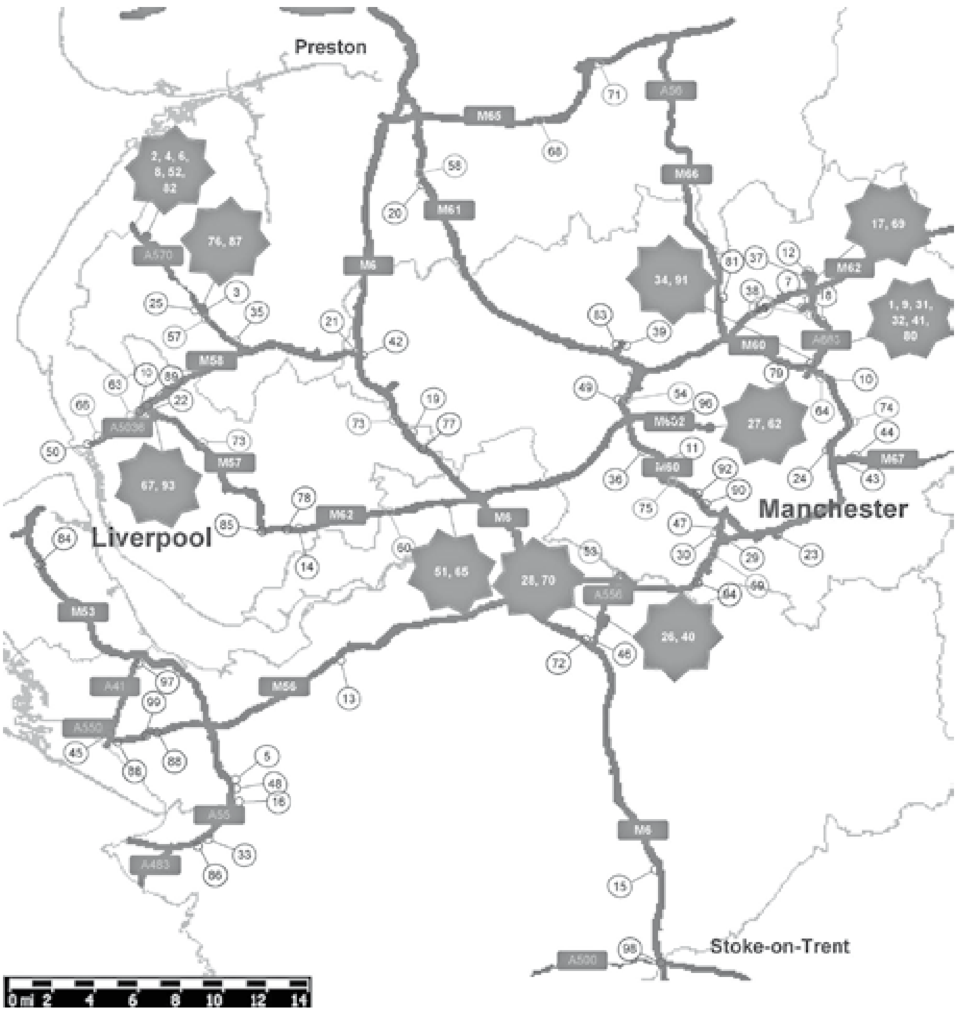

FIGURE 1
WORST 100 SUB-SECTIONS UNDER DEFLECTOGRAPH AND TRACS DATA

ASSET MANAGEMENT

Results of the Deflectograph and TRACS analysis

The analysis ranked 28,961 road sub-sections using Deflectograph and TRACS data. The sub-sections included 25,728 sections of exactly 100m in length and 3,233 sections of length less than 100m, i.e. residual lengths at the end of sections.

For illustrative purposes, the worst 100 sub-sections (0.3% of the network) were plotted on a map of Area 10. Of particular note are sections within a close proximity which are also on the worst 100 list. In the case where a section exhibits two or more 100m lengths in the worst 100 sub-sections, it is recommended that these should be considered for further investigation as possible maintenance schemes.

Results of the SCRIM ranking

The analysis of SCRIM data resulted in the prioritisation of 13,582 road sub-sections. The lengths of these sub-sections varied, with an average of 91m. For illustrative purposes, the worst 100 sub-sections, in terms of SCRIM difference, have been plotted geographically on a map of Area 10, as can be seen below.

Again, of particular note are sections in Area 10 that appear twice or more in the worst 100 sections.

FIGURE 2
WORST 100 SUBSECTIONS UNDER SCRIM DATA

6. Verification

In order to verify the analysis technique, the results were compared to schemes that have been carried out by Area 10 since 2001. If those parts of the network subject to schemes since 2001 are designated as being in poor condition in the ranked list, then some indication of the credence of the model can be established.

691 sub-sections maintained by Area 10 since 2001 were determined to be present in the Deflectograph and TRACS condition ranking.

Figure 3 shows the distribution of these schemes within the overall condition ranking (all 28,000 sub-sections subjected to analysis).

It is interesting to note that a significant majority of the schemes undertaken by Area 10 since 2001 include sub-sections indicated as falling within the worst 60-100% of the ranked sub-sections.

483 sub-sections maintained by Area 10 were present in the SCRIM condition ranking. Figure 4 shows the distribution of these schemes within the condition ranking.

The majority of the schemes undertaken in recent years appeared in the worst 60-80% of the ranked Deflectograph-TRACS list.

Few schemes were undertaken on the worst 50% of road sub-sections when ranked by SCRIM difference. By analysing the SCRIM rankings, it can be seen that the relative condition of sub-sections based on SCRIM data does not correlate with sub-sections where a maintenance treatment has been carried out. This is not surprising as the decision to treat a SCRIM deficient sub-section is rarely based solely on the SCRIM data. Other non-condition related factors are also taken into consideration, such as intervention level, accident rate etc.

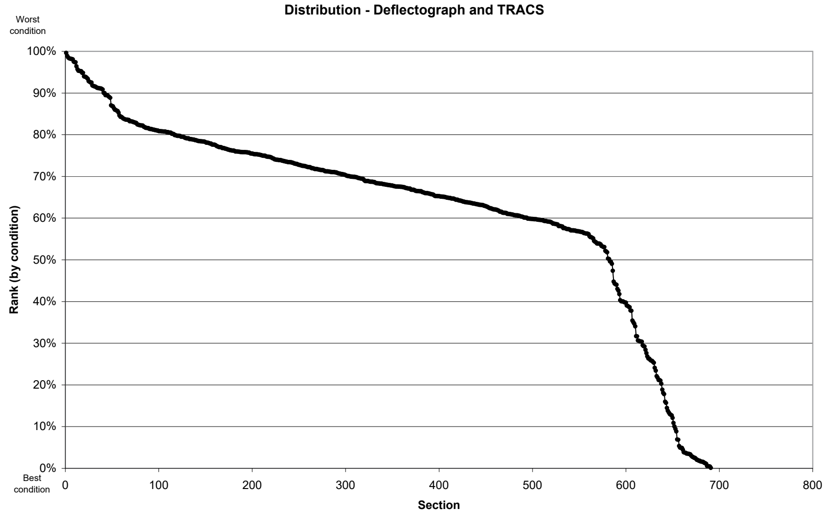

FIGURE 3
DISTRIBUTION OF SCHEMES USING DEFLECTOGRAPH-TRACS RANKING

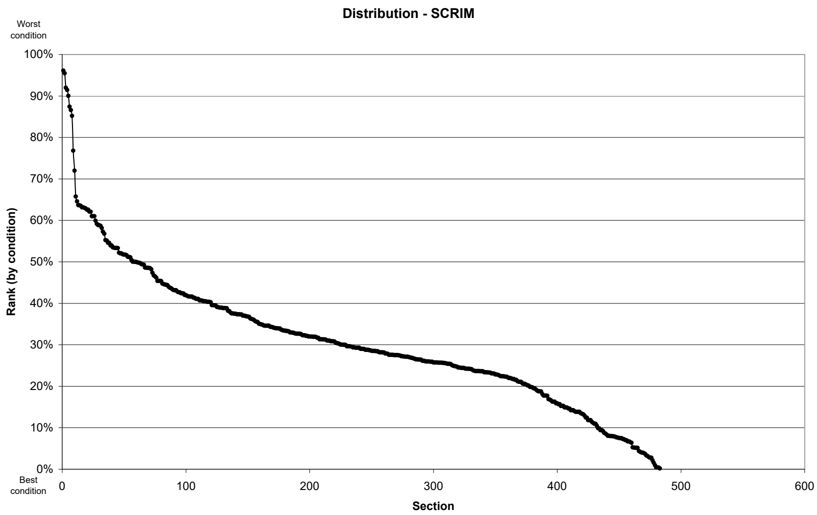

FIGURE 4
DISTRIBUTION OF SCHEMES USING SCRIM-RANKING

ASSET MANAGEMENT

7. Conclusions

This paper has presented a model for ranking a list of sub-sections within a road network in order to identify possible candidates for future works programmes. The sub-sections were ranked in terms of condition using an MCA methodology known as TOPSIS.

The final ranking is only as accurate as the data it is based upon. Therefore the robustness of the methodology is dependent on the accuracy of the data from the surveys, and the completeness and currency of the data in the HAPMS database.

To maximise the area of the network included in the analysis, assumptions were made to cater for missing data. These assumptions resulted in the adjustment of the data that could reduce the accuracy of the final ranked list.

The resultant ranked list of sub-sections is not intended to be used solely as a works programme. However the validation of the model showed that the Area 10 have already considered maintenance on a majority of the road sub-sections that were found to be to be in the worst 40% of condition of the condition ranking.

The model can therefore be used as a basis for a Highway Authority to identify potential schemes. By grouping clusters of road sub-sections (with low rankings) that are in close proximity to each other, a Highway Authority can identify candidate schemes and subsequently commission detailed pavement investigation to investigate the hot-spots further. The methodology provides a systematic, flexible framework that can allow for any number of condition attributes to be considered by the decision maker.

8. Ongoing work

Further validation of the methodology presented is underway. This work is specifically assessing the influence of Deflection data on the model.

The spreadsheet tools used for this study are being developed to allow the methodology to be an efficient process that can be used on an annual basis to update the outputs.

The viability of including of other condition attributes, such as cracking and fretting, is currently being assessed.

The applicability of this MCA technique to identifying potential schemes for non-pavement assets, such as vehicle restrain systems is being studied. This work is developing bespoke machine-based survey techniques and appropriate data assessment software.

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Composite Index of Sustainability (CIS)

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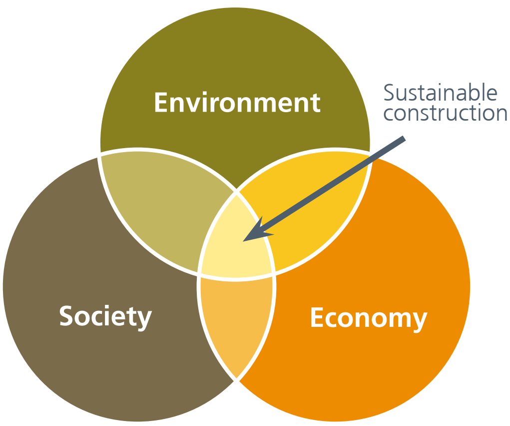
Abstract

"They go on in strange paradox, decided only to be undecided, resolved to be irresolute, adamant for drift, solid for fluidity, all powerful to be impotent." Winston Churchill, 1941

Sustainability is proving to be one of the greatest challenges facing the global community in the new century. Public and private sectors alike are coming under increasing pressure to embed the concept of sustainability into business, and to deliver policies, plans and projects focusing on the long-term view. The challenge facing practitioners is defining what sustainability means to each and every one of us, and turning 'rhetoric into reality' by evaluating and reporting performance in terms of sustainability.

1. Definition and scope

Sustainable Development is a term that is widely used, and one which requires a shift in our thinking to turn the cliché into a practical concept. Countless interpretations have been put forward and numerous appraisal methods exist, capable of measuring one, or all, aspects of sustainable performance dependant on the user's point of view. A general misconception is that sustainability deals solely with environmental impact, however 'to sustain' is merely defined as supporting a system for a long period of time. With this in mind, we look to the so-called 'Russian Doll Model', which is based on the principle that economic growth is defined by social well-being, which is in turn constrained by environmental factors. Economic capital drives development but is confined by our environmental status. Thus, one can not exist without the other. The convergence of these three components can be regarded as the achievement of sustainable development.



Sustainable Development

2. Strategy

Demonstrating and delivering sustainable solutions on planned construction and maintenance projects requires commitment and engagement from management, in order to realise the potential of incorporating sustainable priorities and factors into design. Effective project delivery is based on customer-focus and best value for money; in simple terms high monetary return, low capital expenditure. Incorporating Whole Life Costing (WLC) into the design process, and thereby emphasising the long-term view, enables a balance to be met between construction and maintenance costs; it may require short-term compromises but should provide long-term gain – 'spend to save'.

The movement towards sustainability is, generally, undisputed. The obstacle presented in designing for sustainable solutions is the anticipated higher investment cost over traditional methods. For sustainable solutions to be justified, design steps need to incorporate a development strategy, with the Whole Life Effect of a project incorporated. This should encompass not only financial return and environmental impacts, but all the underlying principles of sustainability such as societal effects, natural resources, health and safety, environmental and economic, in a single measure.

Developing a strategic approach that enables decision-makers to optimise designs whilst meeting objectives and targets, including client and stakeholder requirements, would enable an informed decision to be made that meets the needs of current and future users.

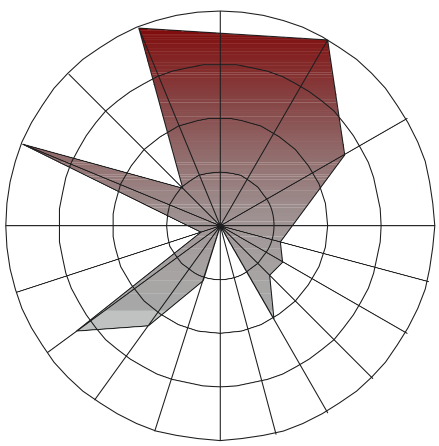
3. Sustainability appraisal

The Composite Index of Sustainability (CIS), established within the Highways Asset Management Group, was initially developed to evaluate the performance of highway renewals work, to compare differing renewals projects and facilitate examination of various solution options for the same projects. The CIS provides:

- A means by which a number of discrete features may be encapsulated
- A single composite measure of effectiveness.
- A composite parameter against which benchmarks and targets may be set
- An 'all around' measure which can be used to demonstrate continuous improvement
- An index which can be readily understood by practitioners
- A single measure which can be used in value management considerations.

The visual impact of the vector approach to the determination of sustainability enables focus to be directed towards those factors that have a major effect and provides a pathway for continuous development.

The outcome and success of a project is predominantly determined in the design stage, allowing a project to be modified to mitigate negative impacts and to enhance benefits. Employing the CIS improves decision-making, enables the application of engineering rigour in the determination of sustainability, and provides a monitoring tool and post-assessment evaluation to determine the strength of predictions, the success of changes made and possibility for future improvements.



SUSTAINABILITY APPRAISAL -
COMPOSITE INDEX OF SUSTAINABILITY

4. Future developments

The CIS is in continual development to enable its application to a wider audience, to incorporate a wider range of indicators and significant factors that best represent sustainable performance and to ultimately produce a sustainability appraisal toolkit that can be easily tailored to and facilitate numerous project disciplines. The CIS is currently in development for use in highway structures projects, and in Intelligent Transport Systems (on Controlled Motorway Schemes and within Control Rooms).

Industry research is being supported by an academic institution through a PhD study title, 'Modelling and Analysis in Engineering for Sustainable Development'. This academic research aims to develop a unique sustainability toolkit capable of evaluating the performance of highways design and maintenance schemes, with potential for application to a wide range of engineering disciplines.

The development of an 'all-embracing' sustainability appraisal toolkit requires commitment, time and resourcing by all involved in designing for enhancement of benefits and condensing negative impacts, so that what we design for today can still be justified in decades to come.

With reference to the quote at the start to this paper, we need to take heed of what the specialists are saying, and to take the initiative to act now, however small our efforts, so we do not act too late or fall behind the inevitable wake of the industry.