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Technical Journal

Papers 016 - 033

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

Chris Hendy
Chair of Technology Board

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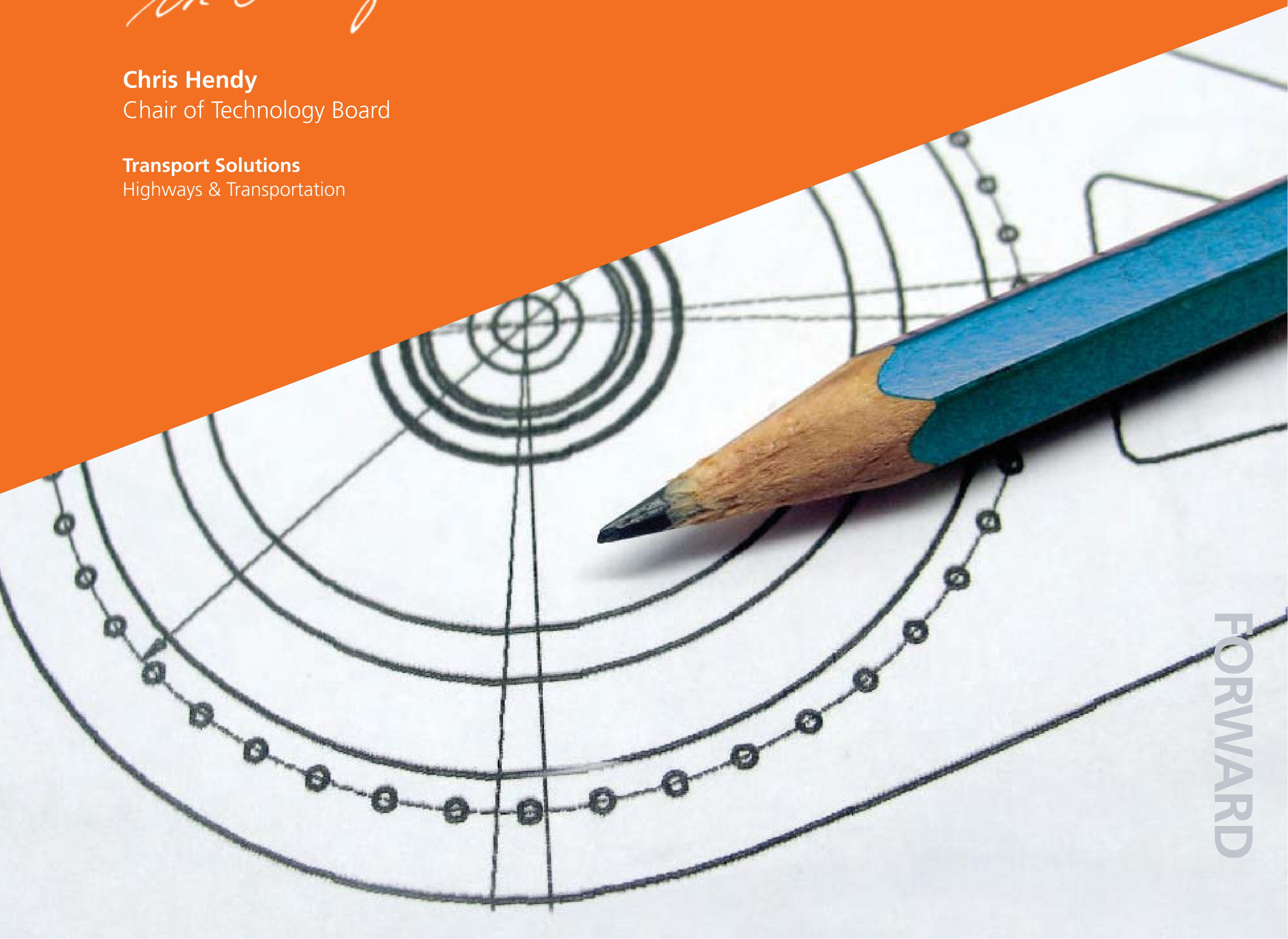

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Bahrain World Trade Centre

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Abstract

The Bahrain World Trade Centre (BWTC) development comprises twin 240m high office towers rising above a 3-storey podium development adjacent to the Sheraton Hotel in the Commercial Business District of Manama, Bahrain. The design features three horizontal axis wind turbines mounted between the towers, taking advantage of the prevailing onshore wind to generate electricity, which posed unique challenges that had to be overcome. This paper covers the basis of design, the wind tunnel testing undertaken for the project and the ground conditions encountered on the site.
The foundation design is described, including consideration of soil-structure interaction effects at the interface between the piled and ground bearing foundation elements. The paper also describes the framing systems adopted for the primary reinforced concrete structure and the floors, with discussion of the benefits of early contractor appointment on the selection of floor systems and formwork.
The structural steelwork elements, including the spire and the panoramic lift enclosure, are also described.

1. Introduction

Bahrain World Trade Centre is located within the centre of the Kingdom's Commercial Business District, Manama, with direct access off the King Faisal Highway. It forms the focal point of a masterplan to rejuvenate an existing hotel and shopping mall. Rising to 240 metres in height, the twin office towers create a dramatic focal point visible throughout the Kingdom.

The concept design was inspired by the traditional Arabian "Wind Towers" in that the shape of the buildings harness the prevailing onshore breeze from the Gulf. Three 29-metre diameter horizontal axis wind turbines are mounted on bridge structures between the sail shaped towers, providing a renewable source of energy for the project.

In addition to the office towers, the development includes an extension to the existing retail mall, new restaurants and cafes, refurbishment of the adjacent 5-star hotel and 1,700 parking spaces.

The twin towers are framed predominantly in reinforced concrete, with structural steelwork used for the clad steel spire, panoramic lift enclosure and mezzanine floors. There are 45 floors (excluding mezzanines) from the ground floor of the podium, as well as a 4.5 metre deep single storey basement.

FIGURE 1. BAHRAIN WORLD TRADE CENTRE DURING CONSTRUCTION WITH THE TOWER CRANE STILL IN PLACE, JUNE 2007

The design was carried out on a fast-track basis following an early appointment of the contractor, Nass Murray & Roberts Joint Venture, The contractor provided value engineering input at the design stage, particularly on the selection of floor and formwork systems.

2. Wind environment and wind tunnel testing

Following consultation with the authorities in Bahrain, a BS 6399-2 mean hourly basic wind speed of 26 m/s was adopted for the design. Wind tunnel testing was undertaken by BMT, modeling both the towers and the surrounding low and medium rise buildings. A force balance test was undertaken to determine structural loads and to predict accelerations at the highest occupied floors. A separate pressure-tapping model was used to determine cladding pressures. The two models were used together in determining the downstream effects of one tower on the other. Tests were also undertaken to evaluate pedestrian comfort at the ground floor and in public spaces on the podium levels.

A cautious approach was adopted in interpreting the structural loads derived from the wind tunnel testing. This was because of uncertainty regarding directional factors appropriate for extreme storm events, particularly given the limited available wind data. As a result, it was decided that a minimum directional factor of 0.9 would be used. This still gave reductions of up to 20 percent for critical wind directions where peaks in wind load indicated aerodynamic lift occurring due to the plan shape of the towers. The peak wind load on each tower derived from the wind tunnel test results was approximately 16MN.

Building accelerations were calculated in order to allow occupancy comfort to be assessed at the upper floors. The results of this assessment, based on 1.5% damping and a 5 year return period were below the criteria of 16 milli-g peak acceleration at the highest office floor.

An assessment was also carried out on aero-elastic instabilities that might affect the building. In particular, the plan shape adopted for the upper section of the towers is known to be susceptible to galloping instability. The critical wind speed above which galloping instability might occur was assessed and it was found that there was a significant margin above the 50-year return period wind speed for which the building was to be designed. The conclusion was that galloping instability was very unlikely to occur and that the level of risk was acceptable.

3. Ground conditions

A ground investigation was undertaken by Al Hoty Analytical Services during March and April 2004. The site is formed on reclaimed land comprising loose to medium dense silty fine sand. The depth of fill above the old seabed ranges from approximately 2.5m to 3.5m and the old seabed comprises a zone of nominally 1.0m thick very soft silty clay.

The old seabed is underlain by weak carbonate sandstone (Calcarenite) and carbonate siltstone (Calisiltite) to approximately 31m, with instances of bands of very weak completely weathered siltstone.

The carbonate siltstone is underlain by moderately strong Limestone at a depth of approximately 31m. The limestone was generally proven to a depth of 36m.

Ground water was observed at approximately 1.8m depth within the made ground and the results of chemical tests indicated high sulphate concentrations in the soils.

4. Foundation solution

Each tower has a separate continuous piled raft foundation at basement level. The raft slabs vary in thickness according to loading and incorporate lift pits. Beneath the main cores the raft thickness is 3.0 metres and the piles are 1200mm diameter, closely spaced and rated at 18MN safe working load. Away from the main core the raft thickness reduces progressively to 2.0 metres and the piles to 1050mm diameter, more widely spaced and rated at 8MN safe working load.

The loads acting on the pile group are predominantly dead loads, imposed loads and wind loads. These act in combination to generate maximum and minimum pile loads. Individual piles were modelled in the finite element analysis as vertical springs. A parametric study was carried out investigating a range of pile stiffnesses. One case in particular modelled the tendency for increased pile loads around the periphery of the pile group (as typically observed in the analysis of pile groups in an elastic continuum). Depending on the assumptions regarding pile stiffness, the maximum pile loads occurred either near the centre of the main core area, or around the perimeter of the group. Some reversal of pile load was observed on the perimeter of the group under minimum dead load combinations.

Bending moments and shear forces for design of the raft and pile loads were derived from the finite element analysis model and by hand calculation- see Figure 2 below.

The piled raft slabs beneath the towers were made continuous with the ground bearing podium basement slabs. This approach was considered preferable to separating the slabs, because of the difficulties in achieving watertight construction with a movement joint.

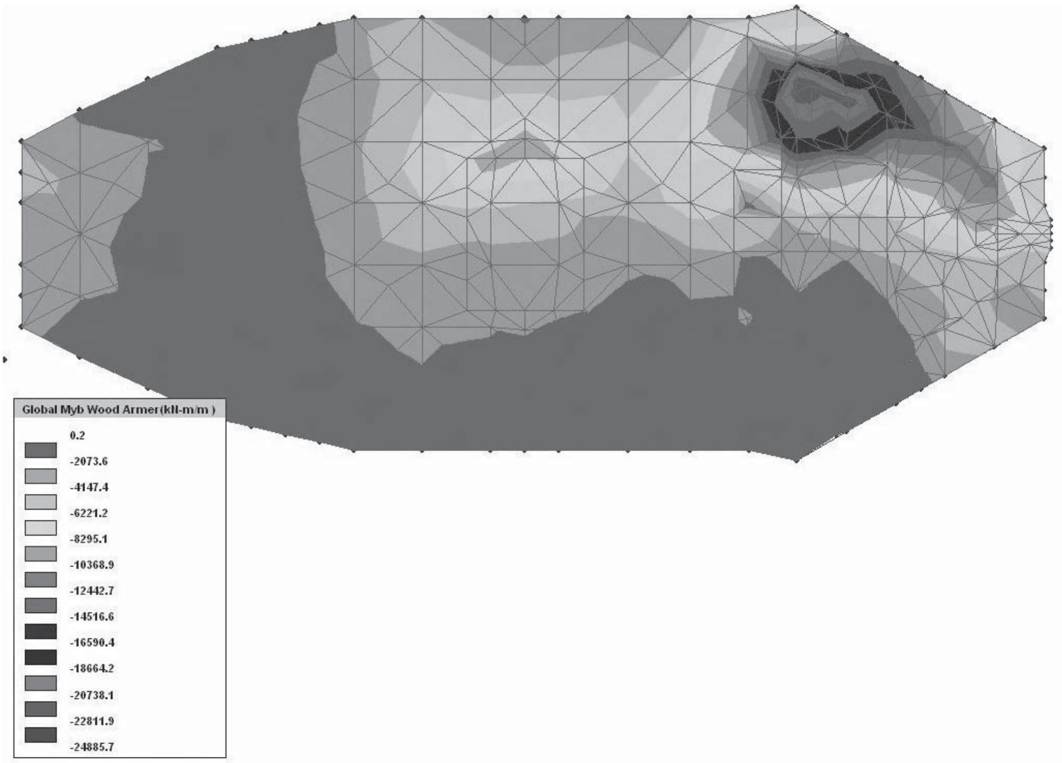

FIGURE 2. ANALYSIS MODEL OF THE PILED RAFT

A two dimensional finite element model was used to investigate soil-structure interaction effects and this model was used to estimate the moments and shear forces in the slabs at the interface between the two foundation systems.

Lift pits within the raft slab were typically relatively shallow because the ground floor was the lowest floor served (so most of the height required for over-run and buffers could be accommodated over the basement storey height).

Service trenches were avoided in the raft slabs, but allowance was made for ducts within the depth for cabling.

External protection was provided to the raft slabs by a two-layer SBS modified bitumen membrane tanking system. The concrete mix incorporated GGBFS in order to enhance the durability of the concrete and to control maximum and differential early age concrete temperatures.

5. Primary structural system

The twin towers are in a 'V' formation, typically mirrored about their axis of symmetry. There are some differences in plan over the height of the podium.

The primary structure comprises main and secondary reinforced concrete cores, the main core housing lifts, escape stairs, plant rooms and toilets and the secondary core housing an escape stair and electrical/telecoms rooms.

The floor plates typically having a storey height of 3.6 metres and are framed with reinforced vertical concrete columns on an 8.0m grid and raking columns which follow the sloping face of the building as it tapers in elevation.

The wind load on the towers is predominantly resisted by the main concrete core. About the weak axis, the secondary core relieves some of the load on the main core, with load transfer between the two cores occurring in the vicinity of the 20th to 24th floor levels.

On the strong axis, the main core and the secondary cores are linked only by the floor diaphragms, but the four raking columns triangulate the cores, which has a stiffening effect. The outer pair of raking columns, following an arc on plan, also have a stiffening effect about the minor axis and tend to attract reversible forces under wind loads.

At the upper levels the secondary core first terminates and the main core then extends to the height of the highest office floor. When this core also terminates, the panoramic lift core extends further stabilising the duplex offices and viewing gallery at a higher level. Above this height the top clad section of the building is framed in lattice steelwork construction to reduce weight. See Figure 4.

The structure was modelled using the S-Frame finite element analysis program. In the model, the raft slab, core walls and floor slabs are represented by shell elements and the columns and steel sections are modelled as beam elements. The raft was supported on springs representing the piles.

Initially, a relatively coarse model was set up in order to obtain the natural frequencies and mode shapes for the structure. This data, together with information about the centre of mass and shear centre for the structure was used by the specialists undertaking the wind tunnel testing to predict the dynamic response of the building.

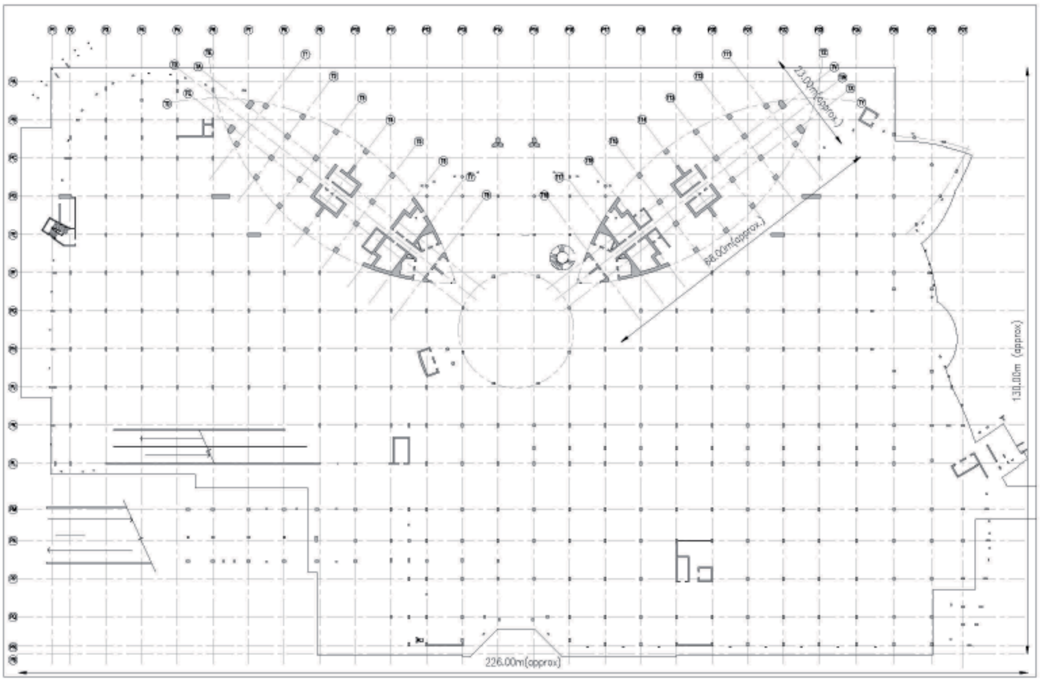

FIGURE 3. PLAN LAYOUT OF THE PODIUM AND TOWERS AT GROUND LEVEL

Once this phase of the work was completed, the model was refined to more accurately model the core walls and raft slab in order to obtain stresses and member forces for final design.

The columns are typically square or rectangular, either 1000mm x 1000mm, or 1000mm x 800mm where they are less heavily loaded. Up to Level 18, where the column loads are typically compressive, Grade C60/20 concrete is used to avoid the need for a larger column size. Above this level the loads have reduced and the concrete grade is reduced to C45/20.

Some of the columns at the upper levels are subject to reversal of forces under wind load. The outer raking columns in particular contribute to resisting wind loads and increase in size to 1500mm x 1000mm above the 36th Floor to accommodate rebar resisting tension.

There are no transfer structures for vertical loads, but at several locations the floor diaphragm is relied upon to cater for forces due to a change in slope of the raking columns.

The design of the core walls is influenced by the need to provide stiffness and reduce sway deflections. The thickness of the primary walls reduces from typically 600mm at basement level (750mm maximum) to 400mm at the upper floor levels. Secondary walls around lift shafts and within stair cores are a constant 300mm thickness over their full height.

At four locations on plan, coupling beams link the core areas either side of the main corridor. These coupling beams are 750mm deep and because of the relatively high stresses due to interaction between the cores, all services pass beneath the beams in a dedicated services zone agreed with the M&E engineers and no penetrations for services through the beams were permitted.

6. Floor system

Alternative floor systems were discussed at an early stage with the contractor. The original solution was a 375mm overall depth in situ wide trough floor system with secondary beams typically at 4 metre centres and primary beams at 8 metre centres. However, this was adapted, eliminating the secondary beams and using 150mm hollowcore units with a 75mm structural topping in lieu of the 150mm solid slabs over the wide troughs originally envisaged. The advantages of this system included:

- Relatively lightweight with less reinforcement than a flat slab
- Partial precasting of the slab using locally available hollowcore planks
- Beamstrips cater for edge cantilevers and re-entrant corners
- Fan coil units can be integrated between the beamstrips, effectively increasing the services zone
- Good diaphragm action compared to fully precast options

Floor slabs in the core areas are typically in situ concrete and stair flights are of precast concrete construction taking advantage of the accuracy of off site precasting.

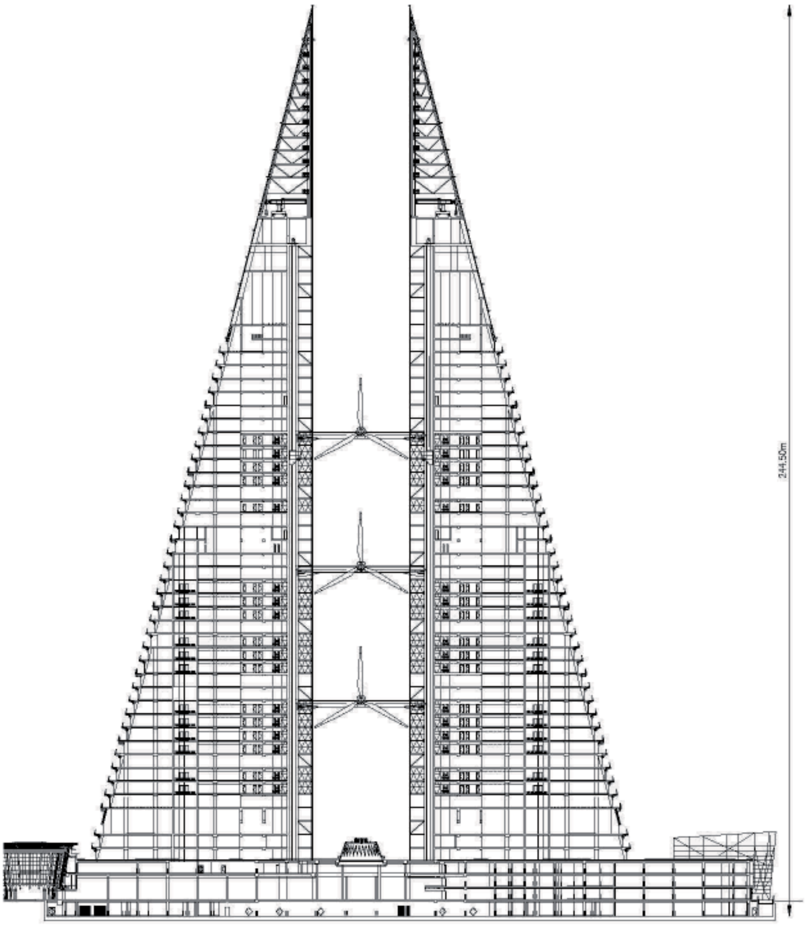

FIGURE 4. SECTION THROUGH THE TOWERS

7. Wind turbines and support bridges

The wind climate in the Arabian Gulf with its dominant sea breeze characteristic is conducive to harnessing wind energy.

The Client readily embraced the concept to portray to the world that Bahrain is committed to options that reduce demand on fossil fuel energy reserves and will move urban and building design in desert climates in a more sustainable direction.

From the outset this project had as its primary basis of design the utilisation of conventional technologies and the development of a built form that would be sympathetic to receiving wind turbines. Three horizontal axis wind turbines have been integrated into the building to generate electricity. The premium on this project for including the wind turbines was less than 3% of project value.

The fixed horizontal turbine suffers the drawback of only being able to operate with wind from a limited azimuth range - if problems with blade deflections and stressing through excessive skew flow are to be avoided.

The sail profile of the towers act as aerofoils, funnelling the onshore breeze between them and accelerating the wind velocity between the two towers. Vertically, as the towers taper upwards, the aerofoil sections reduce. This effect when combined with the increasing velocity of the onshore breeze at increasing heights creates a near equal regime of wind velocity on each of the three turbines.

Extensive wind tunnel modelling that was latterly validated by CFD modelling has shown that the incoming wind is, in effect, deflected by the towers in the form of an S-shaped streamline, which passes through the space between the towers at an angle within the wind skew tolerance of the wind turbine.

The fixed, horizontal axis wind turbines on this project have the following characteristics:

Nominal electrical power generated	225kW
Power regulation	Stall
Rotor diameter	29m
Rotor speed at full load	38rpm
Cut in wind speed	4m/s
Cut out wind speed	20m/s

The nacelles have been designed to sit on top of the bridges, rather than within them, to portray the functionality of the turbine. Each turbine weighs approximately 11 tonnes. Stall control is used as a passive way of limiting power from the turbine. The full power of about 225kW will be achieved at 15 to 20m/s.

FIGURE 5. 3-D ANALYSIS MODEL OF ONE TOWER

The bridges are ovoid in section for aerodynamic purposes and are relatively complex structures because they incorporate low maintenance bearings where they connect to the buildings to allow the towers to move up to 0.6m relative to each other. In addition, the bridges that span 31.7m and support the wind turbines have been designed to withstand and absorb wind induced vibration and vibrations induced by both an operating and "standstill" turbine.

Further precautions are included in the design to allow the bridge to be damped, if in practice vibrations are found to be problematic during commissioning. These precautions include the facility in the design to add tuned mass dampers within the bridges.

The bridge is a shallow V-shape in plan (173º) to take account of blade deflection during extreme operating conditions and to afford adequate clearance and thus avoid blade strike.

The projected energy yield from the turbines taking into account wind and availability data amounts to between 1,100 and 1,300 MWh per year and will provide for approximately 11% to 15% of the office towers' electrical energy consumption.

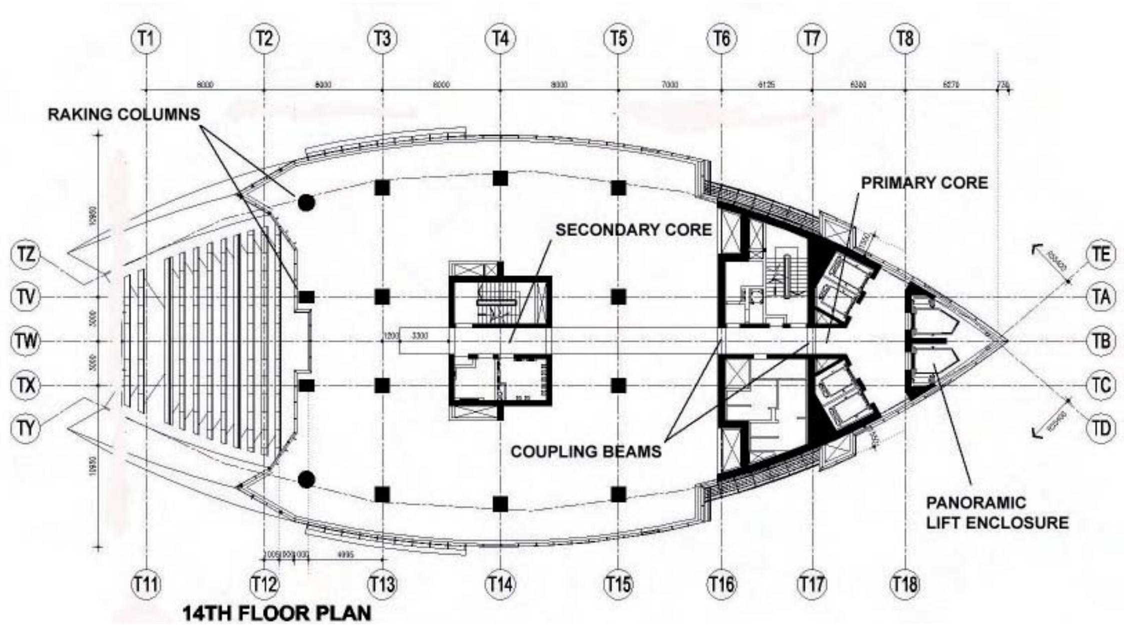

FIGURE 6. TYPICAL TOWER FLOOR PLAN

FIGURE 7. PHOTO OF THE TURBINE
BRIDGE AND TURBINE/PROP

8. Building maintenance unit and spire

The steel framed lattice structure supporting the cladding over approximately the top 60 metres of the height of the building is of tubular steel construction with curved horizontal hollow section rails supporting the cladding system every 3.6 metres in height.

The founding level for this 'spire' is the roof slab over the lift motor room for the panoramic lifts. This slab also supports the Building Maintenance Unit (BMU) for window cleaning, housed within the lower section of the spire with openable elements in the cladding provided to allow full retraction of the BMU equipment. Holding down bolts for the steelwork are sleeved post-tensioned Macalloy type bolts.

9. Panoramic lifts

The panoramic lift enclosure is also framed in tubular structural steelwork. The 'V' section horizontally curved frames are at 3.6 metre vertical centres. The frames are suspended from bracing back to the panoramic lift core with expansion joints in the vertical elements provided to limit stresses due to temperature effects.

10. Concluding remarks

The Bahrain World Trade Centre is the world's first large-scale integration of wind turbines into a building. This integration was one of the principal challenges on the project. The turbines were commissioned in April 2007, and global interest was received from environmental and architectural bodies, media and private institutions across the world.

Other challenges on the project included the 'fast-track' approach adopted for design and construction, the sheer scale and complexity of the project and the integration of the reinforced concrete, structural steelwork and cladding elements of the design.

With their distinctive instantly recognisable design and utilisation of wind power, the twin towers are likely to become well known worldwide and contribute to developing Bahrain's reputation as an appealing destination.

Acknowledgements

Building Architectural and Engineering design was by Atkins Middle East. The wind turbine and bridge design consultants were Ramboll Danmark A/S. The turbines and associated equipment were supplied and installed by Norwin A/S. Wind tunnel testing was undertaken by BMT and Ramboll. Atkins Science & Technology in the UK have executed a high-level technical review. The principal contractor was Nass Murray & Roberts Joint Venture.

This paper was presented at the International Federation of High Rise Structures Conference held in Abu Dhabi, December 2008.

FIGURE 8. COMPLETED BAHRAIN WORLD TRADE CENTRE

Transverse web stiffeners and shear moment interaction for steel plate girder bridges

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Abstract

Many investigations have been carried out to date into the behaviour of transversely stiffened web panels in bending and shear and many different theories have been proposed. Different code rules have been developed based on these theories. The UK's steel bridge code, BS 5400 Part 3, based its design rules for transverse stiffeners on the work of Rockey et al., while early drafts of Eurocode prEN 1993-1-5 were based on the work of Höglund. The formers tension field theory places a much greater demand on stiffener strength than does the latter's rotated stress field theory. Due to a lack of European agreement, EN 1993-1-5 was modified late in its drafting to include a stiffener force criterion more closely aligned to that in BS 5400 Part 3. The rules for stiffener design in EN 1993-1-5 are thus no longer consistent with the rotated stress field theory and lead to a greater axial force acting in the stiffener. The rules for the design of the web panels themselves in shear however remain based on Höglund's rotated stress field theory, creating an inconsistency.

Recent investigations by the authors have suggested that the rules in BS 5400 Part 3 and, to a lesser extent, in the current version of EN 1993-1-5 can be unduly pessimistic. This paper investigates the behaviour of transversely stiffened plate girders in bending and shear using non-linear finite element analyses. It considers slender symmetrical steel girders with and without axial force and also steel-concrete composite plate girders, which are therefore asymmetric. It discusses the observed web post-buckling behaviour, compares it with the predictions of current theories and recommends modified design rules. It includes investigation into whether a stiffness-only approach to stiffener design can be justified, rather than a combined stiffness and force approach. The shear-moment interaction behaviour of the girders is also investigated and compared to the codified predictions of EN 1993-1-5.

1. Background

It has been known since the 1930s that transversely stiffened web panels in bending and shear had a post-critical resistance, but only in the 1950s was the behaviour properly investigated[1&2]. Since then, many investigations have been carried out and many different theories have been proposed. The shear resistance theories behind most codes assume that the web operates in pure shear until elastic critical buckling occurs; subsequently, bands of tension form to carry further increases in shear. What is not agreed at present is the role of intermediate transverse stiffeners when these tension fields develop and, in particular, the forces they attract.

Rockey's tension field theory[3-5] places a much greater demand on stiffener strength than does Höglund's rotated stress field theory[6]. Rockey's theory requires the stiffeners to play the role of compression members in a truss, with the web plate acting as the tension diagonals. Höglund's theory does not require the stiffeners to carry any load other than that due to the small part of the tension field anchored by the flanges at collapse; no force is induced in the stiffeners in mobilising the post-critical resistance of the web. In the absence of a stiff flange to contribute to the shear resistance, the stiffeners simply contribute to elevating the elastic critical shear stress of the web. This has led to some European countries adopting a stiffness-only approach to stiffener design in the past. Adequate stiffness is simply required to ensure that the theoretical elastic critical shear resistance of the panel is achieved, or at least very nearly achieved since no stiffener can be completely rigid.

Earlier drafts of prEN 1993-1-5[7] required web stiffeners to be designed for a force loosely (but not exactly) based on Höglund's theory, together with a check for adequate stiffness. These early drafts raised concern in the UK as the rules led to much smaller forces in the stiffeners than would be derived from the tension field theory approach

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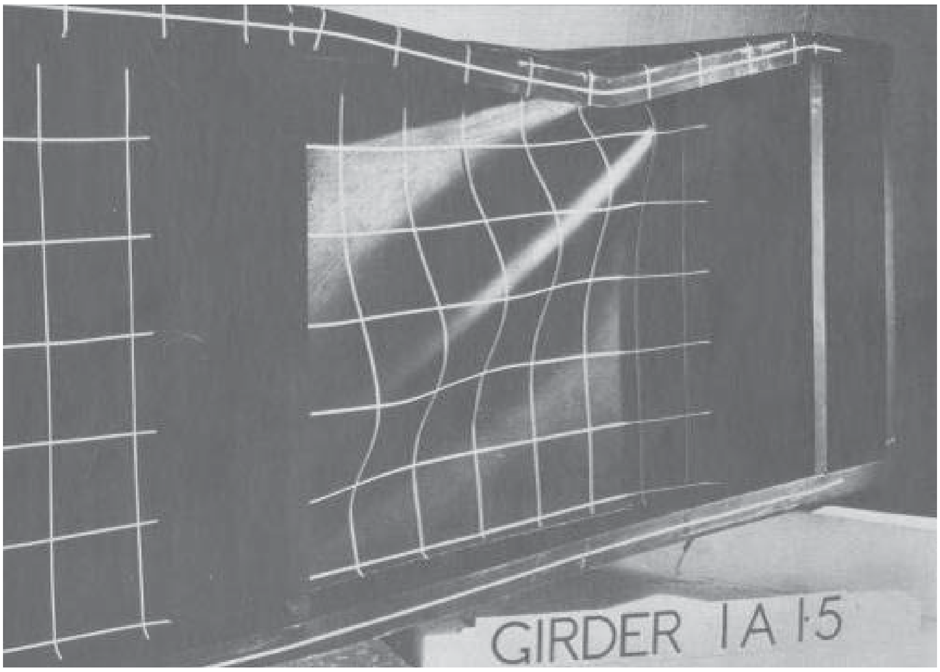

FIGURE 1. HIGH SHEAR TEST[9]

traditionally used in BS 5400:Part 3[8]. As a result, EN 1993-1-5 was modified late on its drafting to include a stiffener force criterion more closely aligned to that in BS 5400:Part 3. The rules for stiffener design in EN 1993-1-5 are thus no longer consistent with the rotated stress field theory and lead to a significantly greater axial force acting in the stiffener, with a consequent loss of economy. The rules for the design of the web panels themselves in shear however remain based on Höglund's rotated stress field theory, creating an inconsistency.

It is undesirable that EN 1993-1-5 should contain a rule for stiffener design that is incompatible with its rules for shear design. It is also undesirable to have a rule that is unnecessarily conservative, particularly if employed to assess existing structures, although it is noted that assessment is strictly outside the scope of the Eurocodes. This paper therefore studies, with the use of a non-linear finite element analysis package, the behaviour of transversely stiffened plate girders and seeks to determine:

1) The true mechanism for resisting shear and its interaction with moment
2) A prediction of the true forces generated in stiffeners and the potential adequacy of a "stiffness-only" approach to stiffener design, based on the stiffness criterion already given in EN 1993-1-5
3) The effects of panel aspect ratio on the collapse load
4) The effects of the ratio M/V of bending moment to shear force on the collapse load and comparison with moment-shear interaction diagrams produced by Eurocode EN 1993-1-5

2. Guidance on Highway Asset Management

Experimental studies have indicated that, when a thin walled plate girder is loaded in shear, failure occurs when the web plate yields under the joint action of the post-buckling membrane stress and the initial elastic buckling stress of the web panel, and plastic hinges develop in the flanges, as shown in Figure 1.

2.1 Cardiff tension field theory

According to the Cardiff tension field theory developed by Rockey et al.[3-5], transverse stiffeners have to fulfill two main functions. The first is to increase the elastic critical buckling resistance, V_{cr}, of the web plate. The second is to act as part of a truss when the web develops a diagonal tension field when the shear force exceed V_{cr}. This effectively leads to the stiffener force being equal to $V_{Ed} - V_{cr}$ for a given shear loading $V_{Ed} > V_{cr}$. This theory is not explained further here as it was not found to be a good predictor of stiffener force or behaviour in this study.

2.2 Stockholm rotated stress field theory

The rotated stress field theory developed by Höglund[6] forms the basis of the design rules in EN 1993-1-5 for the calculation of the ultimate shear resistance of plate girders. The ultimate shear resistance V_{ult} can be expressed as:

$$V_{ult\Delta} = V_{u\,w\Delta} + V_{u\,f\Delta} \quad (1)$$

where $V_{u,w}$ is the load carrying resistance of the web due to its membrane behaviour and $V_{u,f}$ is the resistance provided

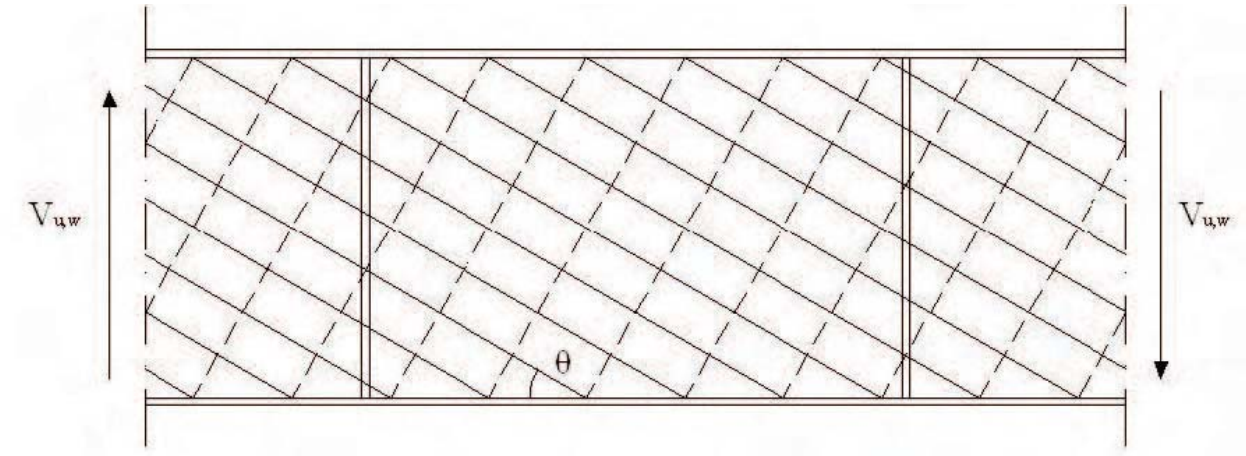

FIGURE 2. SHEAR FORCE CARRIED BY THE WEB

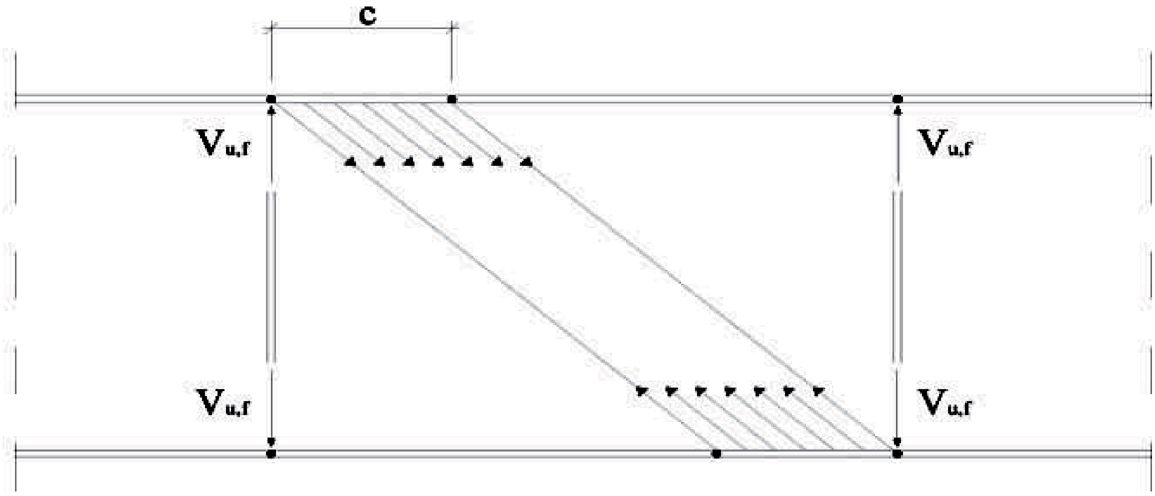

FIGURE 3. SHEAR FORCE CARRIED BY TRUSS ACTION

by an additional tension field anchored by the flanges. In determining $V_{u,w}$ the web panels are represented, in the post-buckling stage, with a system of perpendicular bars in compression and in tension, as shown in Figure 2.

When the load increases, the stress $_{c}$ in the compression bars is constant and equal to the buckling stress $_{cr}$ while the tension bars stress t increases when the angle decreases. This behaviour produces a net axial membrane tension in the web. The value $V_{u,w}$ is obtained when plasticity is reached at the intersection between bars, according to the Von Mises yield criterion.

At failure, four hinges form at the top and bottom flange, with an additional tension stress field developed in the web as shown in Figure 3. The moment at each hinge is assumed to be equal to the plastic moment of the flanges.

The shear force $V_{u,f}$ which is transmitted by the additional tension stress field is obtained from the equilibrium of the flange portion c. This equation gives:

$$V_{u,f} = \frac{4M_{fp}}{c} \quad (2)$$

where c is the distance at which plastic hinges form in the flanges. The stronger the flanges, the greater the dimension c. The stiffener force is thus equal to $V_{u,f}$; no force is predicted in mobilising the post-critical resistance of the web plate alone.

2.3 Design rules for transverse stiffeners in EN 1993-1-5

For the reasons discussed above, the design rules for transverse web stiffeners in EN 1993-1-5 are based on the Cardiff theory, even though the basis for its shear resistance rules is the Stockholm theory. Specifically, the stiffener effective section must resist the force from shear tension field action according to clause 9.3.3, together with any externally applied forces and moments. The tension field force is essentially equal to the difference between the web applied shear and elastic critical shear force:

$$V_{Ed\Delta} - V_{cr\Delta}$$

although a material factor is introduced on V_{cr}.

An additional stiffness criterion is given to ensure that the panel boundaries are sufficiently stiff to produce very nearly the full elastic critical shear resistance of a panel with pinned boundaries.

This is that the stiffener effective section must have second moment of area

$I_{st} \geq 1.5 d^{3} t^{3} / a^{2}$ if a/d $< \sqrt{2}$ and $I_{st} \geq 0.75 d\, t^{3}$

if a/d $\geq \sqrt{2}$

In the above requirement, h_{st} is the stiffener height, f_y is the steel yield stress, a is the panel length, d is the panel depth, t is the web plate thickness, and I_{st} is the stiffener second moment of area.

The remainder of the paper seeks to establish the true behaviour and to relax the EN 1993-1-5 force criterion.

3. Finite element modelling

To investigate the true behaviour of stiffened plate girders in shear and bending, a series of finite element models were set up using the software package LUSAS. The basic girder layout modelled is shown in Figure 4. This comprises an inverted simply supported beam with twelve square panels of length (and height) equal to 2.5 m, giving a total length of 30 m. By using this beam layout, the web panel aspect ratios a/d can easily be varied by removing stiffeners. Global lateral torsional buckling was restrained in the models by providing adequate lateral restraint to the compression flanges. The geometry was intended to simulate the moment and shear loading developed in a girder over an internal support of a continuous bridge.

Two different beam geometries were considered:

- Symmetrical steel girder: a bare steel plate girder with double-sided stiffeners (to eliminate bending effects in the stiffeners from asymmetry) Axial force was also applied in some cases to examine the influence of axial force
- Steel-concrete composite girder: a steel plate girder with a concrete slab on top with single-sided stiffeners, representative of a real bridge beam. The bending moment thus induces a net axial force in the web due to the eccentricity of the neutral axis in the web

In all cases, the intermediate stiffeners were sized such that they just met the EN 1993-1-5 stiffness criteria but would generally not meet the EN 1993-1-5 force criterion; the purpose being to test the conservatism of the force requirements and investigate whether a stiffness-only approach to design would suffice.

Imperfections

Four different initial imperfection types were modelled to investigate the sensitivity of the final collapse mode and load factor to imperfections. These were:

i) Web panel bows

ii) Stiffener bows

iii) Overall imperfection geometry based on elastic critical buckling modes

iv) Imperfection geometry based on a collapse mode from a previous analysis

The latter usually produced the lowest load factor, but load factors were found not to be very sensitive to imperfections in general.

Material properties

The steel yield strength of the plate girder components was set at 355 N/mm^2. The material factor $_M$ was taken as 1.0 for all finite element model components, to enable comparison with code predictions using $_M$ =1.0. After yield, the steel stress-strain slope was set at E/100, in accordance with the recommendations in EN1993-1-5 Annex C.6, to model the effect of some strain hardening. Fracture was assumed to take place at a strain of 5% as shown in Figure 5.

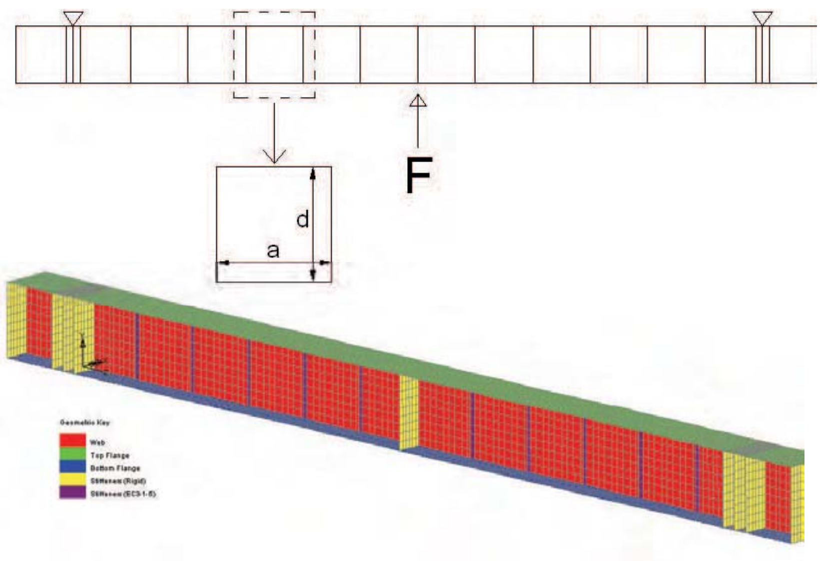

FIGURE 4. GIRDER LAYOUT USED IN FINITE ELEMENT MODELLING

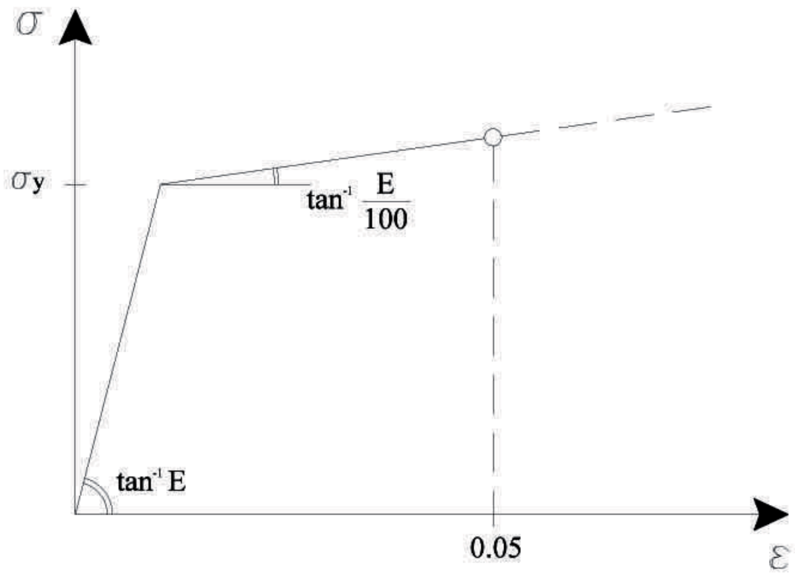

FIGURE 5. MATERIAL BEHAVIOUR ASSUMED IN EN1993-1-5 ANNEX C.6

4. Non-linear finite element study

4.1 Calibration

Calibration of the non-linear modelling techniques proposed above was achieved by modelling plate girders used in physical tests by Rockey et al in 1981. This confirmed the accuracy of the approach and will be discussed more in the conference presentation.

4.2 Symmetrical steel girder

The layout used, shown in Figure 6, produces a high ratio of bending to shear force. This led to the provision of thick flanges to prevent premature failure in flexure, which in turn gave rise to large boundary rotational restraint to the web panel longitudinal edges. Adjustment of the bending/shear ratio was conducted by applying moments at the beam ends but. Most cases investigated were carried out at high shear stress and relatively low flexural stress.

Eleven different beams and/or load cases were considered and for each case, stiffeners were checked according to EN 1993-1-5. The girder ultimate load was generally well in excess of that predicted on the basis of stiffener failure to EN 1993-1-5. Only one representative case (Case 2-1) is discussed in detail.

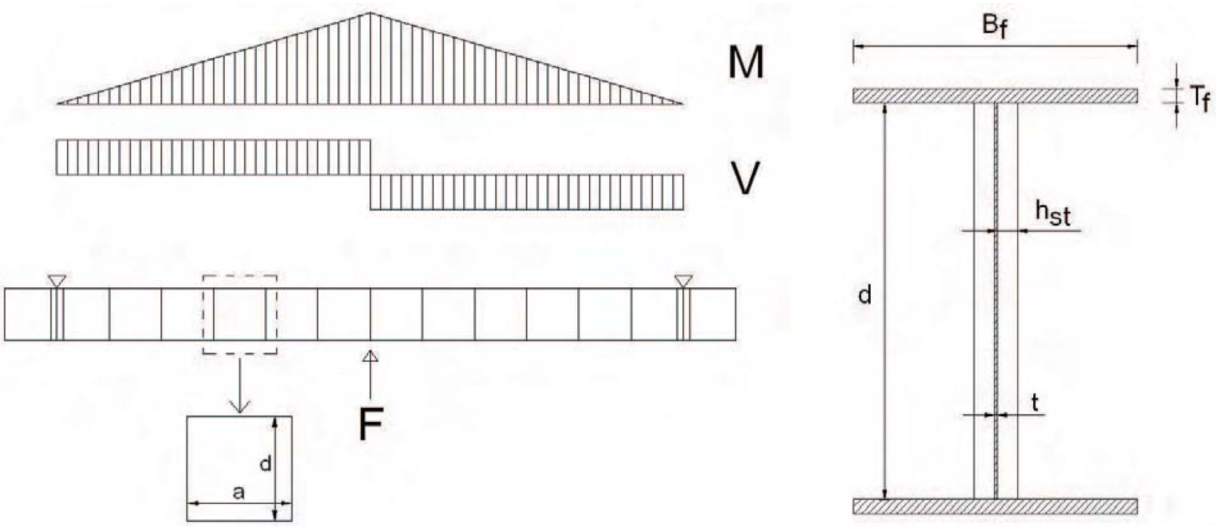

FIGURE 6. SYMMETRICAL STEEL BEAM SECTION AND LOADING

STRUCTURES

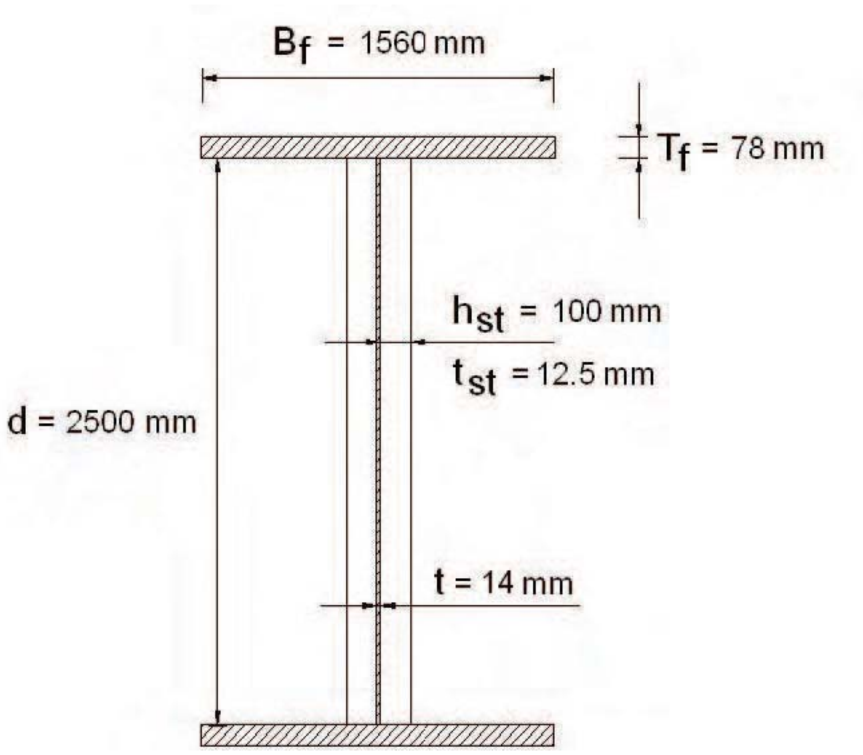

FIGURE 7. SECTION DIMENSIONS

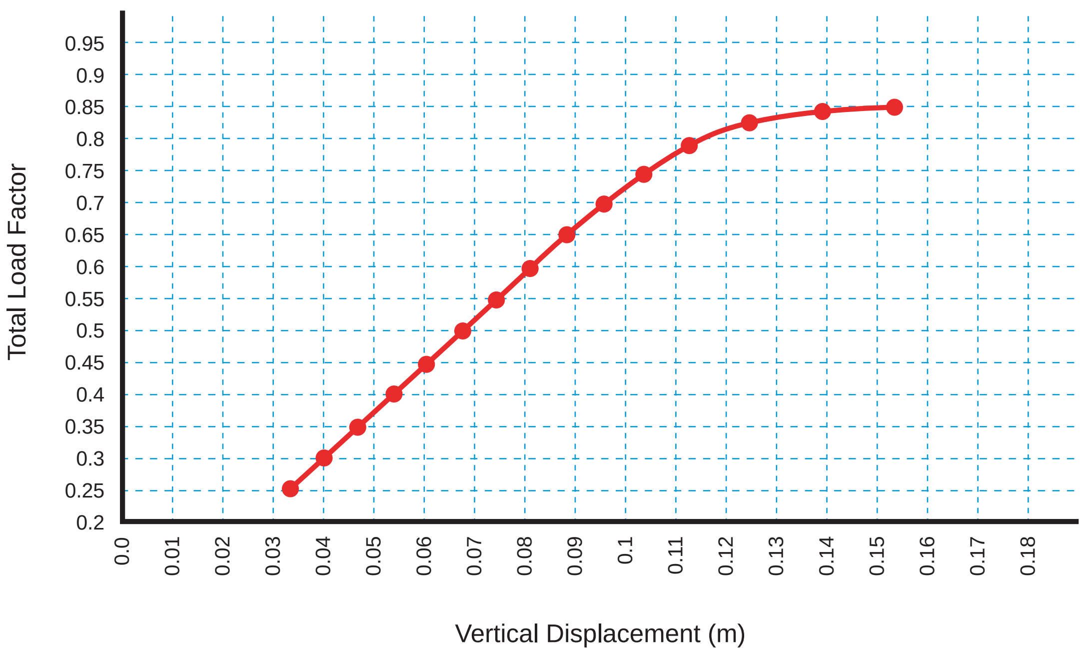

FIGURE 8. VERTICAL DISPLACEMENT VERSUS TOTAL LOAD FACTOR

Case 2-1

The stiffener dimensions were the minimum allowed by the stiffness criterion in EN 1993-1-5.

The panel aspect ratio was a/d = 1 and a central point load was applied without end moments. The elastic critical shear stress and force are

τ_{cr} = 55.6 N/mm2

and

$V_{cr\Delta}$= 1946 KN

The load-deflection curve obtained from the finite element analysis is illustrated in Figure 8. The analysis showed an almost linear behaviour up to a load factor of approximately 0.7, after which it showed a gradual loss of stiffness culminating in a failure at a load factor of 0.85.

STRUCTURES

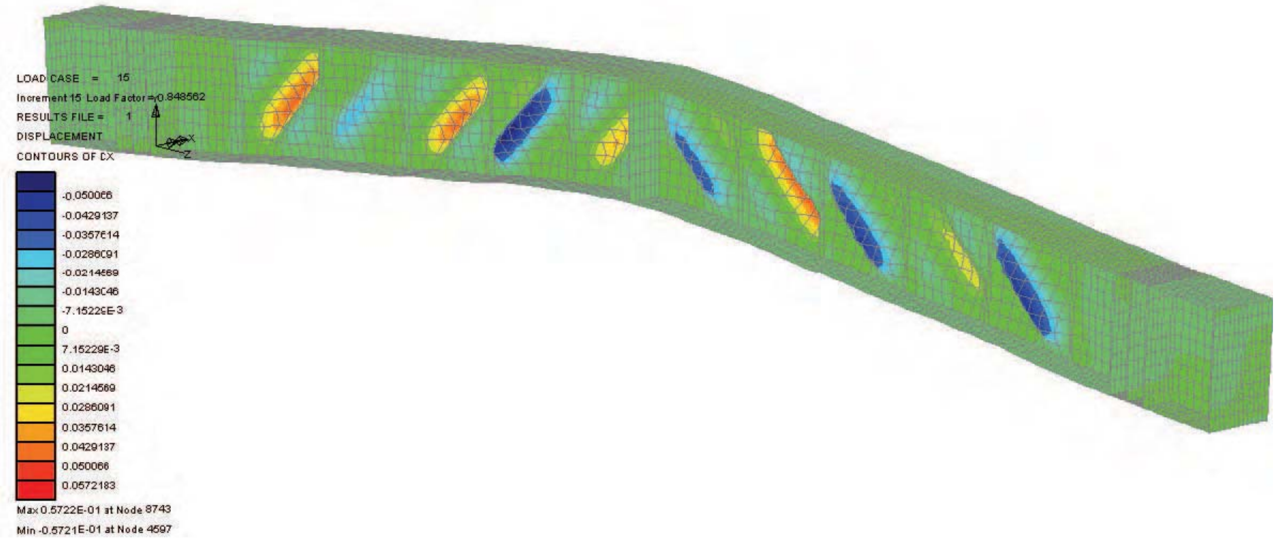

FIGURE 9. LATERAL DISPLACEMENT CONTOUR AT FAILURE (M)

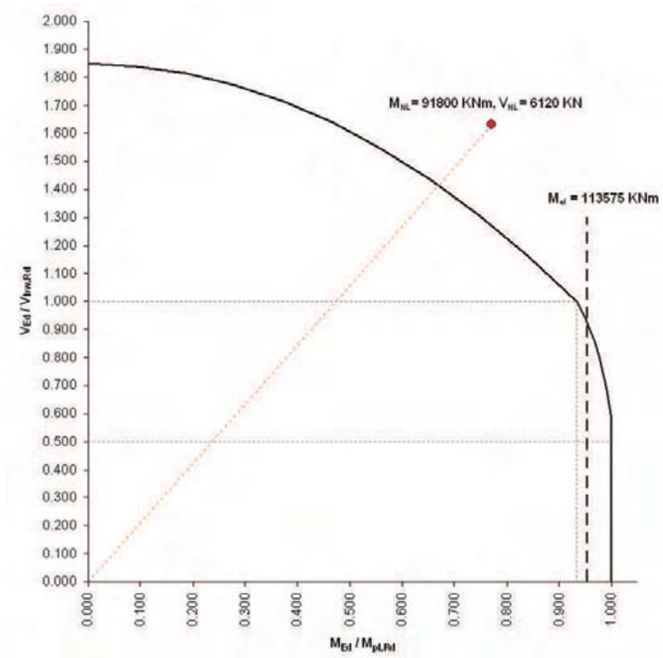

FIGURE 10. EUROCODE M-V INTERACTION DOMAIN AND RESULT FROM NON-LINEAR ANALYSIS (SIMPLY SUPPORTED PLATES)

The lateral deflections of the web at failure is illustrated in Figure 9, where it can be seen that the girder failed by the web bowing out laterally and yielding, while the stiffeners twisted in sympathy but did not themselves cause the ultimate failure.

The M-V interaction domain from EN 1993-1-5 and the results obtained from the non-linear analysis are illustrated in Figure 10. The girder showed an extra resistance of about +15% when compared with the Eurocode. The interaction curve was built according to EN 1993-1-5 clause 7.1(1) and doesn't account for the reduction of moment along the web panel. This domain was built considering a $_{cr}$ value derived for a simply supported plate loaded in shear. A closer match to EN 1993-1-5 is obtained if the rotational restraint offered by the flanges is considered when determining $_{cr}$ but some care is needed following this approach because the reduction factor curve itself makes some allowance for flange rotational restraint.

The location of the sections taken through the girder at various stages in the analysis to establish the distribution of internal forces are illustrated in Figures 14.

From Figure 12 it can be seen that for load increments 1 to 3, the longitudinal stresses in the web vary more or less linearly as expected from elastic beam theory.

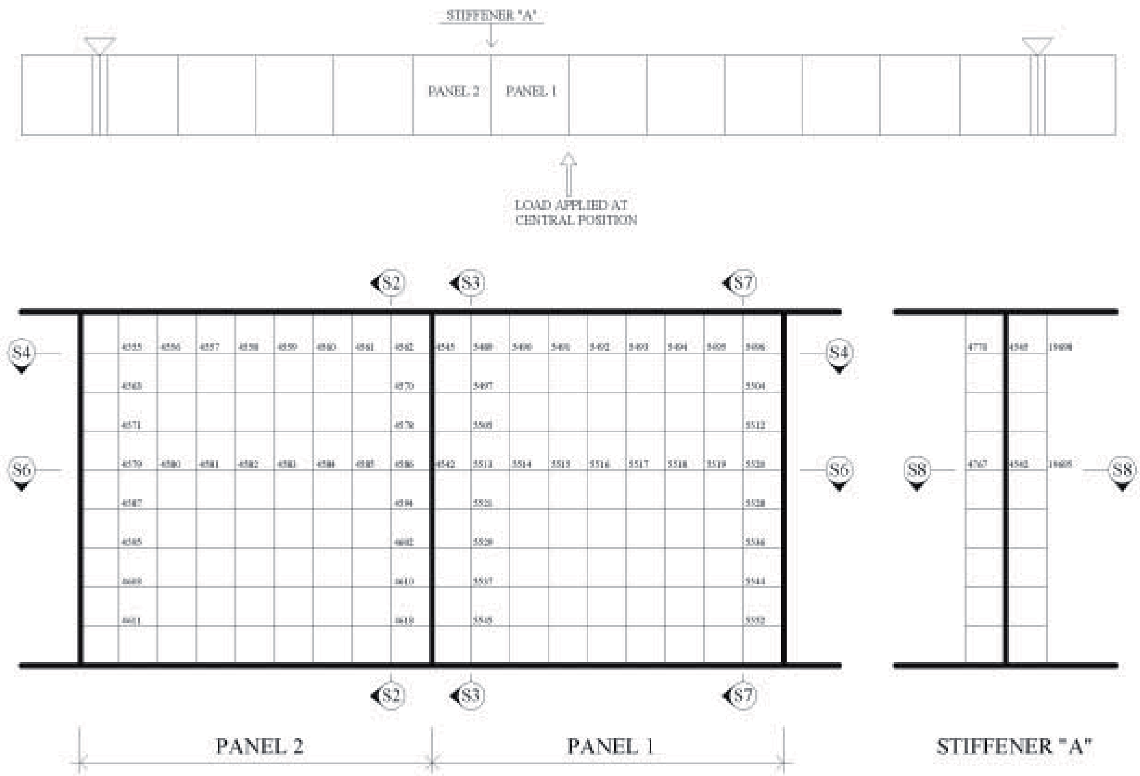

FIGURE 11. INVESTIGATED AREA AND LOCATION OF SECTIONS

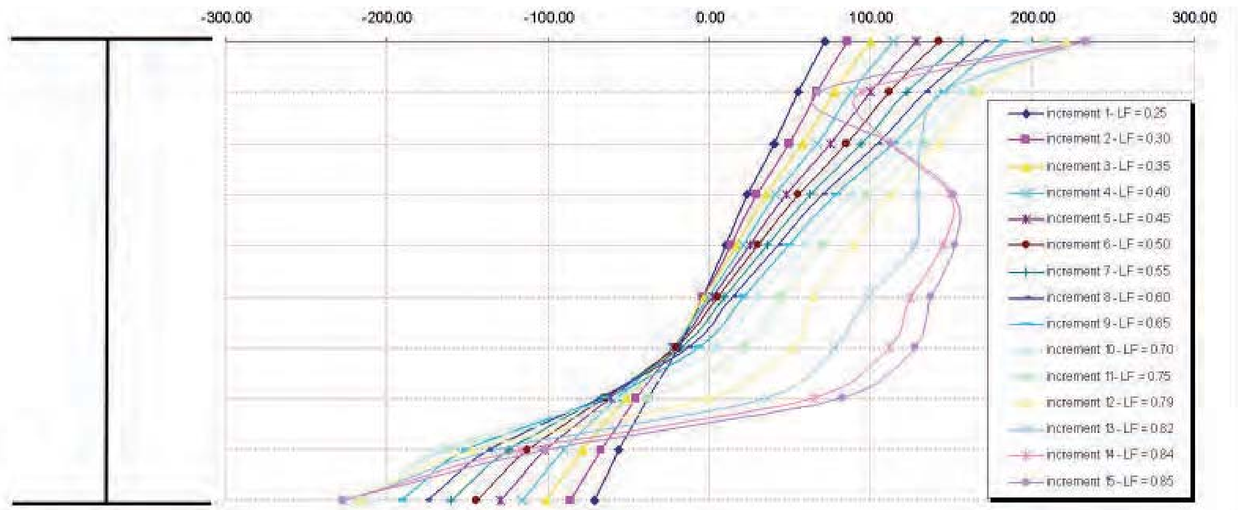

FIGURE 12. SECTION S3: LONGITUDINAL STRESSES IN THE WEB

Tension field effects appear beyond increment 3 at which the mean shear stress is about 72 N/mm^2, compared with an elastic critical buckling stress of 86 N/mm^2 (for fully clamped edges) and 55 N/mm^2 (for simply supported edges); it is therefore consistent with theory. Beyond this increment, a membrane tension develops which modifies the distribution of direct stress in the girder. This gives rise to a net tension in the web, balanced by compressive forces in the flanges which add to the flexural compressive stress in one flange and reduce the flexural stress in the tension flange as shown in Figure 13.

Web stresses either side of the stiffener are very similar, indicating that the web tension field stresses carry through the intermediate stiffener with little transfer of stress to it.

Figure 14 illustrates a simplified way how the total web stress is influenced by the membrane stress, assuming a parabolic distribution of membrane tension. Reference 10 provides a means of estimating the magnitude of this membrane stress.

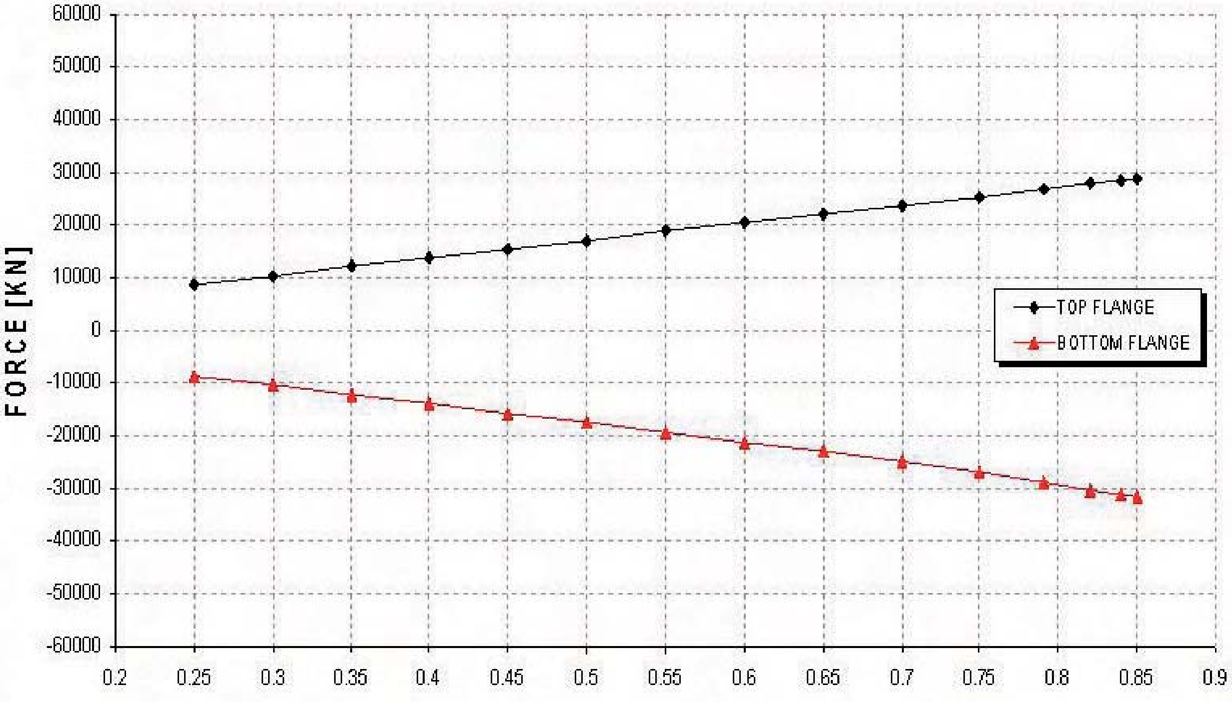

FIGURE 13. SECTION S3:
LONGITUDINAL FORCES IN THE FLANGES

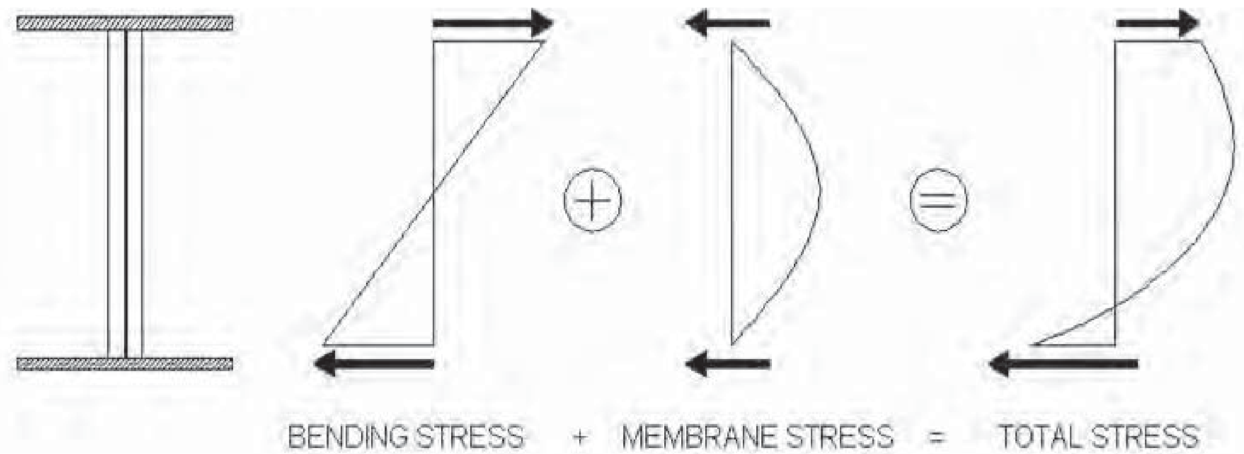

FIGURE 14. STRESS DISTRIBUTION UNDER BENDING AND SHEAR AT THE ULTIMATE LOAD AWAY FROM THE INTERNAL SUPPORT

Forces in the intermediate stiffeners

The stiffener forces given in Figure 15, at mid-height of the stiffener, are plotted against the load factor. These were obtained by integrating the vertical stress over either the stiffener outstands alone (which gave 428 kN) or the stiffener effective section defined in EN 1993-1-5 clause 9.1(2) (which gave 1030 kN). The figure illustrates how critical the choice of area is in calculating the force. There are clearly very significant flexural stresses also attracted to the stiffener as shown in the vertical stress profiles at mid-height across the stiffener in Figure 16. This implies that care is needed with the use of a force criterion because it may not be a very good predictor of overall stress level on its own. To check this, the stiffener effective section was considered as a pin-ended strut with axial force equal to 1030 kN as calculated above and with an initial imperfection as given in clause 9.2.1 of EN 1993-1-5. Second order effects were included in the calculation and the resulting stress distribution is shown as the hatched line on Figure 16. Whilst this gives a reasonable prediction of the actual stress variation across the stiffener, it underestimates the extreme fibre stress from the non-linear analysis. A slightly larger fictitious axial force would be required to recreate this peak stress and the proposal made below satisfies this.

STRUCTURES

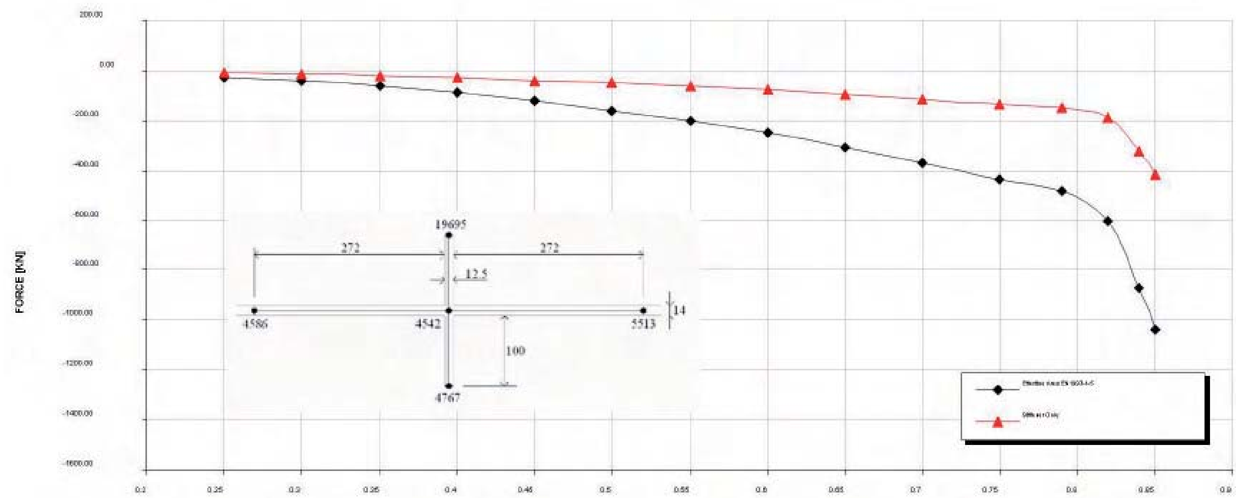

FIGURE 15. SECTION S8: VERTICAL FORCES IN THE STIFFENER (-VE COMPRESSION)

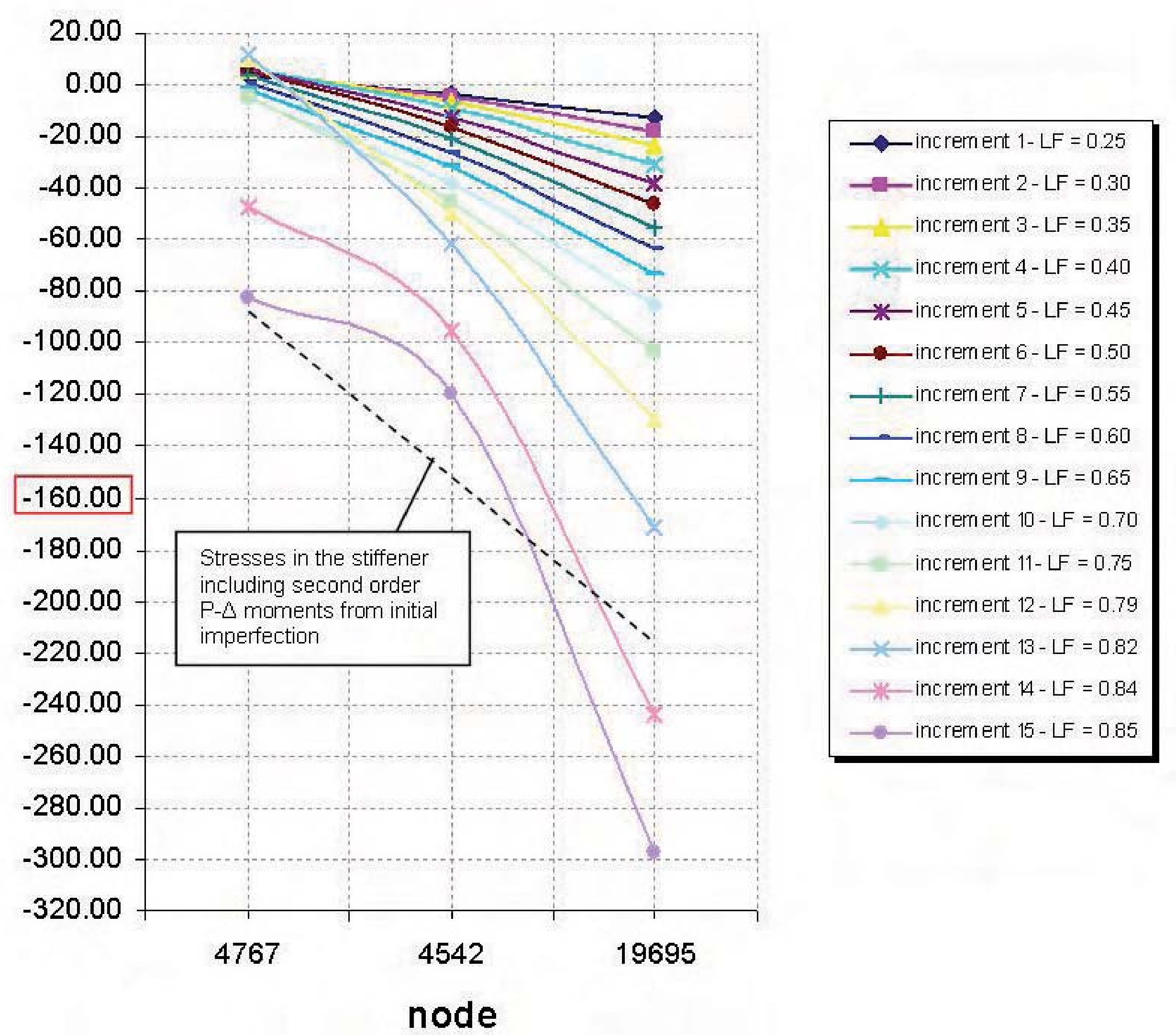

FIGURE 16. SECTION S8: VERTICAL STRESSES IN THE STIFFENER OUTSTAND (-VE COMPRESSION)

TABLE 1. FORCES IN STIFFENERS IN KN

Case	V_{ult}	V_{cr}	$V_{bw,Rd}$	Stiffener force according to:			
	FE Model	simply supported boundaries	EN1993-1-5	$V_{ult} - V_{cr}$	$V_{bf,Rd}$ [6]	$V_{ult} - V_{bw,Rd}$	FE Model
1-1	9009	4864	6306	4145	1665	2703	425
1-2	8811	4864	6306	3947	1665	2505	175
2-1	6120	1946	3750	4174	1545	2370	1030
2-2	6120	1946	3750	4174	1545	2370	1075
4	8613	4864	6306	3749	1048	2307	290
11	4653	4864	6306	0*	0**	0**	130

* Vult <Vcr

** Vult < Vbw,Rd

The stiffener force predicted by EN 1993-1-5, calculated as the difference between the observed ultimate shear force (6120 KN) and the elastic critical shear force (1946 KN), amounts to 4174 KN compared with a value derived from the observed stresses of about 1030 KN for the effective area at mid-height of the stiffener. If the above second order calculation is repeated for the stiffener using the force from the design standard, it leads to a predicted stiffener extreme fibre stress of over 1000 MPa which is far in excess of that predicted by the non-linear model (and indeed yield) and implies that the Rockey approach is very conservative.

Results for some of the other model cases are shown in Table 1. The stiffener forces derived from the non-linear analyses were compared to those predicted by EN 1993-1-5, which assume that the forces equate to the observed ultimate shear forces minus the critical shear forces, $V_{ult} - V_{cr}$.

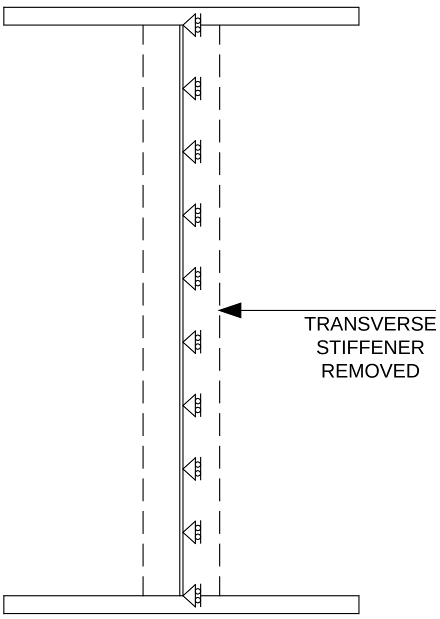

FIGURE 17. GIRDER SECTION AT STIFFENER LOCATION AND LINE SUPPORT ALONG WEB PLATE

Also shown in Table 1 are the predicted stiffener forces calculated in accordance with Höglund's[6] theory, which are those based solely on the final part of the shear resistance mobilised by the flange contribution $V_{bf,Rd}$. Also based on Höglund's[6] theory is a proposed design value of $V_{ult} - V_{bw,Rd}$ which allows for the fact that the shear carried in the non-linear analysis exceeded the codified shear resistance. For design in accordance with EN 1993-1-5, $V_{ult} - V_{bw,Rd}$ $V_{bf,Rd,}$ unlike in Table 1.

It can be seen that the approach used in EN 1993-1-5 gives high values for the forces in the stiffeners compared to the ones obtained from the non-linear analyses. In order to get a better correlation with the FE results, an alternative criterion is required. From the evidence that the tension field passes through the stiffeners after elastic buckling has occurred, it appears reasonable to base the stiffener forces on the difference between the applied shear force and the shear strength of the web $V_{bw,Rd}$. The force in the stiffener, applied in the plane of the web, can then be expressed as follows:

$$F = V_{Ed} - \quad V_{bw,Rd} \geq 0 \quad (3)$$

which contrasts with a value of $V_{Ed} - V_{cr}$ as implicit in the EN 1993-1-5 approach.

The enhancement factor was intended to allow for secondary compatibility bending stresses that develop in the stiffeners due to their function of keeping the panel straight along its boundaries. For this purpose, it would be expected that would be less than unity. However, all the results except those for case 11, indicate that is greater than unity and can conservatively be taken as unity. The reason for the greater than unity value of is likely to result from an underestimate of Vbw,Rd in EN 1993-1-5 and the beneficial effects of strain hardening. The lack of conservatism of the proposal for case 11 is mitigated by the additional stiffness criterion in EN 1993-1-5 which would not be met if a stiffener was designed for the very small force predicted by the FE model.

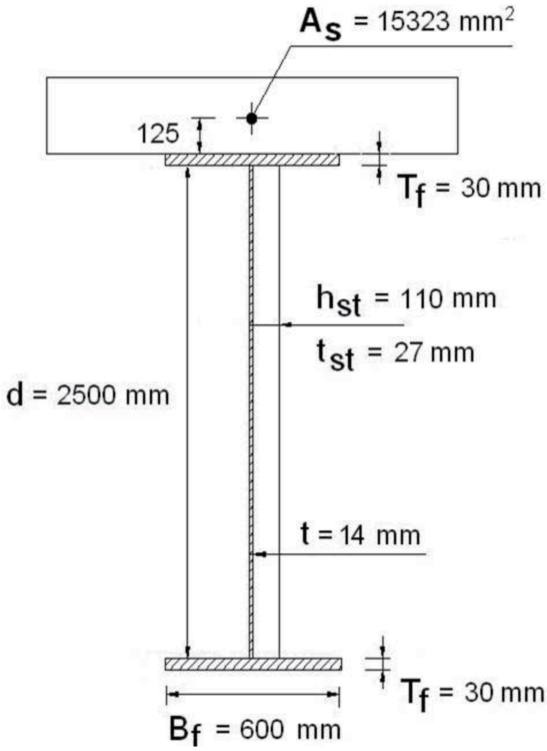

FIGURE 18. SECTION DIMENSIONS

Stiffener with no axial strength

An extreme situation was also investigated. The Case 2-1 girder arrangement was modified, and "Stiffener A" in Figure 11 was removed and replaced with a line support vertically down the web plate at its location, which prevents out of plane movement along the line but allows vertical movement, as shown in Figure 17.

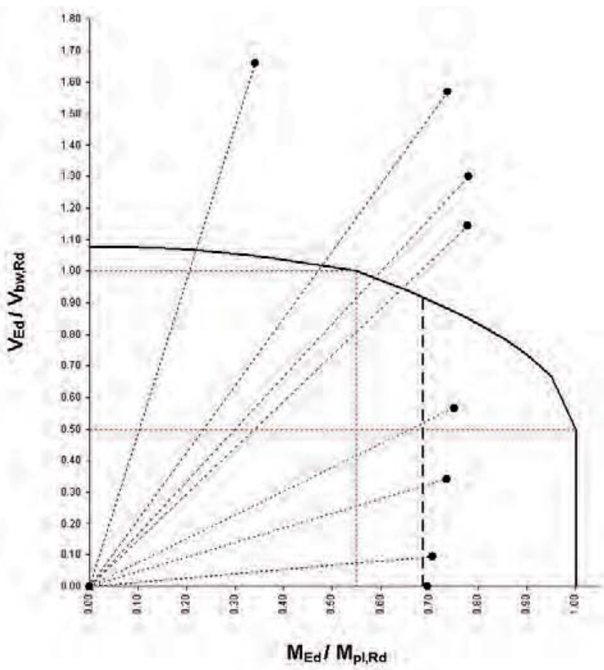

FIGURE 19. EUROCODE M-V INTERACTION DOMAIN AND RESULT FROM NON-LINEAR ANALYSES

This is effectively a transverse stiffener with no capacity to carry axial force but with a rigid out of plane bending stiffness.

It is interesting to note that the non-linear analysis stops when it fails to find equilibrium beyond a load factor 0.88, which is slightly larger than the load factor obtained when a transverse stiffener with the minimum EN 1993-1-5 stiffness was provided. The load factor in that case was 0.85 and the failure was by panel yield rather than by stiffener buckling. This means that the girder still achieves a shear of $V_{bw,\,Rd}$ to EN 1993-1-5 even if the stiffener cannot carry any axial force in a truss mechanism and that out of plane bending stiffness is more important than axial resistance. The mechanism predicted by Rockey does not actually occur and the development of stiffener axial force in a truss behaviour is not necessary for loading beyond V_{cr}.

Steel-concrete composite girder

A typical cross-section from an existing UK bridge was considered (see Figure 18). The concrete was assumed to be cracked in flexure and hence only the reinforcement was modelled. The girder's shear-moment behaviour and the resulting stiffener forces were examined, together with the effect of varying the strength and stiffness of the stiffeners. Stiffeners were again designed to comply with the EN 1993-1-5 stiffness criterion, but not the force criterion. Despite this, in no case did the stiffener cause the ultimate failure.

To investigate different ratios of bending moment to shear force, ends moments were applied. In particular, to obtain low ratios of M/V at mid-span, the end moments were generally bigger than those at mid-span and so to avoid a premature failure at beam ends, the girder was "strengthened" in the model there.

Several cases were studied for shear-moment resistance and a graphical summary is shown in Figure 19 for the cases with no imposed axial force. The results are there compared with the predictions of EN 1993-1-5 where, once again, the effect of moment reduction along the panel was ignored. Additional cases were also considered to test the effect of external axial force in the girders, but these are not considered further here.

Stress distributions in the girder web and stiffeners were determined in the same way as for the bare steel beams. They are not shown here but are similar to those for the bare steel beams.

Behaviour

As discussed for the bare steel girder, in all cases there was a development of a web membrane tension, which passed through the intermediate stiffener inducing much smaller forces in the stiffeners than assumed in EN 1993-1-5 and BS 5400:Part 3. There was also a reduction in tension flange force and increase in compression flange force as discussed above and illustrated in Figure 14.

TABLE 2. FORCES IN STIFFENERS IN KN

Case	V_{ult}	V_{cr}	$V_{bw,Rd}$	Stiffener Force			
	FE Model	simply supported boundaries	EN1993-1-5	$V_{ult} - V_{cr}$	$V_{bf,Rd}$ [6]	$V_{ult} - V_{bw,Rd}$	FE Model
1	5950	1925	3750	4025	65	2200	430
3	2200	1925	3750	275	0	0**	95
6	6289	1925	3750	4364	250	2539	965

Forces in the intermediate stiffeners

As for the symmetrical steel girder cases, the forces derived from the non-linear analyses results were compared to those predicted by BS 5400:Part 3 and EN 1993-1-5 as shown in Table 2.

Once again, it can be seen that the approach used in EN 1993-1-5 gives high values for the forces in the stiffeners compared to the ones obtained from the non-linear analyses. It can also be noted that the proposed approach of basing the stiffener forces on the difference between the applied shear force and the shear strength of the web as above leads to smaller forces in the stiffeners and a better agreement with the non-linear analysis.

M-V interaction

Figure 19 shows the interaction curve for bending and shear according to EN 1993-1-5 and the results from the non-linear analyses for different M-V ratios.
It is evident that the rules are conservative for both bending and shear. For low shear, the resistance to bending moment is close to the EN 1993-1-5 prediction.
It is interesting to note that the bending resistance increases slightly when a small shear force is added. Similar results were obtained in other studies[11].
The increase can be attributed to the moment gradient applied. In girders with low shear the moment gradient is small and this leads to a longer plastic zone than in a beam with a steeper moment gradient. For low bending moment, the resistance to shear is much higher than predicted. This could be partially attributed to boundary rotational restraints of the panel not considered in the construction of the interaction domain to EN 1993-1-5. The interaction is in any case very weak except at very high shear.

5. Conclusions

The finite element modelling for both steel and composite girders showed that in no case was the overall failure due to local stiffener failure, as long as the stiffener's stiffness was in accordance with the minimum required by EN 1993-1-5 and its yield strength was the same or greater than that of the web panel. Failures were located in the web panel and the Eurocode prediction for overall girder strength was always safe

The axial stress, considered in some of the analyses, had an influence on the final load bearing resistance of the girder as one would expect, but had limited effect on the stiffener forces

A revised proposal for the design force, $F_{Ed,}$ in a stiffener has been made in order to get a better correlation with the finite element results: $F_{Ed} = V_{Ed} - V_{bw,Rd} \geq 0$;

The effects of different M-V ratios were investigated and compared with the moment-shear interaction diagram predicted by EN 1993-1-5. The results of EN1993-1-5 were found to be conservative

Girder behaviour under shear and moment is well described by Höglund's theory

Girder resistances were relatively insensitive to initial imperfections in the stiffeners and web panels

Evidence was produced for the adequacy of designing the stiffeners for a stiffness-only criterion, but this requires further investigation before such a proposal could be made

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Wind-induced dynamic failure of a slender sculpture 018

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Abstract

The Mersey Wave sculpture consists of twelve 30m tall masts located on either side of Speke Boulevard in Liverpool (Figure 1). The sculpture was opened on 15 December 2003. The masts were taken down approximately one month later, following the observation of large deflections of the masts under moderate wind conditions and the appearance that at least one of the masts had been overloaded

The sculpture was constructed under a design and build contract commissioned by Liverpool Land Development Company. Following the masts being taken down, Atkins was employed as technical advisors to Liverpool Land Development Company. This paper describes the work subsequently carried out to understand the behaviour of the masts and to identify a suitable remedy.

1. Observed behaviour

In its initial form, the sculpture consisted of twelve relatively tall slender masts canted over at angles to the vertical varying up to 20 degrees. Each mast was constructed from welded steel circular hollow sections with step changes in diameter to accommodate the external cladding which had a prismatic (triangular) cross section, tapering as it rose in height. As shown in Figure 2, six masts were located on either side of the boulevard, orientated in opposite directions on each side, such that an apex of the triangular cross-section pointed away from the boulevard.

Following construction the masts were observed to be oscillating with a high amplitude in moderate wind conditions. A video of the behaviour taken on 8 January 2004 showed the masts on the north side of the road oscillating far more than those on the south side. From the movement of the clouds in the video, the wind appeared to be blowing in a direction across the road from the South East to the North West direction (Figure 2). The maximum deflection of the masts was of the order of ±1.5m across the direction of the mean wind and the frequency of oscillation was approximately 0.7Hz. At least one of the masts appeared to have been overloaded as permanent deformation could be observed.

Wind data subsequently obtained for nearby Liverpool airport for 8 January 2004 indicated that the mean wind speed at the time the video was taken was approximately 12m/s at a bearing of 160° from North. This corresponds to the wind blowing across the line of the road from the South East, in agreement with the observations.

From the observations it was apparent that the structures were not performing as intended, were causing concern to passing motorists, and the sculpture was dismantled. An assessment of the mast performance was instigated, with a view to re-instating the sculpture as soon as possible, incorporating any redesign required.

2. Dynamic instability

It was clear from the observed behaviour that the masts were exhibiting an extreme dynamic response to the imposed wind load. A wind speed of 12m/s corresponds only to a stiff breeze, which would normally not be strong enough to generate such an extreme deflection. In addition, the observed deflection was across the direction of the wind, so it was clearly not generated by a direct wind load.

It is unusual for tall slender structures of this nature to be triangular in cross-section, they are more usually circular. While very little information on the dynamic wind loading on structures with a triangular cross-section could be found in the literature, a significant

FIGURE 1. MERSEY WAVE - ORIGINAL CONFIGURATION

proportion of the papers gave evidence of the inherent susceptibility of tall flexible structures to vortex-induced vibrations, and to a lesser extent galloping.

To identify the cause of the extreme response in this case, it is necessary to consider the typical characteristics of oscillations induced by vortex shedding and by galloping. The different mechanisms are illustrated in Figure 3.

Vibration due to vortex shedding arises when the vortices shed from first one side of a structure and then the other, generates oscillating pressures on the structure which cause it to move. The principal danger from this type of loading arises from the possibility of resonance between a natural frequency of the structure and the frequency of vortex shedding whereby large amplitudes of oscillation can be caused.

The characteristics of vortex-induced vibration include the following:

- It can occur for any cross-sectional shape
- The oscillation is transverse to direction of the wind
- The theoretical maximum deflection is of the order of 1/4 – 1/2 the width of the structure but is usually lower than this.
- Structures with a tapering cross section are less prone to vortex-induced vibration because the critical wind speed is proportional to structure width.

By comparison, galloping can occur when movement of a structure causes a change in angle of attack of the wind which generates a force causing further movement in the same direction. This positive feedback mechanism is aerodynamically unstable and very large amplitudes of vibration, i.e. galloping, can result. The characteristics of oscillations due to galloping include the following:

- It can only occur for non-circular cross sections
- The oscillation is transverse to the direction of the wind
- Extremely large deflections can occur
- Structures with a tapering cross section are still prone to galloping.

Given the large amplitude of the observed oscillation and the tapering shape of the members it was considered to be unlikely that the excessive motion observed could be attributed to vortex shedding. Instead, the most likely cause of the large oscillations was considered to be galloping.

3. Physical models

In order to investigate such effects, a number of simple physical models were constructed in order to explore the dynamic performance of structures with different shaped cross-sections. Simple flexibly mounted columns of circular and triangular cross-sectional profile were placed in front of a fan (Figure 4).

It was immediately obvious that while a column with a circular cross section moved a small amount in the lateral direction under the wind load, replacing the circular cross section with a triangular or semi-circular section resulted in much larger oscillations across the direction of the wind. This demonstrated the different effects of vortex-induced motion for the circular section and galloping for the triangular section.

The models clearly illustrated the importance of galloping for tall, slender structures of triangular cross-section.

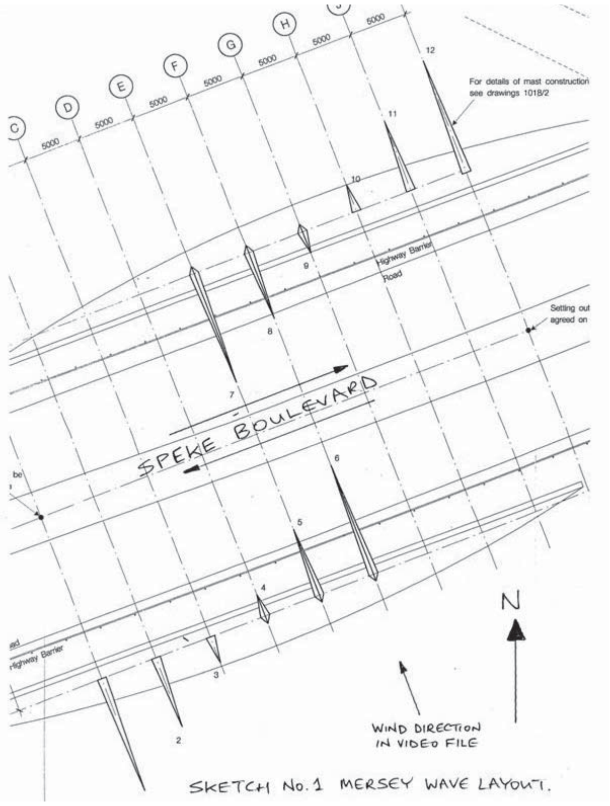

FIGURE 2. MAST LOCATION

FIGURE 4. TRIANGULAR PROILE PHYSICAL MODEL

4. Identifying a solution

In order for the sculpture to be reinstated, a redesign of the mast was proposed by the contractor, taking into account the aesthetic purpose of the structure in addition to the structural integrity requirements. To avoid galloping, several options were theoretically available.

First, the section could be stiffened to increase the natural frequency. This idea was discarded as it was not feasible to significantly stiffen the mast while retaining the mast shape and sculptural integrity. Secondly, dampers could be incorporated into the design to damp out the galloping motion. However, this was not practical as there was insufficient space within each mast to insert dampers. Finally, the section profile could be modified to prevent galloping. This option was chosen and several different profiles were tested by the contractors in a wind tunnel.

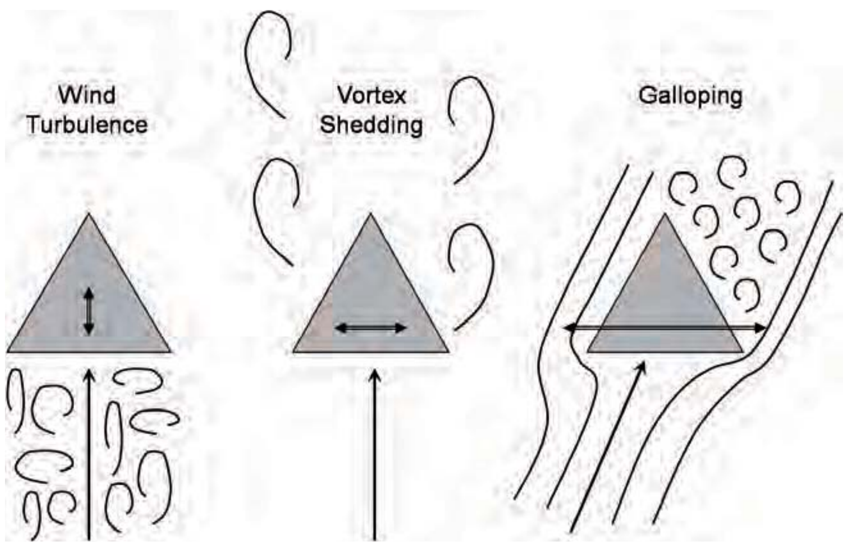

FIGURE 3. DIFFERENT TYPES OF WIND LOADING

STRUCTURES

The solution chosen was to increase the porosity of the mast. This solution was tested by the Contractors in the wind tunnel and was shown to work. Independent tests were conducted using the simple physical model with a porous profile and the galloping was shown to have been eliminated. The final redesign involved replacing the top two thirds of each mast with an open lattice structure.

From the point of view of the re-construction of the sculpture it was important to identify which parts of the original masts could be re-used. To this end, a fatigue analysis was performed on the original design to see what proportion of the fatigue lives of the individual components had been used up.

5. Analysis of structural implications of galloping

A global finite element model of a 20° angled mast was constructed to analyse the natural frequencies and deflections of the mast, with a second, more detailed model used to look at local stresses at the joints between the tubular sections. The models were created using the ABAQUS finite element package.

The models had to take into account the structural design of the mast, in particular the construction of the central core, which was constructed from welded steel tubes with step changes in diameter to accommodate the tapering external shape. The tube sections were designed to overlap by between 350mm and 200mm with the non-concentric connection between the differing diameter tubes constructed from stiff plates in a cruciform layout, except at the top joint where a welded flange connection was used. The design of the joints was such that there was no discontinuity in rotation at that location. The cladding attached to the central core was represented in the model by added mass as it was deemed to have little structural value.

The model was validated against the observed static and dynamic behaviour of the mast, and the effects of galloping were assessed by considering the maximum deflection observed and the corresponding stresses. From this, the fatigue life of the mast components could be assessed, and the results were compared against the observed failures in the mast structure.

6. Natural frequency analysis

The frequencies of the first 6 modes of the mast were calculated using the natural frequency extraction procedure within ABAQUS. The first frequency of the mast was calculated to be 0.67Hz, which is close to the observed frequency of oscillation of approximately 0.7Hz estimated from video of the mast in motion. The frequencies of the first 6 modes of the mast are given in Table 1. The in-line direction is vibration in the same plane as the direction of tilt and lateral is perpendicular to the direction of tilt. The modes in the two directions are at similar frequencies because the frequency is controlled by the pipe sections rather than the joints or the base, which are the only asymmetrical features on the mast.

TABLE 1. NATURAL FREQUENCIES

Mode	Frequency (Hz)
1st lateral	0.67
1st in-line	0.67
2nd lateral	1.47
2nd in-line	1.47
3rd lateral	2.83
3rd in-line	2.84

The mode of vibration observed during the galloping episode appears to be made up entirely of the first lateral mode of vibration, as illustrated in Figure 5.

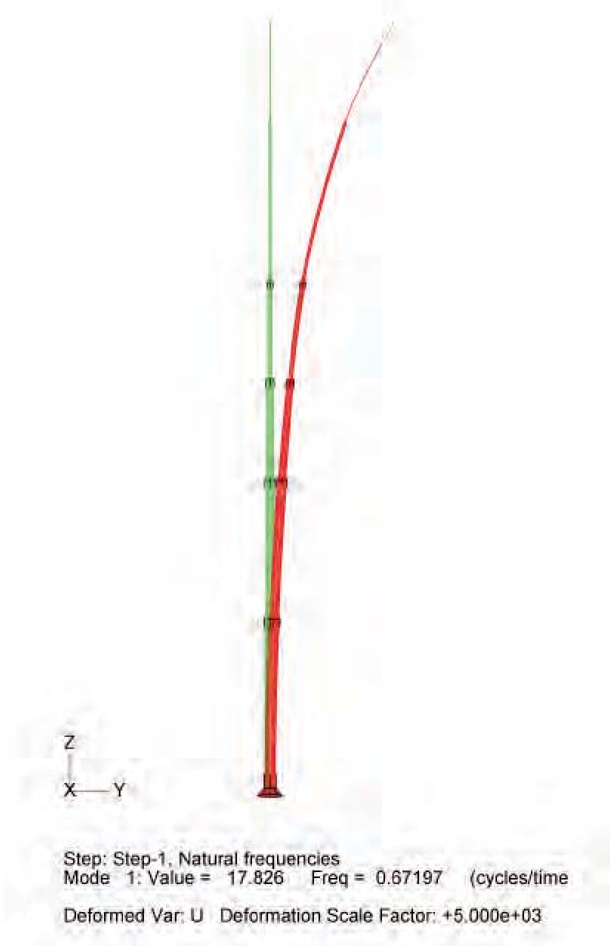

FIGURE 5. FIRST LATERAL MODE OF VIBRATION (0.67HZ)

STRUCTURES

7. Fatigue analysis

The estimated deflection amplitude observed during the galloping episode was 1.5m. In order to estimate the corresponding stresses experienced by the structure this deflection was applied to the finite element model by factoring the first mode of vibration of the structure to match the tip deflections. The stresses were extracted at locations adjacent to the weld locations, since these locations are most susceptible to fatigue.

High stresses were predicted in all of the cruciform joints and at the mast base, and were above the yield stress for the material in the top two joints in the structural core (Figure 6). The stress distribution confirmed that under deformation in the lateral direction only two of the cruciform arms were effectively acting at each joint, leading to high stresses in the joint.

It was clear from the model results that, had the structure been allowed to continue oscillating at ±1.5m, fatigue failure at the joints and base would have been expected to occur after a matter of days. Given the high stresses throughout the structural joints and the short fatigue lives predicted, it was suggested that significant fatigue damage could have already occurred throughout the structure while in service, prior to it being dismantled.

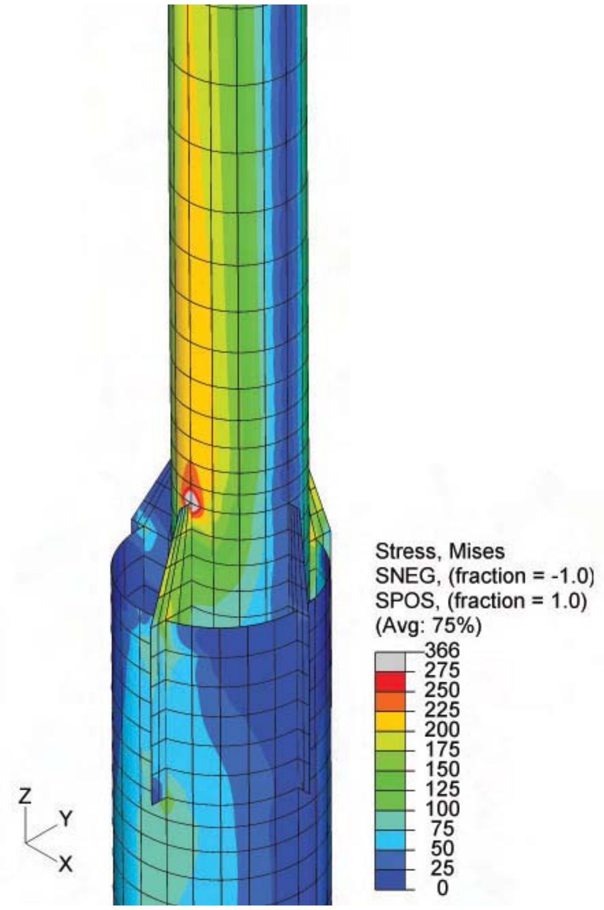

FIGURE 6. STRESSES IN CRUCIFORM AT 20.5M UNDER GALLOPING LOAD

The permanent deformation as a result of overstressing could be seen in the masts after they were taken down. Photographs of one of the masts show a permanent deformation at the top connection location (Figure 7) and also that the connection between the stainless steel cladding and the triangular plate to which it is screwed had partially failed due to being overloaded. The evidence also suggested that the cladding was supporting the connection at this location when it was highly loaded.

FIGURE 7. PERMANENT DEFORMATION IN MAST DUE TO GALLOPING

A photograph of the failed section of cladding shows that the screws holding the cladding had ripped out of the stainless steel skin, indicating that it was carrying significant load during the galloping episode.

8. Implications for revised design

The fatigue analysis suggested that the significant fatigue damage had already occurred at the joints due to the high loads. As a result the joints and base were removed and redesigned by the contractors. The top two thirds of the mast were replaced with an open lattice structure, with the long lattice members filled with sand at the contractor's suggestion to prevent vortex-induced vibration of these members. However, it was possible to re-use the bottom two tubular sections, albeit with re-designed joint details, as the analysis showed that these sections were largely unaffected by the galloping load.

The sculpture was successfully reinstated in June 2005, as shown in Figure 8. No further problems have been reported to date.

FIGURE 8. THE RE-DESIGNED MERSEY WAVE

9. Conclusions

This paper has described the process undertaken by Atkins in the technical support of Liverpool Land Development Company following the necessary dismantling of the Mersey Wave sculpture after damage due to galloping. The results of the analysis undertaken clearly indicate that in a structure prone to galloping, the effects can be catastrophic. The Mersey Wave was an unusual structure in that the masts were tall and slender with a triangular cross-section. As structural design pushes the boundaries of convention, and structures in general tend to become lighter and more slender, it is essential to consider dynamic behaviour at the design stage and to design out any features which may give rise to excessive dynamic response.

The role of aggregate in reinforced concrete exposed to chloride environment

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Abstract

Corrosion of reinforced concrete is generally considered to be controlled primarily by the properties of the paste fraction, with aggregate particles considered to be inert inclusions. Indeed, the key Code of Practice for structural concrete make limited reference to the characteristics of aggregates that may affect chloride ingress. However, a theoretical analysis by Hobbs has challenged this view and has demonstrated that if the aggregate had an interconnected porosity, then it could be up to 1000 times more permeable than the surrounding paste. Furthermore, the interfacial zone between the aggregate and paste is also an important influence over the movement of chloride through concrete.

This paper reports a laboratory-based experimental programme that studied the performance of concrete containing different sources of aggregate with a wide range of water absorption. The result of the experimental programme clearly indicates that aggregate interconnected porosity has significant effect on corrosion activity. This has important implications for concrete specifiers in geographical areas where high porosity aggregate rock types are encountered. The current specifications (BS 8500 or BS EN 206) approach of minimising chloride ingress into structural concrete by limitation on the water/cement ratio is in isolation, therefore inadequate to ensure the long term durability of structures exposed to chloride environments. Preliminary guidance has been provided for identifying those aggregates likely to require additional mix limitations, or design consideration.

1. Introduction

One of the major problems concerning durability of reinforced concrete structures is corrosion of steel reinforcement, induced by the presence of chloride ions. For this reason, the mechanism of chloride ingress into concrete and the consequent corrosion of embedded reinforcement have received a great deal of attention from the research community. The majority of these studies generally assumed that ingress of external chloride ions into concrete is dependent only on the quality of the cement paste fraction of concrete, with the effect of aggregate on chloride ingress often being ignored. This assumption is probably correct, providing the aggregate is dense and has no interconnected porosity.

A study by Power et al[1] has shown that the water permeability of aggregate can vary widely, and that the water permeability of a mature paste with w/c ratio of 0.48 can be between 10 to 0.001 times that of an aggregate likely to be used in concrete. Given that the aggregate could form 65% to 75% by volume of the concrete, and that a wide range of aggregate qualities are increasingly used to utilised to achieve sustainability, variation in aggregate permeability could potentially have a major influence on the ingress of chloride ions into concrete[2].

To date, little qualitative data exists on the influence of aggregate on chloride ingress. The permeation and diffusion properties of aggregates appears to have been largely ignored in the materials selection and specification process, with current concrete specification based on the performance of aggregates having low absorption. The introduction of BS 8500[3] and BS EN 206[4] for concrete, does not take account of possible effect of aggregate. The assumption that these concrete specifications are valid for aggregates having higher absorption may not be correct. Hence, with a wide range of aggregate commonly used in concrete, it is possible that the use of some aggregates will be detrimental to the intended performance and working life of reinforced concrete structures. This research work is completed in 2004.

TABLE 1. PROPERTIES OF COARSE AGGREGATE (20-10MM FRACTION)

Aggregate Type	A	B	C	D	E	F
Shape	Angular, Irregular	Angular	Rounded	Rounded	Partially Crushed, rounded	Elongated, Flaky
Particle Density (kg/m³ SSD)	2630	2680	2630	2600	2550	2560
Absorption (% Dry Mass)	0.5	0.6	0.9	1.2	3.1	3.2

TABLE 2. PROPERTIES OF FINE AGREGATE (UNDER 5MM)

Sand Type*	B	C	D	E	F
Shape	Angular	Rounded	Rounded	Partially Crushed, rounded	Elongated, Flaky
Particle Density (kg/m³ SSD)	2665	2660	2650	2590	2560
Absorption (% Dry Mass)	0.3	0.6	1.1	1.8	2.0

* No fine aggregate with rock type A and concrete was produced with fine aggregate D

2. Materials and mix proportions

Cement

The cement used was a Portland cement conforming to CEM I 42.5 of BS EN 197-1[5]. This was used both alone and in combination with 30% PFA to BS 3892-1[6] (IIB-V) or 50% GGBS to BS 6699[7] (IIIA) by mass.

Aggregates

Six normal weight coarse aggregates were used in the study. All aggregates conformed to BS 882[8] or BS EN 12620[9]. The characteristics of these aggregates are provided in Tables 1 and 2.

All aggregates were obtained in single bulk supplies from quarries throughout the UK, in 5 to 10mm, 10 to 20mm, and 20 to 40mm fractions. Water absorption and porosity were generally being lowest for crushed AGG A while AGG F had the highest absorption. The mechanical properties followed similar trends of water absorption.

All coarse aggregates were combined with fine aggregates obtained from the same source as the coarse aggregate, except AGG A, where SAND D was used as fine aggregate, since fine AGG A was not available.

Plasticising chemical admixture

A standard superplasticizer, conforming to BS EN 934-2[10], was included, where required, in the concrete mixes to maintain workability in the range of 75-100mm slump without altering the free water content. It was a dark brown liquid, having specific gravity of 1.32 at 20°C and zero chloride content.

Carbon steel reinforcement

High-yield, hot rolled type 2, 10mm diameter reinforcing steel bars, conforming to BS 4449[11] were used in the study.

Test mix proportions

Concrete mixes containing CEM I, with each of the six aggregate types was produced at water/cement ratios of 0.40, 0.50 and 0.60, as shown in Table 3. For concrete containing combinations IIB-V and IIIA, only one water/cement ratio (0.50) was investigated, but all six aggregates were included in the experimental programme. These mixes were produced to the same target slump as CEM I mixes.

Aggregate preparation and pre-mixing saturation

All aggregates were soaked for 24 to 48 hours prior to use. After soaking, the aggregates were spread on dry Hessian to remove the access water, being spread at one layer thickness and left for 4 to 6 hours until the surface water on the aggregate evaporated. The aggregates were then stored in airtight tubs, and used within 3 to 5 days. The procedure was followed to minimise the effects of air release from partially dry aggregate prior to setting in the cement paste at the surface of aggregate particles.

Curing method

The specimens were initially cured for 24 hours under damp hessian and polythene sheeting to maintain high humidity. The specimens were then demoulded and subsequently stored under sealed curing (20°C, Relative Humidity >95%). All samples were wrapped in 3 layers of cling film after demoulding.

TABLE 3. CONCRETE MIX PROPORTIONS AND CONCRETE PROPERTIES

Cement Type	AGG Type	W/C Ratio	Cement (kg/m$^{3)}$	Coarse AGG (kg/m^3)	Fine AGG (KG/m^3	Sp* (% Cem)	Plastic Density (kg/m^3)	28d Cube Strength (N/mm^2)
CEM I	A	0.40	425	1175	630	0.15	2400	63.5
	B			1195	640	0.16	2430	64.5
	C			1175	630	0.13	2400	58.0
	D			1160	625	0.15	2380	58.0
	E			1140	615	0.13	2350	45.5
	F			1155	620	0.12	2370	48.0
	A	0.50	340	1200	690	0.10	2400	54.0
	B			1210	700	0.13	2420	49.0
	C			1200	690	0.08	2400	42.0
	D			1185	680	0.12	2375	43.0
	E			1170	670	0.13	2340	39.0
	F			1180	675	0.10	2365	38.0
	A	0.60	285	1160	790	0.09	2405	43.0
	B			1175	800	0.08	2430	41.0
	C			1140	790	0.09	2385	35.5
	D			1145	780	0.09	2380	35.5
	E			1125	770	0.08	2370	29.0
	F			1140	780	0.09	2375	30.5
IIB-V	A	0.50	340	1210	675	0.12	2425	41.0
	B			1215	685	0.10	2410	43.0
	C			1200	675	0.11	2385	38.0
	D			1190	670	0.09	2370	36.0
	E			1170	660	0.08	2340	32.5
	F			1175	665	0.08	2350	31.0
IIIA	A	0.50	340	1210	680	0.13	2420	45.5
	B			1220	690	0.12	2420	46.0
	C			1195	685	0.12	2395	39.0
	D			1190	680	0.09	2380	36.5
	E			1170	670	0.08	2350	33.5
	F			1180	670	0.08	2360	36.5

*SP - Superplasticizer

3. Test methods

Monitoring of corrosion during the study was carried out using polarisation resistance and corrosion potential measurements. The corrosion equipment employed in the programme was developed by McCarthy [12].

Corrosion initiation was determined as the time to achieve current indicative of a significant level of corrosion. This critical level was based on the value of corrosion current suggested by McCarthy [12], i.e. 0.1-0.2mA/cm^2 and corrosion potential of -250mV.

The samples are exposed to cyclic wetting and drying tank (6 hours drying and 6 hours wetting) developed at University of Dundee. The chloride solution used has a concentration of 2 Mol. The corrosion tests were carried out at regular intervals, so that sufficient data was collected to consider the initiation and propagation stages of reinforcement corrosion in concrete. In parallel to accelerated laboratory tests, a series of specimens were exposed to natural highway and marine environments.

A test period of 12-18 months was used. At the time of 'corrosion initiation' or end of test period, specimens were split and visual observations of the recovered steel were made.

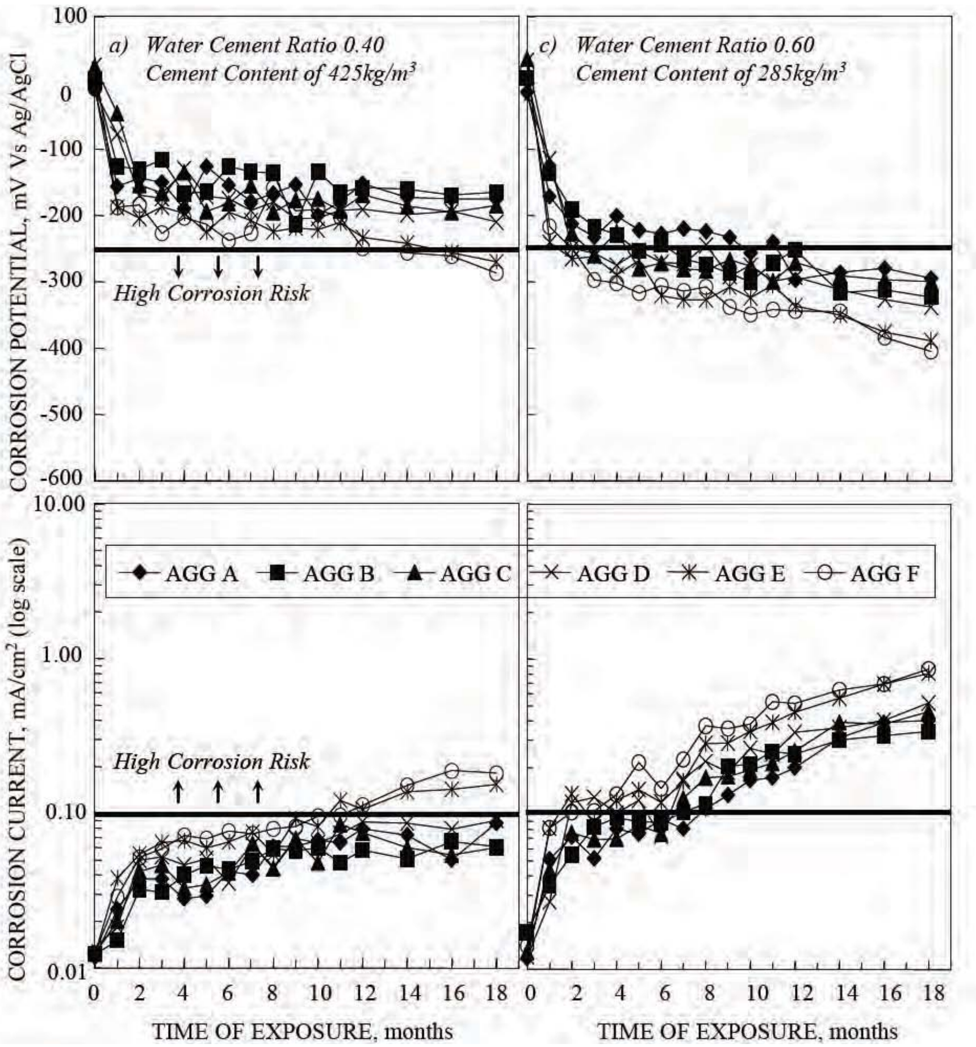

FIGURE 1. VARIATION IN CORROSION POTENTIAL FOR CEM I CONCRETE WITH 25M COVER

Testing of chloride contents at the depth of reinforcement was also carried at the time of initiation of corrosion to obtain an indication of the chloride content required to place reinforcement in an active corrosion state. Both total and water soluble chlorides were considered in this work.

4. Results and discussion

Corrosion potential and corrosion current measurement results

Figure 1 shows the typical corrosion potential and current measurements over the 18 months exposure period. Corrosion potential measurements were erratic during the exposure period, but the general trend was a decrease measured in corrosion potential (more negative) with time. The downward trend in measured corrosion potentials continued throughout the exposure period, but 'level off' towards the end of the test period.

Effect of aggregate characteristics

The result for 12 months exposure are summarised in Table 4. CEM I Concrete made with AGGs E and F produced the most negative potential values after 12 months. Potentials as low as -345mV and -351mV were recorded for AGGs E and F, respectively, suggesting that severe corrosion of the steel may taking place. This can be attributed to the high porosity of the AGGs E and F concrete which during exposure to a chloride environment, allow absorption of large quantities of salts and ingress of chloride through the interconnected pores in aggregates. Chloride ion remains inside concrete following the evaporation of water during the dry period. Thus, high concentrations of chloride ion build up and tend to migrate toward reinforcement owing to capillary absorption and diffusion phenomena.

TABLE 4. VARIATION IN CORROSION POTENTIAL AND CORROSION CURRENT AFTER 12 MONTH EXPOSURE

Cement Type	W/C Ratio	Cement Content kg/m³	Aggregate Type					
			A	B	C	D	E	F
Corrosion Potential after 12 months, mV								
CEM I	0.40	425	-171	-175	-186	-203	-231	-253
	0.50	340	-199	-198	-230	-225	-305	-302
	0.60	285	-299	-312	-259	-301	-345	-351
IIB-V	0.50	340	-202	-195	-231	-241	-245	-253
IIIA	0.50	340	-185	-196	-185	-195	-210	-220
Corrosion Current after 12 months, mA/cm²								
CEM I	0.40	425	0.08	0.06	0.07	0.09	0.11	0.11
	0.50	340	0.09	0.08	0.09	0.09	0.29	0.25
	0.60	285	0.33	0.23	0.31	0.41	0.45	0.49
IIB-V	0.50	340	0.09	0.08	0.07	0.11	0.19	0.18
IIIA	0.50	340	0.06	0.08	0.07	0.08	0.10	0.11

TABLE 5. TOTAL CHLORIDE CONTENT (WATER SOLUBLE CHLORIDE) OF CONCRETE AT INITIATION OF CORROSION

Cement Type	W/C Ratio	Cement Content kg/m³	Chloride Content, % mass of cement					
			Aggregate Type					
			A	B	C	D	E	F
CEM I	0.40	425	-	-	-	-	0.87 (0.79)	0.91 (0.83)
	0.50	340	-	-	-	0.53 (0.49)	0.67 (0.61)	0.69 (0.63)
	0.60	285	0.45 (0.41)	0.46 (0.42)	0.47 (0.42)	0.49 (0.45)	0.62 (0.59)	0.60 (0.56)
IIB-V	0.50	340	-	-	-	-	0.54 (0.45)	0.52 (0.44)
IIIA	0.50	340	-	-	-	-	0.49 (0.42)	0.50 (0.44)

TABLE 6. BS 8500 REQUIREMENTS FOR XS3 CONDITIONS

Cement/Combination	CEM I	IIB-V	IIIA
Minimum Strength Class	C40/50	C28/35	C28/35
Maximum Water/Cement Ratio	0.40	0.50	0.50
Minimum Cement Content (kg/m³)	380	340	340

Effect of water cement ratio

As expected, the influence of aggregates at various w/c ratios is similar to the effects described above for AGGs E and F (higher porosity). At all w/c ratios, corrosion potentials were less negative and corrosion currents were lower with reducing absorption of aggregates. Indeed, for concrete made with AGGs A and B (least porosity), the corrosion potential and current measurements suggest significant improvement in corrosion resistance.

The result also demonstrated that variation of aggregate porosity could have more influence on corrosion resistance than varying w/c ratio at free water content. For example, at w/c ratio 0.50, after 12 months exposure, the potential difference between AGG A (lowest porosity) and AGG F (highest porosity) CEM I concrete was 103mV. When the w/c ratio was varied from 0.50 to 0.40, the change in potential difference for a given aggregates typically ranged between 28mV and 74mV.

Effect of cement type

Inclusion of PFA or GGBS significantly reduces the rate of corrosion, as shown in Table 4. For IIB-V concretes made with high porosity aggregate (AGGs E and F), no corrosion was apparent until 12 months exposure, compared to corresponding CEM I concrete where initiation was noted after 4 to 6 months. Furthermore, there was no indication of corrosion for AGGs A, B, C or D after 12 months exposure for IIB-V concrete. Similar trends observed with IIIA concrete.

Critical chloride content

Chloride contents were measured using comparable un-reinforced concrete specimens positioned adjacent to the reinforced concrete specimens. Total and water soluble chloride contents were measured when monitoring of the reinforced specimens indicated that corrosion was initiated (corrosion current >0.1mA/cm^2). Chloride levels were monitored over the range of 22.5 and 27.5mm. These critical chloride contents at corrosion initiation provide an approximation to the chloride threshold level that causes damage during service life. The results are summarised at Table 5.

It is apparent that aggregate porosity had an effect of the quantity of chloride required to initiate corrosion. Concrete made with AGGs E and F reached initiation earlier than the other aggregates, with the critical water soluble chloride content approximately 0.79 to 0.83% Cl- by weight of cement for the high porosity aggregates. Generally, the chloride threshold value for concrete increases with aggregate porosity. Typically, at w/c ratio 0.60, the water soluble chloride threshold level is about 44% greater with AGG E and 37% greater with AGG F, compared to AGG A. This suggests that chloride can transported through the aggregate as well as the surrounding cement paste.

5. Implications for specifications

The results obtained in this study are directly applicable to XS3 (according to EN 206[4]) conditions where the concrete is maintained in an essentially moist environment. The requirements for BS 8500[3] for an intended working life of 50 years exposed to XS3, with 20mm aggregate and cover of 50mm are given in Table 6.

The results from Table 6 showed that for w/c ratio of 0.50, CEM I concrete containing AGGs E and F do not conform to the C40/50 strength class, which will eliminate the potential problem caused by aggregate porosity. However, IIIA concrete containing AGG F does conform to C28/35 strength class. This has demonstrated that specifying minimum concrete strength and water/cement ratio are not sufficient robust for concrete containing highly absorptive aggregates.

From the same experience programme investigation effect of aggregate in chloride diffusion has been reported by Price et al[13]. Similarly, it is suggested current limitation are not satisfactory means of achieving the desired working life of a structure. All these have challenges the assumption that aggregate has no or minimal influence over chloride induced corrosion. This also suggests that additional specification requirements on the porosity or water absorption of aggregate may be necessary for concrete exposed to XS3 concrete.

From this study, it is suggested that the current guidance for concrete exposed to XS3 conditions is extended to include a requirement that the water absorption of the aggregate is generally limited to a maximum of 1%.

When the absorption of the aggregate is between 1% and 2%, appropriate checks on the potential durability of the concrete should be required. In cases where water absorption is greater than 2%, addition protection (such as coating etc) should be applied to the structure.

6. Conclusions

The conclusions to be drawn are as follows:

- The chloride ion can be transported through the aggregate as well as the surrounding cement paste
- Limiting the water/cement ratio of concrete exposed to chlorides may not in itself, be sufficient robust to ensure adequate long term durability. Therefore, the introduction to specifications of a maximum aggregate water absorption limit for concrete exposed to chlorides is both desirable and technically justified.

Acknowledgements

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Design of continuity slabs and the Metro viaduct designs

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Abstract

The Emirate of Dubai is one of the major cities of UAE, which is expanding rapidly in all sectors, resulting in rapid growth of population and severe traffic congestion problems. The Dubai Roads and Transport Authority (RTA) has come up with "Dubai Metro" as a solution to the problem.

Dubai Metro Project is owned by Dubai's RTA, and comprises two lines, the Red Line and the Green Line. The Red Line extends for 51.8km from Rashidiya to Jebel Ali; 47.4km of which is elevated and the remaining 4.7km in bored or cut and cover tunnel. The Red Line will serve 30 stations; four of which will be below ground including two interchange stations at Burjuman and Union Square. The Green Line extends for 16.8km from Dubai Health Care City to Dubai Airport Free Zone; 10km of which will be elevated and 6.8km in bored or cut and cover tunnel. The Green Line will serve 15 stations; six of which will be below ground.

Atkins' team in Abu Dhabi was commissioned for the detailed design of the Green Line's special precast segmental decks with horizontal radius down to 200m and in particular to study the effect of the tight radii on the behaviour of the decks.

This report presents the longitudinal and transverse analyses, including checking the effects of construction loading on the design, the computer models setup, analysis of the results and the detailed design carried out for the 200m radius decks with spans of 17m, 20m, 24m, 28m, 30.6m, 32m and 34.4m in accordance with BS 5400:Part 4.

The report tries to shed some light on the 3D solid modelling of complex structural shapes, rail loading applied to bridge structures, segmental construction technology and its effect on the design.

1. Introduction

This report outlines some of the procedures employed in the analysis and design of the tight radius curved precast segmental spans of the Green Line viaduct.

Analysis and design is based on a rigorous analysis using brick element solid modelling of the u-shaped deck section.

2. Deck analysis and design

The viaduct is designed for a special carriage defined by the owner as described in section 2.1, and the code applied for the design is BS5400: Part 4.

The deck is designed to satisfy all the specified code serviceability and ultimate limit state (SLS and ULS) criteria.

The deck's cross-section geometry as seen in Figure 1 is an open thin-walled u-shaped section. For such an open web section, it would be difficult to analyse the section using a Finite Element (FE) shell element model. This is because of the transverse flexibility and three-dimensional behaviour of the section.

If we take an example of a typical box girder as in Figure 2, an FE shell element model for this deck is easily set up by just extruding the centrelines of the slabs and the walls into the shown shell elements model.

Unlike any typical box girder decks, the u-shaped deck is more complex to prepare its analysis model.

The main girders on both sides of the u-trough have non-uniform cross section and the down stand cross section at supports differs from that at the mid-

FIGURE 1. SEGMENT CROSS SECTION

span. In addition, the top flange thickness is varying along the span. All this necessitated modelling of this deck using brick elements solid model.

For ease of construction, the deck is made up of precast concrete segments, standardised in terms of their length. For most of the spans, the end segment, which incorporates a down stand, is 1.65m long, and the rest of the segments are approximately 4.0m long. For each segment, we used a naming convention of referring to its start or end by "Joint", a typical convention for segmental construction.

2.1 Loads considered in analysis

For purpose of analysis and design of the decks, the following loads were evaluated from BS5400: Part 2

a) Deck self weight (SW)
b) Superimposed dead load (SDL) -Trackform plinths - running rails and fixings – cable trays and cables
c) Vertical train load (as in Figure 3).
d) Rail vehicle impact factor (determined from a separate 3D solid element model on ABAQUS software)
e) Braking and traction forces
f) Centrifugal forces
g) Nosing forces
h) Lurching forces
i) Derailment loading
j) Gantry, transporter and construction loading (defined by the contractor)
k) Wind loading

LUSAS analysis was done for the load cases a, b, c and d. For other load cases, we developed separate spreadsheets to calculate their values and effects on the overall design of the deck.

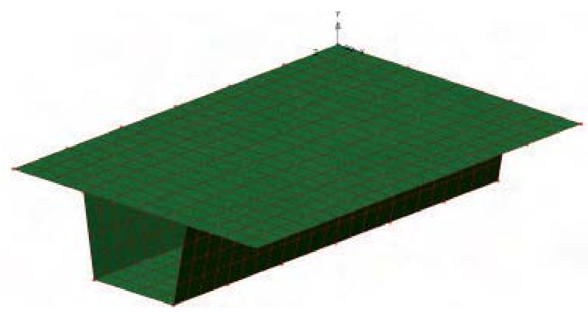

FIGURE 2. LUSAS MODEL, FE SHELL ELEMENTS MODEL FOR A TYPICAL BOX GIRDER

2.2 Solid element analysis:

For the purpose of studying the stresses in the deck due to its self-weight, super imposed dead load, and train load, we used LUSAS software.

LUSAS is an FE analysis software specialized in solid element modelling, and has many features that eases the pre-processing (geometry input) and post processing (output extraction) operations.

When modelling in LUSAS, we used the element type (HX8M) for meshing. (HX8M) elements are 3-dimensional solid hexahedral elements comprising eight nodes each with three degrees of freedom.

We used HX8M elements because it was more efficient for the computer run time, and its results are suitable for the design purposes (an element with 20 nodes could be used but it would take more computer run time, and its results are more useful for research purpose).

Analysis using solid element modelling of the deck in LUSAS ensured that the stress distribution on the deck cross-section at different locations is reflecting the actual behaviour of the deck and that the shear lag effect near supports and effect of the lateral buckling of main girders at mid-span are accounted in the 3-D analysis.

When modelling on LUSAS, we faced a problem that it does not have the ability to model prestressing tendons in brick elements. The software requires that prestressing be applied to frame elements only.

That means for modelling the prestressing force using LUSAS, we would have to model the curved tendons in 3-D space as dummy frame elements with nominal properties; and then try linking those dummy frame elements with each node at the intersection between the solid element and the tendon; which is a very lengthy procedure and prone to error.

To study the prestressing force effect, the prestressing force can be applied to a line beam model, and the stresses calculated at regular intervals along the deck due to this applied force (while studying serviceability limit states, prestressing effect is studied in linear elastic range of both steel and concrete).

By using RM2006, we adopted this approach in the deck analysis and used the prestressing effect from RM analysis into the design.

To study the moving load effect on the deck, train wheel locations were determined using the influence lines extracted from the RM model.

2.3 Line beam analysis

For the purpose of studying the prestressing effect on the deck and obtaining the influence lines, we used RM2006 software. RM2006 is a very powerful tool for assessing the prestressing effects, and for obtaining short and long-term losses in the prestressing due to creep and shrinkage of concrete, steel relaxation and elastic shortening of concrete.

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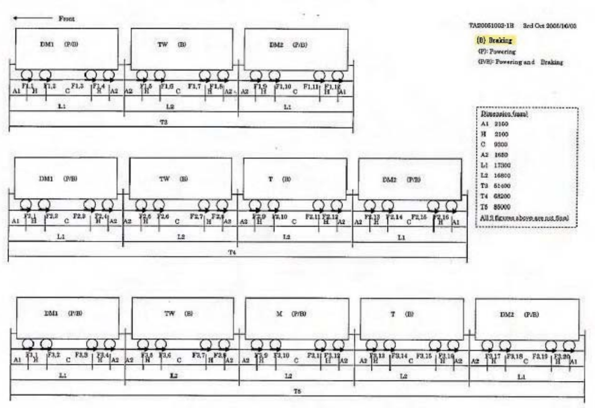

FIGURE 3. TRAIN CONFIGURATION AS SPECIFIED BY THE CLIENT

From RM influence line analysis, we obtained the train locations that would give the maximum straining actions at segments' joints.

As the span/width ratio was less than two for some spans, we made a grillage model based on the recommendations by EC Hambly "Bridge Deck Behaviour".

The main difficulty of applying prestressing to segmental bridges' decks is the standardisation of tendon locations at each joint; these points must be respected while determining the cable profile, as this is a construction requirement.

The fixed locations for tendons do not give any flexibility to the design, and care should be taken to make sure that the profile adopted in design is applicable and matching the real profile executed on site.

2.4 Construction load analysis

For the deck construction, a steel launching gantry and a segment transporter are used. The steel launching gantry used for construction has three main supports: The Front leg; which is supported on the viaduct piers, the Rear leg and the Auxiliary leg; both are supported on the adjacent deck's girders.

The launching steel gantry is a heavy structure of about 515 tonnes; which during its launching forward, one of its legs (the Auxiliary leg) carrying about 260 tonnes is directly supported on a segment joint.

During construction, tendons are not grouted. This means that when determining the ULS flexure capacity of section, during construction the prestressing tendons shall be considered as "external" tendons, thus reducing flexure ULS capacity of the deck by about 10 to 30%.

3. Design

Longitudinal design of the deck was done in accordance with BS 5400 code provisions for prestressed concrete sections; and the transverse design was carried out to the code provisions for reinforced concrete sections.

3.1 Longitudinal Design

For longitudinal design, SLS check of deck is done by extracting the LUSAS stress' output for the load cases: Self Weight, superimposed dead load and Train load.

A combination is made from these effects with the effects from other load cases; applying the appropriate load factors. Then these stresses were combined with the stresses from RM analysis and three cases were checked.

Case 1: Self Weight + Prestressing effect: Immediately after jacking, so that it includes immediate losses in prestressing force: wedge draw-in, friction losses, steel relaxation and concrete elastic shortening.

Case 2: Service Loads + Prestressing + Short term Creep and shrinkage:

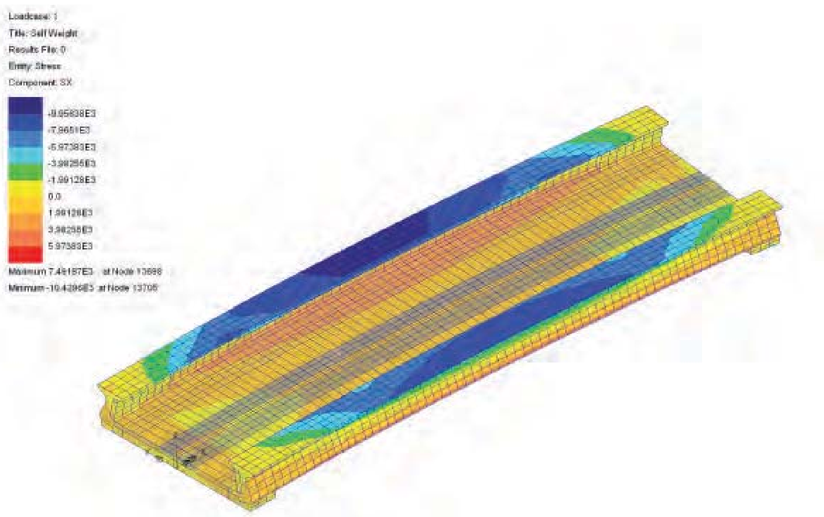

FIGURE 4. LUSAS MODEL, SHOWING STRESS DISTRIBUTION DUE TO SELF WEIGHT OF DECK (34.4M SPAN WITH RADIUS 250M)

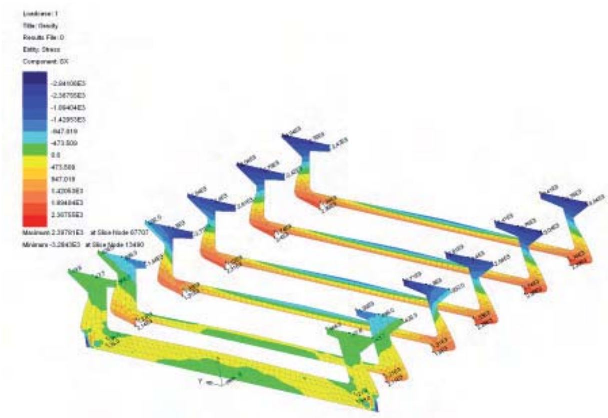

FIGURE 5. LUSAS MODEL, SHOWING STRESS DISTRIBUTION DUE TO SELF WEIGHT OF DECK (SLICES AT JOINTS LOCATIONS)

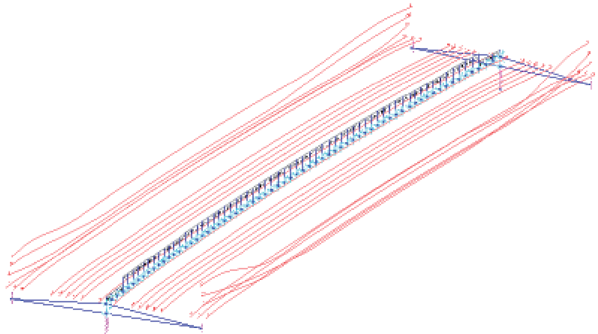

FIGURE 6. RM MODEL, SHOWING PRESTRESSING TENDONS

Representing working loads of the structure plus short term creep and shrinkage losses in prestressing for a period of 180 days.

Case 3: Service Loads + Prestressing + Long term Creep and shrinkage:

Representing working loads of the structure plus long term creep and shrinkage losses in prestressing for a long term period or the structure life time of 16500 days.

The stresses were checked for the points on the deck cross section shown in Figure 10.

An additional "During Construction" case, where the stresses on the deck needs to be checked for the gantry load, and also where the stressing sequence of cables needs to be checked and accounted for in the end block design.

Depending on the deck configuration (whether a straight or curved deck), the stresses on the left and right girders were studied to ensure that the final stresses are within the allowable stresses of 1MPa in tension (approved as departure from standard by RTA), 20MPa in compression.

ULS flexure capacity of the section was computed considering tendons unbonded to concrete during the construction stage. Ultimate shear stresses on the deck were checked as the envelope for maximum shear stress due to the loads (SW + SDL + Train load) and the resulting stress checked against the code limits of (5.3/5.8 MPa for concrete grades 50/60 MPa respectively).

The ULS shear capacity of section, capacity of shear keys and epoxy glue between concrete segments were evaluated according to AASHTO/ LRFD provisions for shear friction design.

3.2 Transverse design

For transverse design, the segment is considered as a reinforced concrete section, and code checks are made accordingly. For this design, local distribution of wheel load, together with Self Weight and SIDL results, were extracted from LUSAS. A useful tool in LUSAS to extract results called "slices". By using "slices" command, we extracted the stresses distribution in the transverse direction of the deck under the train loads. In addition, from these slices, the software would integrate these stresses, and extract an output of Flexure, Shear and Torsion straining actions, to be used later in the design.

We used SAM software to evaluate the ultimate flexure capacity of the deck's transverse cross section, and to check the SLS crack width criteria of (0.2mm).

For transverse design, a special case of bearing replacement was also checked. In this particular loading case hogging moments near girders had to be checked, and top reinforcement at edges were introduced to cater for this particular loading case.

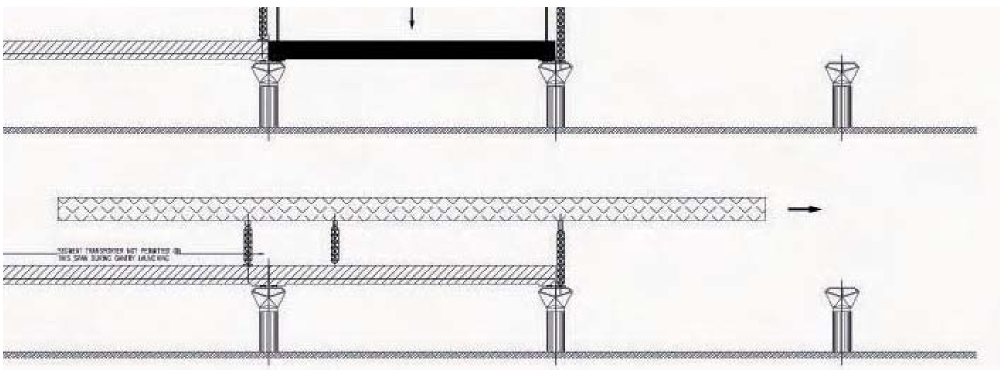

FIGURE 7. SCHEMATIC FOR GANTRY CONFIGURATION

FIGURE 8. GANTRY'S FRONT SUPPORT RESTING ON A PIER, ERECTED DECK IN POSITION AFTER ITS PRESTRESSING IS COMPLETED

FIGURE 9. GANTRY'S REAR AND AUXILIARY SUPPORTS RESTING ON DECK'S GIRDERS

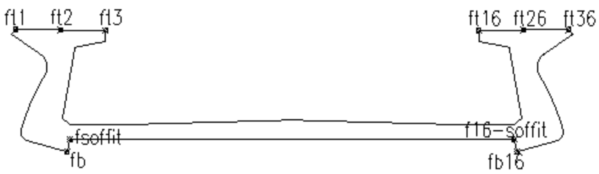

FIGURE 10. CONTROL CHECK-POINTS FOR SLS STRESS CHECK

FIGURE 11. SHEAR KEYS

4. Deck construction

A special area is established for the assembly of reinforcement, called rebar jigs, as seen in Figure 12, and then reinforcement cages are moved into the casting cells for concrete casting.

Segments are made in a purpose built casting yard, where a specially designed steel formwork known as longitudinal mould or casting cell is used to cast the precast segments, as seen in Figure 13.

The casting cell consists of side shutters, bulkheads, a soffit form and an internal shutter, as seen in Figure 14. The side shutter forms the external shape of the u-trough; the inner shutter forms the inner shape of the u-trough, while the bulkheads form the shear keys and tendons' duct locations.

The bulkheads vary in terms of shear keys shapes and sizes and ducts' locations according to the cast segment's location along the deck.

FIGURE 12. ASSEMBLY AREA FOR SEGMENTS' REINFORCEMENT, REINFORCEMENT CAGE IS READY TO BE MOVED TO CASTING MOULD

FIGURE 13. THE STEEL FORMWORK USED FOR SEGMENTS CASTING, SHOWING THE TENDONS' FIXED POINTS

Inner shutter is adjustable for the down-stand segments, as the girder web thickness is thicker than that for a mid-span segment. For some decks, the top flange thickness at mid span increases compared to that at near ends. This also will need to have an adjustable inner shutter.

For this type of construction, the gantry used will have a height over deck more than twice the deck height. First all segments will be lifted by cables hanging from the gantry and adjusted on two levels, to ease the application of epoxy glue to the segments' interfaces.

The epoxy glue used in this project is a tropical grade to give a setting time of about 2 hours. The workers should apply this glue manually, so a suitable gap between segments is required.

After applying the epoxy glue, the segments are matched together, and temporary prestressing using steel brackets (Figure 18) is applied to hold the segments until finishing threading of prestressing tendons, and application of permanent prestressing force.

FIGURE 14. INNER SHUTTER AND BULK-HEAD FOR A MID-SPAN SEGMENT

FIGURE 15. STEEL CAGE OF ONE SEGMENT AND THE DUCTS IN POSITION, BULK-HEADS IN POSITION, WHILE INNER SHUT-TER BEING SLIDE INTO THE DECK

FIGURE 16. ONE SEGMENT AFTER CONCRETE HARDENING WHILE STILL ON THE SOFFIT SHUT-TER, READY TO BE MOVED TO STORAGE YARD

FIGURE 17. STORAGE YARD

FIGURE 18. STEEL BRACKETS FOR AP-PLYING TEMPORARY PRESTRESS

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Replacement of the stay cables on Penang Bridge

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Abstract

Penang Bridge links Penang Island with the Malaysian mainland peninsula. It was completed in 1985 and is currently the only link and is therefore a vital crossing. The main cable stayed bridge has a span of 225m with side spans of 107.5m. The bridge carries two carriageways, each with three lanes.

The bridge cables comprise lengths of coupled high-yield bars within a grouted steel tube. An assessment of the structure in 1996-1999, using modern design standards, identified large overstress in the shortest cables anchored close to the pylons. Other cables were also found to be overstressed, but less so. As a result, bearings were installed at the piers to relieve load from the shortest cables and the second shortest sets of cables were replaced. An acoustic monitoring system was also installed on all cables to detect any problems. Since then, two bar breaks have been detected in other cables which led to their replacement. Inspection and testing showed fatigue of the couplers to be responsible for the breaks.

Following the first bar break detected in December 2004, an additional assessment was completed by Atkins. This paper describes the assessment of the bridge cables, examines the consequences of a cable failure and discusses the reasons why the decision to replace all the cable stays was made. It also describes the specification for the new replacement stay system and its advantages over the old system.

1. Background to the project

Penang Bridge links Penang Island with the Malaysian mainland Peninsula. It was completed in 1985 and is currently the only link and is therefore a vital crossing. The overall length of the viaduct is approximately 13.5km, which mostly consists of 40m span post-tensioned beams. The main cable stayed bridge has a span of 225m with side spans of 107.5m. The bridge carries two carriageways, each with three lanes. The bridge is shown in Figure 1 and the numbering system for the cables is shown in Figure 2.

The bridge cables comprise lengths of coupled high-yield bars within a grouted steel tube. An assessment of the structure in 1996-1999, using modern design standards, identified large overstress in the shortest cables, M1 and E1, anchored close to the pylons. Other cables were also found to be overstressed, but less so. As a result, bearings were installed at the piers to relieve load from the shortest cables and the second shortest sets of cables were replaced. An acoustic monitoring system was also installed on all cables to detect any problems. Since then, two bar breaks have been detected in other cables which led to their replacement. When the first of these cables was removed, an additional three bars were found to be broken at couplers. Testing of the couplers showed that some were showing signs of fatigue damage, while fatigue was found to be the cause of the three broken ones.

Following the first bar break detected in December 2004, an additional assessment was completed by Atkins. This identified that many of the current cables were significantly overstressed in accordance with current standards. The predicted overstress in the cables, combined with the history of bar breaks, led to the decision to replace the remaining cables on the bridge. The remainder of this paper describes the assessment of the bridge cables, examines the consequences of a cable failure and discusses the reasons why the decision to replace all the cable stays was made. It also describes the specification for the new replacement stay system and its advantages over the old system.

FIGURE 1. PENANG BRIDGE

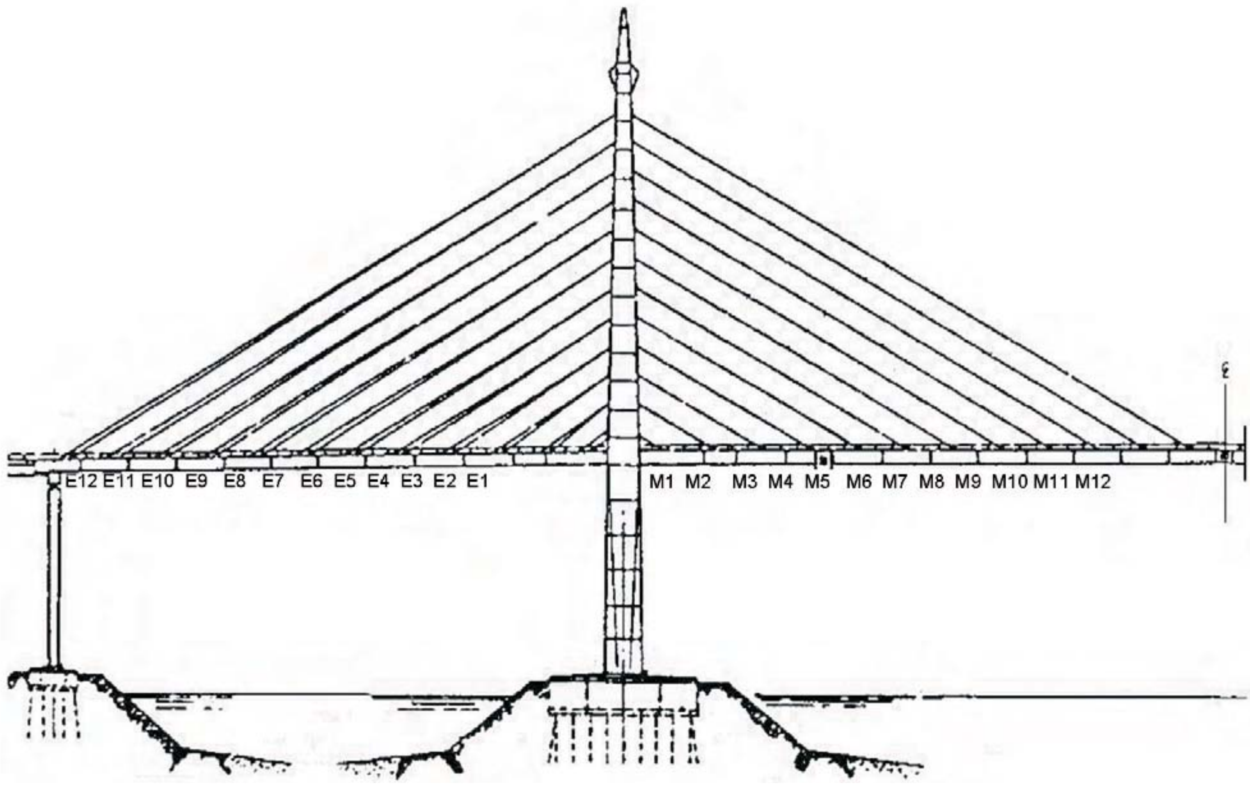

FIGURE 2. CABLE LAYOUT AND NUMBERING SYSTEM

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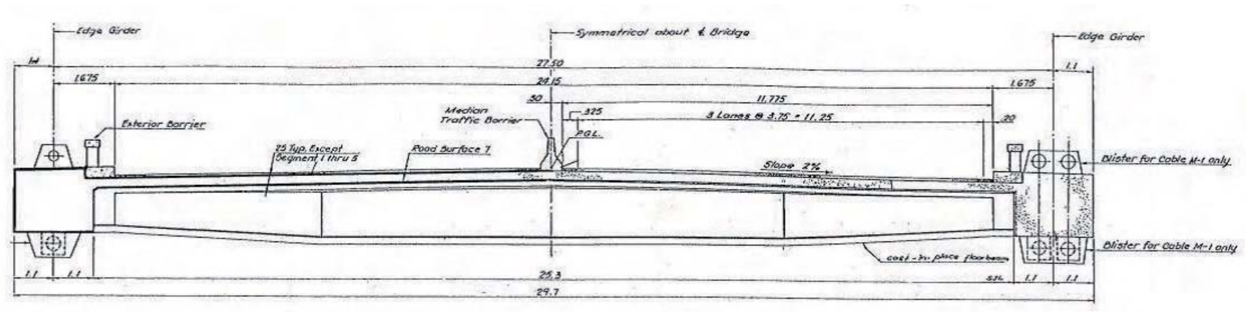

FIGURE 3. CROSS-SECTION THROUGH DECK

2. Description of cable-stayed bridge

The cross-section of the deck has two edge girders 1.75m high and 2.20m wide, spaced at 27.5m centre to centre. These act compositely with the deck slab to form a cross-section with total width of 29.7m. Post-tensioned transverse floor beams, 3.0m centre to centre longitudinally, connect the edge girders and support the 25cm thick roadway slab spanning longitudinally. Figure 3 illustrates a cross section through the deck.

Each of the two edge girders is supported by a vertical plane of stays cables with a harp configuration. The main span is supported by 12 single cables and the back spans are supported by 12 pairs of cables, with a total of 144 cables used to support the structure.

3. Original cable system and history of bar breaks

The original stays were constructed using up to ten high-yield bars, of 36 or 32mm diameter and strength 1100 MPa which were grouted into steel tubes of either 273.1 mm or 168.3mm diameter, see Figure 4. The complete stay was made by joining lengths of bar with threaded screw couplers. The cables anchor at concrete "blisters" cast on the edge beams on the deck. The steel tubes are structurally connected to the bridge and were grouted up after completion of the bridge deck construction and adjustment of the cable stay bar loads. The bars pass through the edge beams and terminate with end plates on the beam soffit.

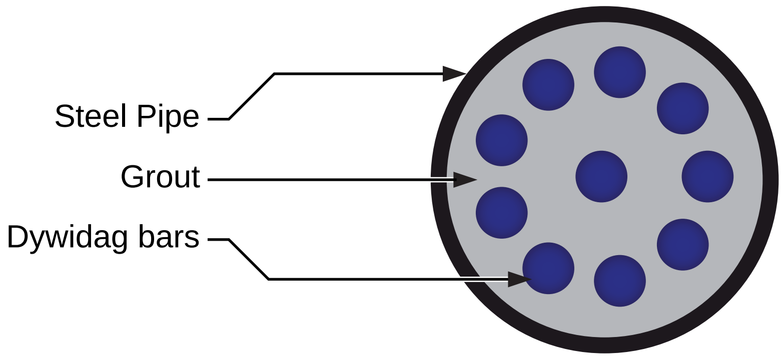

FIGURE 4. TYPICAL EXISTING CABLE

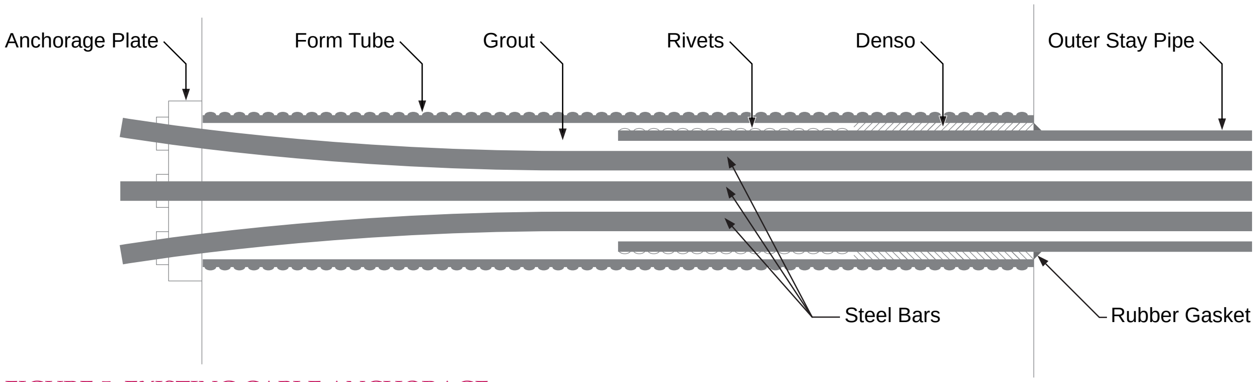

FIGURE 5. EXISTING CABLE ANCHORAGE:
(I) EXISTING ANCHORAGE DETAIL

FIGURE 5. EXISTING CABLE ANCHORAGE:
(II) DETERIORATION OF RUBBER GASKET

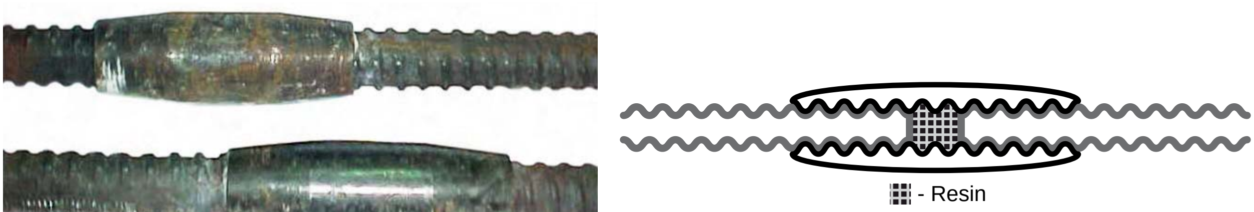

FIGURE 6. TYPICAL COUPLERS AND DIAGRAMMATIC VIEW OF RESIN INJECTION

The high-yield bars are grouted up within this pipe. The full stay load is partly transmitted by the bars through their end plate and partly through bond along the ribbed steel tube anchored in the concrete. The anchorage detail is shown in Figure 5. The anchorages were not detailed to be replaceable; the outer steel pipe is concreted into the edge beam within a steel form tube and cannot be removed, while the bars are grouted up within the pipe. The outer form tube is cast flush with the face of the concrete blisters which allows the pipe to corrode. The resulting expansive corrosion products cause the concrete to spall which then causes the rubber gasket at the pipe exit to separate. This potentially allows further ingress of contaminants.

The anchorage plate and form tube are also shown being installed during original construction in Figure 17. During construction, the bar couplers were identified as having a poor fatigue resistance with two coupler failures recorded during the construction of the bridge. As a result, laboratory tests were performed on the coupler detail which showed that the basic coupler was equivalent to a "G" detail to BS 5400:Part 102. This indicated that it had a poor fatigue resistance. Further tests were done with a resin injected into the couplers to try to improve the fatigue resistance of the detail by eliminating play and ensuring a more even distribution of stress to the threads. This resin injection improved the fatigue detail to

FIGURE 7. TYPICAL COUPLER FAILURE

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the equivalent of a "D" detail to BS 5400:Part 10. Figure 6 schematically illustrates the area the glue was intended to be injected into to improve the fatigue resistance.

The initial site failures also led to proof testing of all the couplers to a load equal to 80% of the ultimate tensile strength (UTS) of the bars. This testing led to nine further coupler failures during the proof load testing which were attributed to defective material and workmanship problems.

Since the construction of the bridge, couplers have been recovered from the existing cables and these samples have shown that under site conditions, it had proved very difficult to inject the glue effectively. The large number of couplers which were glued during the construction meant that, statistically, some would be inadequately treated.

In 2000, when bearings were installed at the piers, the second shortest cables, M2 and E2, were replaced. Examination of these cables showed that one of the 92 couplers had failed. An acoustic monitoring system was installed in 2003 to monitor the cables and detect bar breaks. In 2004/2005, cable M9 on the southwest side of the structure was replaced after a bar break was detected in the cable. After the bars within the cable were examined, four of the 80 couplers were found to be broken or damaged. Tensile tests were subsequently carried out on 40 bars containing couplers. Of these, one coupler failed at a load of 274.57 kN, 25% of its expected breaking load and a further eight bars ruptured at the coupler in a brittle manner below the expected breaking load.

Typically, the couplers had failed by splitting into two 'C' shaped pieces as shown in Figure 7. Inspection by Universiti Teknologi Malaysia (UTM) with a scanning electron microscope revealed that cracking had initiated at the first thread, where the load was concentrated, and had propagated until brittle fracture had occurred. The shape of the couplers, with the thickness tapering towards the outer threads, did not help this. It is believed that the couplers were initially of constant thickness throughout, but were modified to be this shape early on during construction to facilitate placement of bars.

In 2006 a further bar break was detected, and the cable E9 on the southwest side was replaced. Crucially, since the acoustic monitoring system was not installed until 2003, it was uncertain what the condition of the other cables was and how many bar breaks had occurred before the monitoring system had been installed.

The anchorages themselves also gave some cause for concern. There was uncertainty in their condition with the acoustic monitoring system detecting activity in 30 of the 120 anchorages; four of these were significantly more active than the others. This activity could have been deterioration, but could not be confirmed.

4. Assessment of the structure

As a result of the bar break detected on cable M9 in 2004, Atkins assessed the structure in 2005. The assessment considered the key stages of the bridge construction including:

- Incremental launching of the deck
- Application of tie-down forces at end supports
- Installation of longitudinal prestress in the deck slab at midspan

Previous strengthening operations including installation of bearings at the pylons, replacement of the M2/E2 stays and addition of longitudinal buffers between the main towers and deck to enhance seismic resistance.

Although the whole structure was assessed, the key focus was on the design of the cables due to the previous failures detected. Cables were checked for static strength, fatigue performance under traffic and vibration. Seismic assessment was also carried out using response spectra supplied by UTM and using load factors specified in BS EN 19901, but this was found to be less critical than the static loading and is therefore not discussed further here.

Static strength assessment of the cables

The cables were assessed statically to determine their adequacy. To reflect the construction sequence of the structure, the assessment assumed that the self-weight load was carried by the internal bars only with the SDL loads and live loads carried by both the bars and the external steel tube which is grouted to the bars and grouted into the edge beam. Three live load conditions were considered:

i) HA and HB (45 units) loading to BS 5400:Part 2 - 19783

ii) Bridge Specific Assessment Live Loading (BSALL) which had been derived in an earlier commission and which was slightly less onerous than the BS 5400:Part 2 - HA loading

iii) Special vehicles defined by JKR, the Malaysia Public Works Department, in conjunction with associated HA loading to BS 5400:Part 2.

The bars in the cables were checked at ULS and SLS by comparing the combined load effects from the model with stress limits of 67% and 45% of the ultimate bar resistance at ULS and SLS respectively. These are the current stress limits provided in Eurocode BS EN 1993-1-11 : Design of Steel Structures : Design of structures with tension components made of steel4 when there is no special provision to prevent bending at the anchorages as was the case on Penang. These codified stress limits at SLS and ULS for the bars are based on general fatigue considerations, particularly adjacent to anchorages, and are well below the elastic limit of the bars, generally taken as the 0.1% proof stress, f0.1%, which is around 85% UTS. However, as some creep strain can occur beyond 80% UTS, this lower stress was regarded as a practical limit to elastic

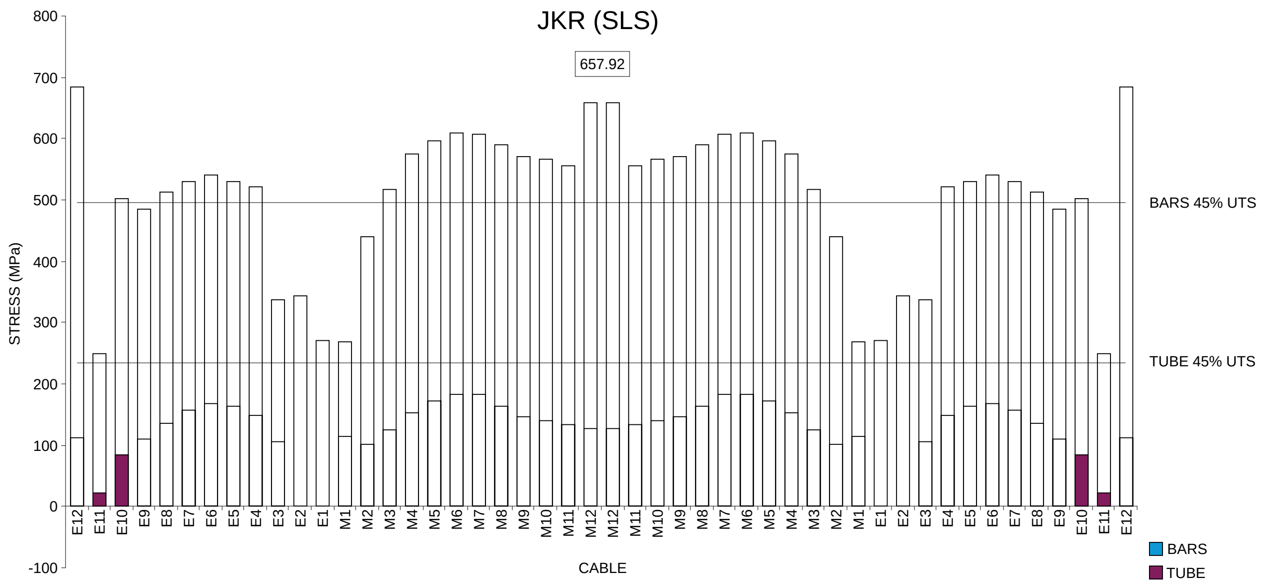

FIGURE 8. CABLE STRESSES UNDER JKR LOADING AT SLS

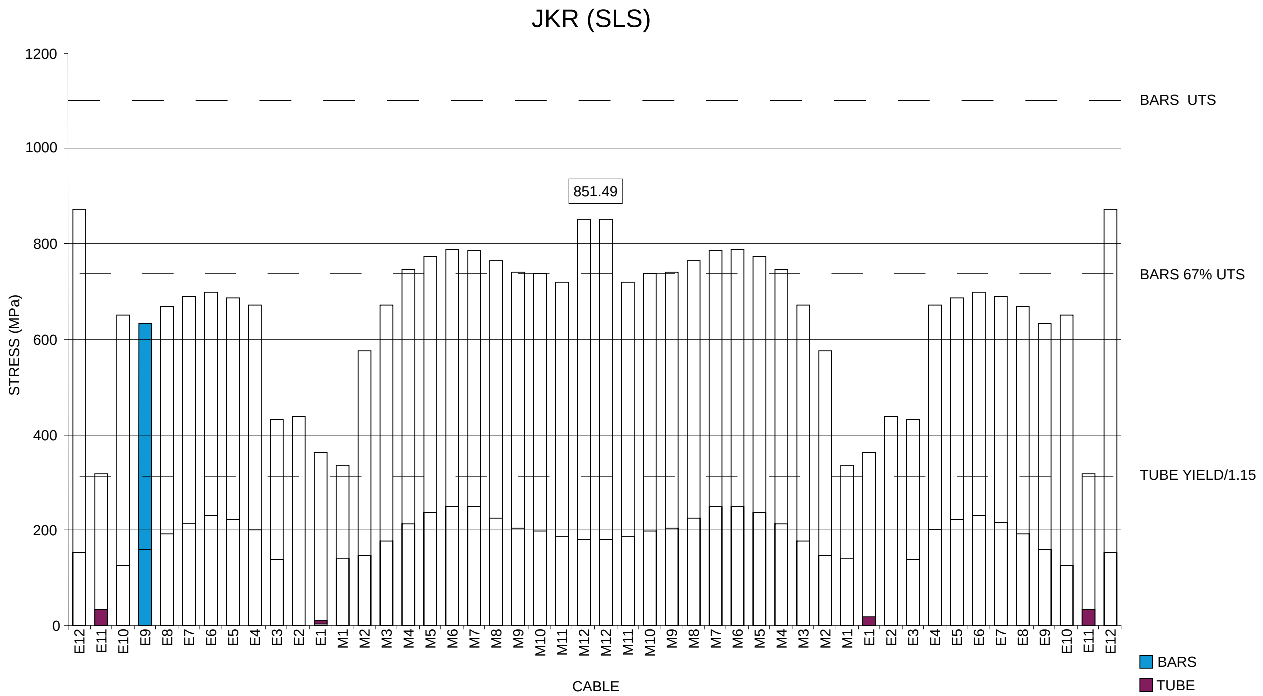

FIGURE 9. CABLE STRESSES UNDER JKR LOADING AT ULS

behaviour. 80% UTS was also the load to which the bars and couplers had been proof tested during construction.

For the tubes, a stress limit of 45% UTS, i.e. 234 MPa, was adopted at SLS and at ULS, a limit of 0.87 fy was adopted, i.e. 313 MPa.

The assessment showed that the build up of stress in the bars and the outer steel pipe led to many of the bars in the cables having an apparent overstress, some as much as 39% based on the above current codified SLS criterion under the JKR vehicle loading. This was the most adverse of the live loading situations considered. The greatest SLS stress in a bar under this loading equated to 63% UTS which was however still well below the practical elastic limit of 80% UTS. Figures 8 and 9 show the bar stresses at SLS and ULS respectively for JKR loading. The overstress was less for HA live loading to BS 5400-2:1978 and BSALL loading.

At ULS, most bars met the proposed stress limit of 67% UTS with the exception of cables E12, M12 and M5 to M8. However, all cables complied with a higher stress limit of f0.1% / γm = f0.1% / 1.15 = 74% UTS corresponding to design yield.

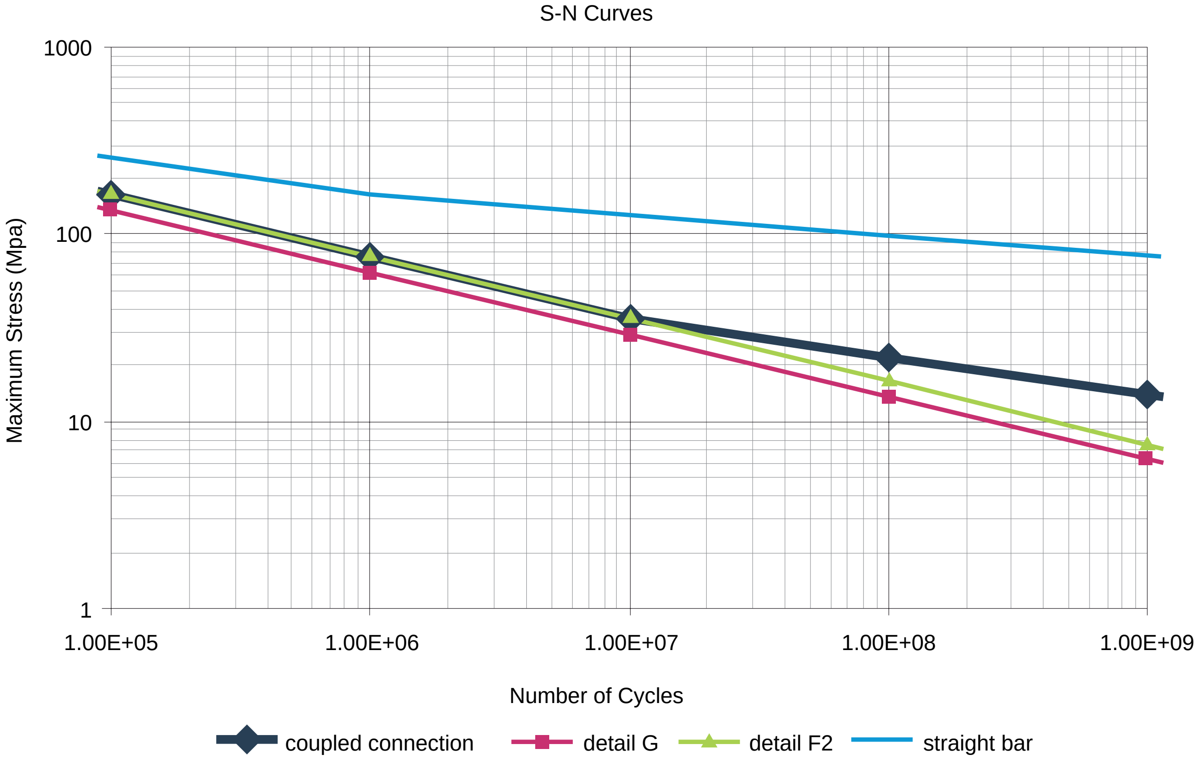

FIGURE 10. COMPARISON OF EUROCODE PREDICTIONS FOR S-N CURVE FOR A COUPLER AND FOR A BS 5400 PART 10 G DETAIL

Because of the high ULS stresses in some cables, the cable forces were also compared against the total resistance available from the bars plus pipes acting together, assuming redistribution from bars to pipes. This shows a good reserve of total strength in a stay at ULS if the failure is ductile, but it is unlikely that the bars (governed by the coupler connections) would have adequate ductility to realise this distribution.

A very important consideration in this assessment was that the actual condition of the stays was unknown. The stress predictions and conclusions for the bars above made no allowance for possible existing bar breaks in cables and consequent loss of steel area. This uncertainty, coupled with the already significant overstresses, was highly influential in arriving at a decision to replace the cables.

Fatigue Assessment of the Cables

The bridge was assessed for fatigue using the BS 5400:Part 10 fatigue vehicle to determine the stress ranges in the cables from live load. There was an awareness that the fatigue performance of the couplers was highly sensitive to the quality of glue injection carried out. Therefore, the assessment assumed that the couplers could have a detail anywhere between class D and class G, though the class G detail was considered the most appropriate as many of the cables which had thus far been recovered showed little glue had entered the couplers. With the assumptions of a class G detail, there were numerous failures predicted in the cables within the bridge's remaining intended design life.

The fatigue checks were performed using the simple method of assessment specified in clause 8.2 of BS 5400:Part 10. This method is intrinsically conservative, but traffic-induced vibration and stay bending at anchorages was not initially considered. As discussed in the next section, the detailing of the anchorages and low natural frequencies of the cables made these effects significant.

The results showed that most of the back stay cables complied with the stress limit for a class G detail using the simple method of calculation to clause 8.2. Those exceeding these limits would have complied with a class F detail, thus some of the bars might have had adequate fatigue life if the resin injection had been adequately performed. Most of the main span cables did not comply with the stress limit for a class G detail, but they all complied with the stress limit for a class F detail, again making the adequacy of the stays very dependent on the workmanship for the resin injection.

It is unlikely that the original designers knew that the fatigue performance of the coupled bars would be as poor as that corresponding to a class G detail. For example, it was only in the early 1990s that threaded couplers became subject to British Board of Agrément (BBA) and UK Certification Authority for Reinforcing Steels (CARES) certification for fatigue applications in bridges in the UK. More recent testing has however shown that the fatigue performance of threaded couplers can indeed be very poor. Eurocode BS EN 1992-1-15 now includes S-N curves for mechanical couplers that can be used in design calculations.

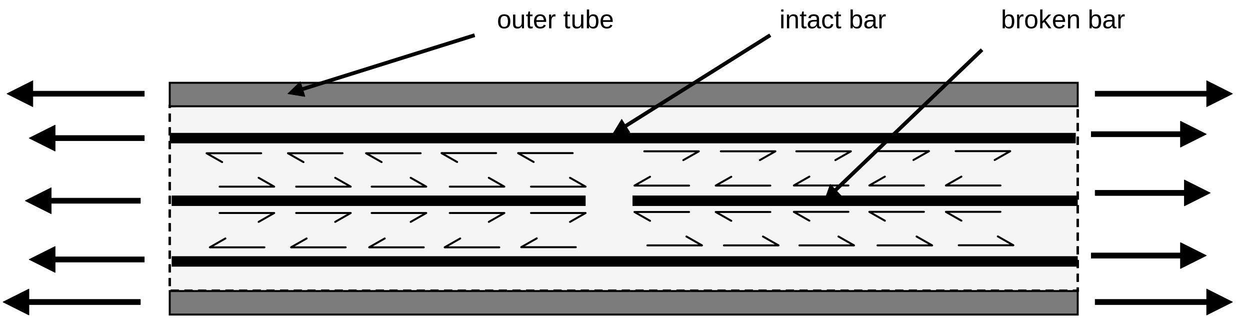

FIGURE 11. RE-ANCHORING OF BROKEN BAR AND TRANSFER OF FORCE TO ADJACENT BARS

Comparison of the S-N curves for such couplers reveals a predicted fatigue performance close to that of a class F2 detail to BS 5400:Part 10 (which is in turn close to class G) as shown in Figure 10. The performance of the threaded screw couplers on Penang is therefore similar to that which would be predicted by the Eurocode.

Vibration of cables

The cable stays on Penang Bridge comprise bars and steel tubes operating at relatively low stress compared to a modern stranded cable and are also comparatively heavy given the low operating stress and grout filling to the outer tubes. The combination of low stress and high mass per unit length means that the cables have low natural periods of vibration compared to modern stranded cables.

Inspection of the main bridge revealed that there were observable vibrations of the cables. These appeared to be caused by live load as significant vibrations were observed in the absence of any discernable wind. This so-called parametric excitation occurs because a stay can only balance its self-weight by taking up the form of a catenary. Increasing the cable axial force causes the cable to straighten a little. Live load crossing the bridge causes a transient increase and reduction of axial force in the cable, which provides the input motion for vibration to start. Cable vibrations can build up when a natural frequency of the structure closely matches that of the cable stay. Some vibration might be expected on Penang because it has atypically low natural frequency in their fundamental modes of vibration.

From site observations, all cables appear to vibrate with an amplitude of around 50mm and a frequency of around 2HZ. Short cables vibrate in one single half wave while longer cables vibrate in multiple wavelengths at similar frequency. Using these assumptions, hand calculations on cable M5 suggested that vibration might induce a stress range of about 20 N/mm2 in the cables away from the anchorages and a potentially higher stress range at the anchorages where bending of the cables is restrained by the concrete encasing the outer steel pipe. This was considered to be potentially a significant increase to the live load stress range from traffic. The results were however sensitive to vibration amplitude and mode shape, including coexistent movement of the deck, and these were not measured directly on site. Accelerometers could have been installed to gather this data but the decision to replace the cables was made before this could be done.

5. Consequences of stay cable failure

The combination of high cable static loads, fatigue concerns and the history of bar breaks in the cables gave rise to concerns over possible stay cable failures. This led to the decision to check the adequacy of the bridge assuming the loss of a cable. The main purpose was to establish whether the bridge should be shut to traffic immediately or whether there was some residual load capacity remaining if a cable should fail. This was not a condition considered in the original design.

As some permanent damage could reasonably be expected in such an extreme event as a cable failure, only ultimate limit states were considered in the analysis. The specific ultimate limit state combination considered (using terminology in BS 5400) assumed the following reduced load and material factors based on Eurocodes BS EN 1990 and BS EN 1992:

$\gamma f3 = 1.0$

$\gamma m = 1.0$ (for reinforcement)

$\gamma m = 1.25$ (for concrete)

γfL = full ULS values to BS 5400 Part 2 1978 (for Dead Loads)

$\gamma fL = 1.0$ (for Live Loads – BS 5400 Part 2 1978)

The reduced load factor for traffic actions was considered suitable for analysis of cable failure cases because the duration before intervention and the imposition of traffic restrictions once a cable failure had been detected would be relatively short. The design value of traffic actions to consider in this period would therefore be much less than those appropriate to the 120 year design life. (The loading to BS 5400 is slightly greater than the BSALL). These load factors are also broadly consistent with the recommended values for accidental situations in Eurocode BS EN 1990.

The effects of a complete cable failure (all bars and the outer pipe) were considered by analysing the structure with one cable removed. The effects of permanent loads

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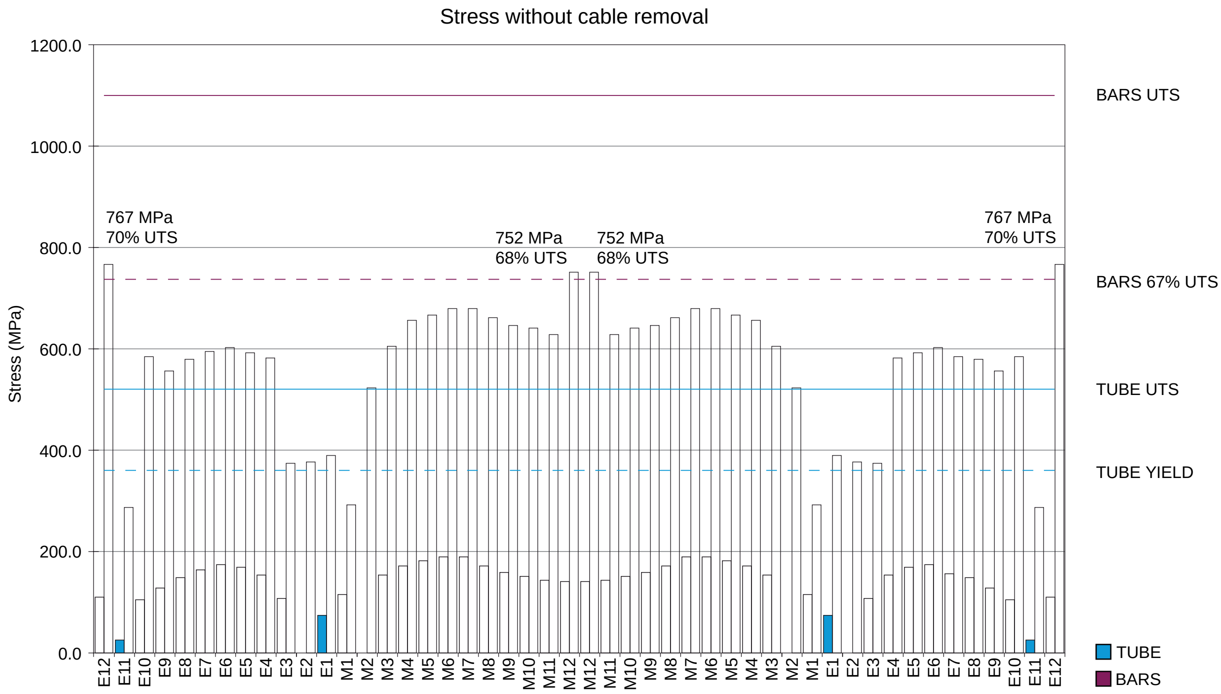

FIGURE 12. STRESSES IN CABLES (WITHOUT CABLE REMOVAL) AT ULS FOR CABLE SEVERANCE PARTIAL FACTORS

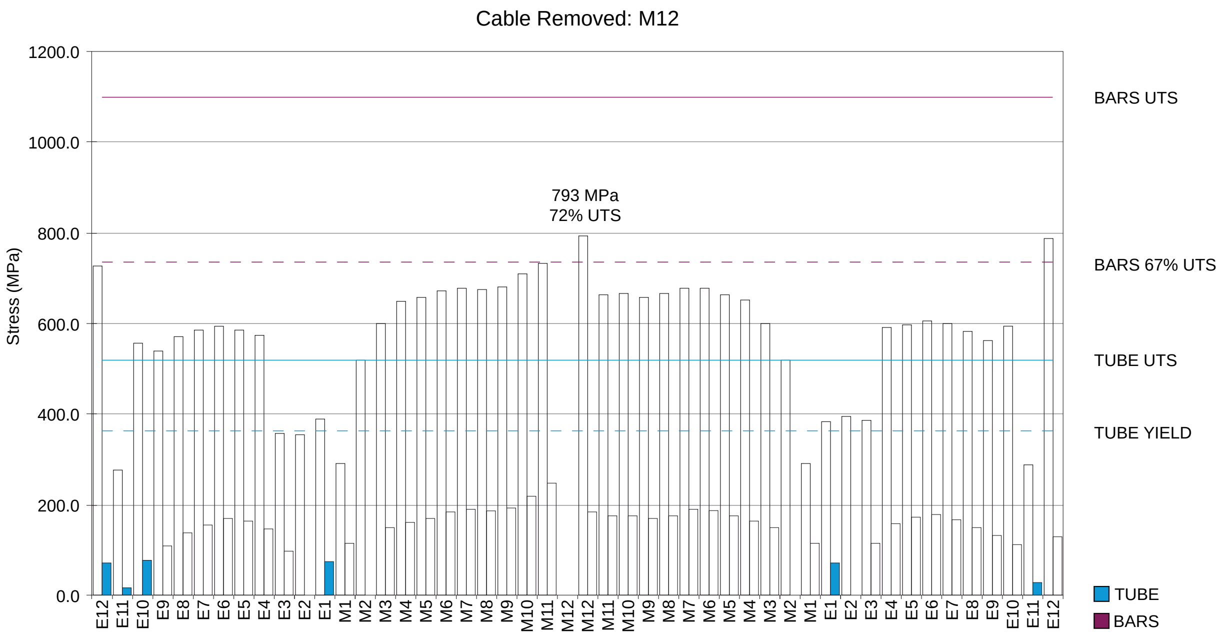

FIGURE 13. STRESSES IN CABLE AND BARS AFTER REMOVAL OF A CABLE M12

were redistributed to the other elements and the live load applied to the modified structure. The dynamic effect of a cable failure caused by bar fracture was considered to be small with this particular stay system because of the large damping that would be provided by the yielding outer steel pipe. This analysis was considered to be conservative as it is unlikely that a cable would lose all its strength, as discussed below, other than perhaps through a vehicular impact.

Failure modes

In the event of a brittle bar failure, the force in that bar would locally be re-distributed to the other bars and the tube. Remote from the failure location, the broken bar would re-anchor due to the bond to the surrounding grout as shown in Figure 11 and the overall stiffness of the cable would not be significantly affected. As a result, the total cable force would not reduce significantly. At the failure location, the stress would increase in the other

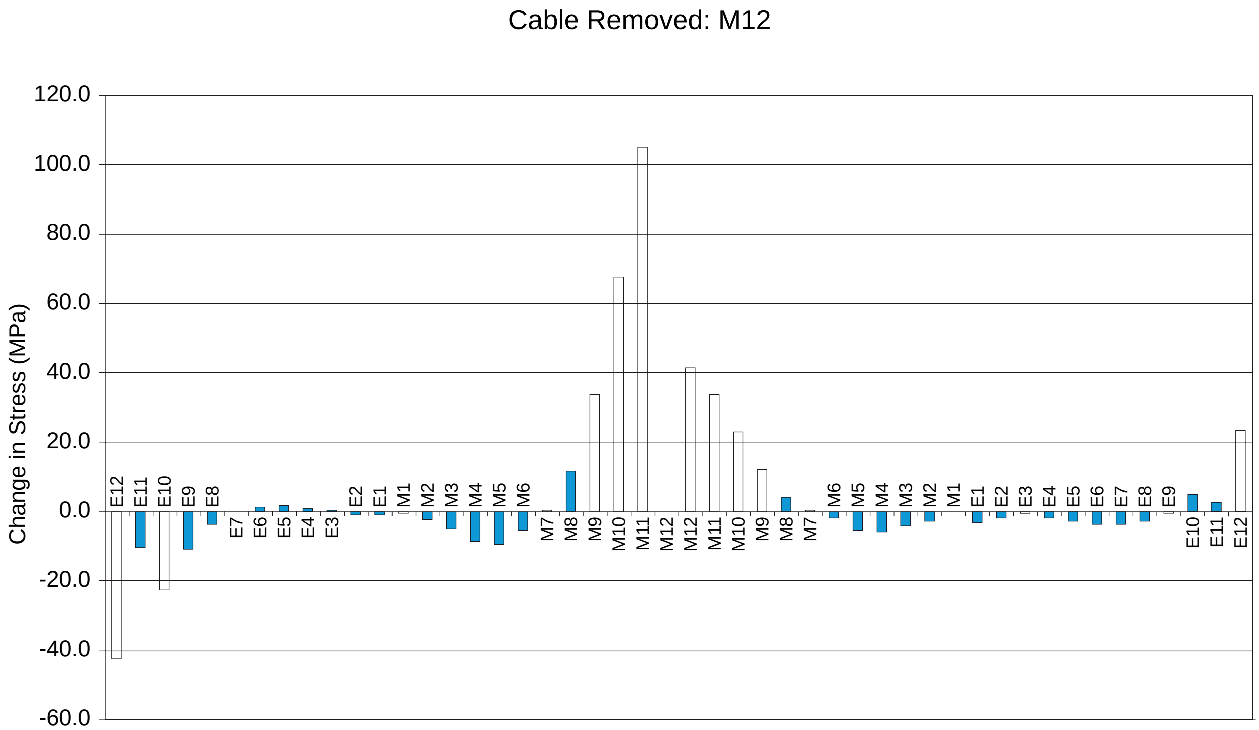

FIGURE 14. CHANGE IN STRESSES IN CABLE AND BARS AFTER REMOVAL OF A CABLE M12

bars and the risk of failure in one of the remaining bars would increase. Should another failure occur, further stress increases would occur in the remaining bars and there would be a significant risk of progressive failure of all the bars within that cable. At the same time, some tension would also be transferred to the outer tube.

In the absence of a brittle fracture of the tube (which would not be expected), it might be expected that the tube would have sufficient ductility to carry some residual load after all the bars had failed. Re-anchoring of the bars away from the fracture location and bond to the steel tube could mean that this plastic deformation was very localised to the bar fracture site, leading to only limited total extension before rupture of the tube occurred. If this available extension before rupture was small compared to the deflection of the deck at the cable anchorage in the event of complete unloading of the cable, then the stress in the pipe would not be significantly relieved as it deformed plastically and it would rupture, losing its entire load.

However, if the plastic extension available were to exceed the deck deflection during complete unloading, the pipe would not rupture and would be able to carry some residual force even after failure of all the bars within it. This extreme aspect was investigated by calculation, assuming upper-bound bond values for the concrete-tube interface, and the available extension of the tube was found to be sufficient to avoid rupture and to lock in some residual cable force. There was however the possibility that the tube anchorage itself could fail if the tube picked up too much load; there was significant acoustic activity detected in many of the anchorages.

A greater concern was that there might be no transfer of force from bars to tubes through bond in the event of bar failure (because of weak or incomplete grouting), but instead the bars could progressively fail with force transferred to the tubes at the end anchorages only. Analysis with this assumption suggested that the force increase in the tubes, if all bars failed, would be insufficient to cause the tube to yield. Thus, although the tube would continue to carry some load, there would be little obvious visual sign of distress in the outer tubes, which would remain taut, albeit with some increased sag. This situation would be potentially very dangerous because a further cable could then fail as a result of the load shed; this could lead to collapse of the entire deck. This scenario could however be detected by either vibration testing of the cables (the natural frequency would drop) or by level survey along the edge beam (a local dip centred on the broken cable would be expected).

Residual force in the tube was not considered in the analysis, which gave additional comfort that the structure would not fail if all the bars within the cable failed. Some residual force in the tube would be likely in the event of bar failures, other than in the unlikely event of an anchorage failure.

Cables

Figure 12 shows the stresses in the cables (bars/tubes) prior to a cable removal. This showed that the ULS stress limit of 67% UTS is still marginally exceeded in the bars at four locations under this reduced load combination. The stresses in the tubes are generally well below their yield stress of 360 N/mm2.

The analysis showed that if one of the cables failed, the forces in the adjacent cables typically increase by around 25%. Cables were checked at ULS comparing the combined load effects from the model with a stress limit of 67% UTS for the bars and a yield stress of 360 N/mm2 for the tubes.

Figure 13 shows the stresses in the cables when cable M12 is removed. Figure 14 shows how the cable forces change when cable M12 is removed, giving a marked increase around the area where the cable is removed and a decrease in the corresponding side span cables. The remaining cables are almost unaffected. Similar behaviour occurred for all the other removal cases. Although the bar stresses were found to reach high values in this accidental situation, they were less severe than those in service under full design load factors appropriate to the 120 year design life. In all cases, the stresses in bars remained below 80% UTS. This, together with the assumption of neglecting the residual strength in the pipe of a cable where all bars had failed, gave some confidence in the structure's ability to withstand the loss of a cable, providing there was to be swift intervention once this has occurred.

Deck and towers

The deck and towers were found to be adequate in the event of a stay failure; there would be some yielding, redistribution of moments and significant cracking, but the deck would not collapse.

6. The case for wholesale cable replacement

Assessment had suggested that the structure could accommodate a cable failure without collapse of the structure occurring, provided that:

- Swift action was taken to restrict traffic over the bridge
- Swift action was taken to replace the cable
- No significant deterioration existed in the cables adjacent to the cable which had failed.

This led to the conclusion that the bridge did not need to be closed to traffic immediately but the latter point proved to be the critical one. There remained concern that the initial condition of the stays was unknown and that significant pre-existing damage could lead to progressive collapse. This made it impossible to quantify the actual consequences of a complete cable failure.

The as-built assessment showed that fatigue in the couplers was a potential problem limiting design life to less than 120 years. Static SLS stresses were also found to be too high. Vibration of the cables observed during site visits added further stresses to the cables that were difficult to quantify but would also add to the fatigue damage. Coupler failures were known to be a reality. From the 13 cables that had so far been replaced, five out of 172 couplers were found to be broken; 3% of the total.

It was clear that wholesale cable replacements would be needed within the design life and that piecemeal cable replacements would also be necessitated by any bar breaks detected, as had happened previously. Replacements could be managed in this reactive way but it would be uneconomic to keep remobilising construction teams, would lead to repeated disruption to traffic and the risk of overall collapse. As a result, in 2006 it was decided to replace the remaining cables on the structure due to the uncertainty of the cable conditions and the high static and fatigue load usage.

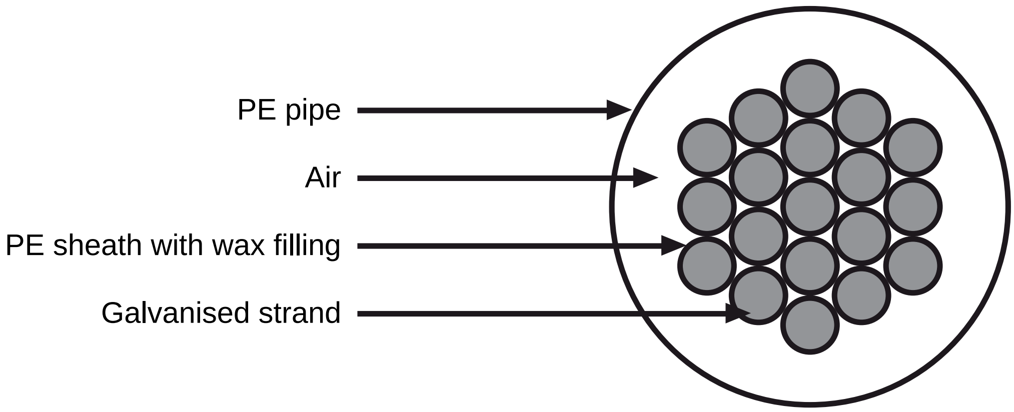

FIGURE 15. ILLUSTRATION OF NEW CABLE SYSTEM WITH MULTI-LAYER PROTECTION

7. Cable replacement

In 2006, Atkins produced a detailed design and workmanship specification for the cable replacement scheme, comprising replacement of all remaining 117 cables. Atkins other responsibilities were to perform the tender assessment, check the permanent and temporary works designs and to audit construction. UEM Construction, together with specialist contractor Freyssinet, were awarded the contract to design and install the new system including all the necessary temporary works.

7.1 Specification for new cable system

The new cable tender specification was intended to address many of the problems which were encountered with the existing cables. The specification therefore represented current best practice and was strongly informed by Eurocode BS EN 1993-1-11 and reference 6. The resulting specification formed the basis for the UK National Annex to BS EN 1993-1-11.

New cables

The specified cable system comprised uncoupled parallel strands. These have excellent fatigue performance in stark contrast to the existing coupled bar system. A multi-layer corrosion protection system was specified as shown in Figure 15. The strands themselves are protected by galvanising and are individually encased in a wax-filled high density polyethylene sheath. The group of cables is then protected within an outer high density polyethylene sheath filled with dry air. There is no grout filling material within this outer duct unlike in the original stay system.

Recent experience has shown that grout may not be as effective a form of corrosion protection as once assumed because the grout tends to crack, reducing the effectiveness of the protection, and also causing fretting in the strands.

A major benefit of the new specified system is that each strand within the cables is individually replaceable, which would minimise the amount of work required in any future replacements. The existing stay system is not replaceable as described in section 3 and thus the temporary works needed to replace each cable was extensive as discussed in section 7.3.

Vibration of the existing stays was also considered to be a potential problem. Vibrations arose because of the low natural frequencies of vibration and low inherent damping. To mitigate the risk of vibration problems with the new stays, the specification gave a requirement for the provision of spiral ridged cable sheathing to counter rain and wind induced vibrations and also internal replaceable dampers to control stay vibration amplitudes. Damping to give a logarithmic decrement of at least 4% at all vibration amplitudes was specified for all cables.

The possibility of wind-induced and traffic-induced vibrations also was specified to be considered in the design by means of a dynamic analysis. Dynamic analysis was specified to be carried out on the basis of the vehicles defined in Table 1 (from traffic data obtained on the bridge in 2005), traversing the bridge with a design speed between zero and 60 miles per hour. The weights of 2, 3, 4 and 5 axle vehicles were be assumed to be as given in Table 11 of BS 5400:Part 10.

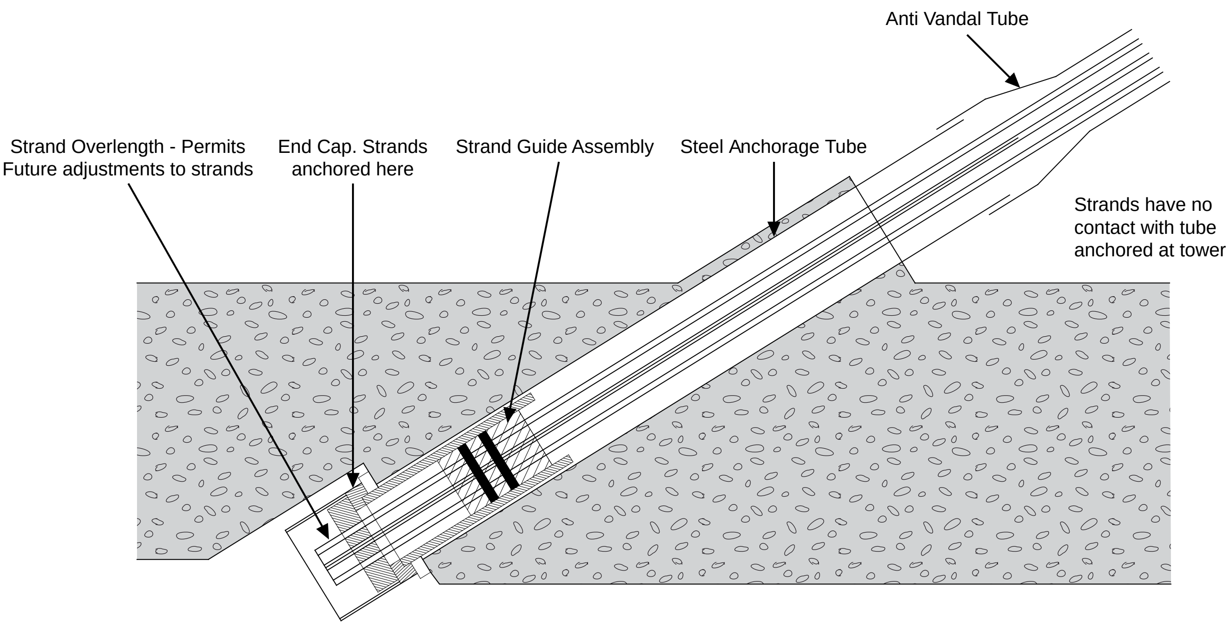

FIGURE 16. SCHEMATIC NEW ANCHORAGE DETAIL (DECK ANCHORAGE SHOWN; TOWER ANCHORAGE SIMILAR)

STRUCTURES

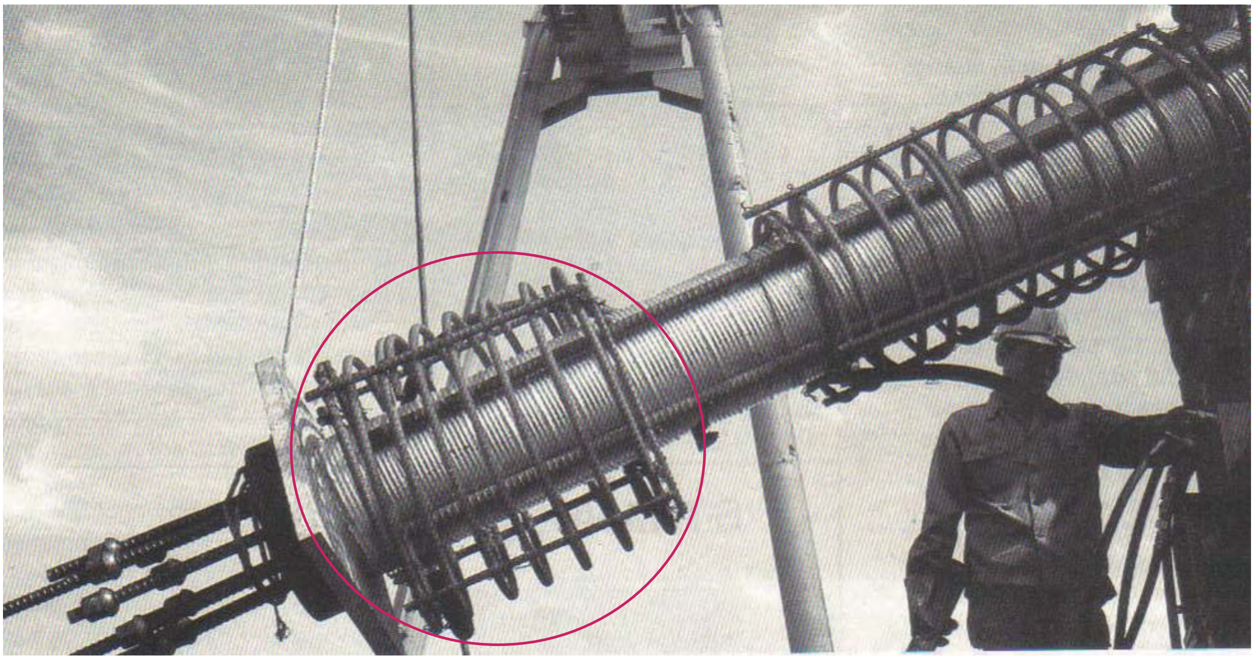

FIGURE 17. EXISTING ANCHORAGE DURING INITIAL CONSTRUCTION SHOWING BURST-ING REINFORCEMENT TO BE RETAINED

FIGURE 18. TYPICAL TEMPORARY WORKS RE-QUIRED TO REPLACE A SET OF SIMILAR CABLES

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TABLE 1. TRAFFIC DATA FOR THE BRIDGE

Vehicle type	Number vehicles /year (in 2005)
Motor cycles	5,027,071
Cars	14,340,146
4WD/van	1,512,573
2 axles bus/lorry	687,279
3 axles	55,920
4 axles	94,111
5 axles	25,710

In addition to the structural requirements above, for users comfort and safety, the amplitude of cable stay vibration was specified to be limited such that with a moderate wind velocity of 15 m/s the amplitude of cable stay vibration should not exceed 100 mm.

New anchorages

The essential features of the new anchorages were that they should be replaceable and they should provide angular tolerance at strand entry to minimise bending stresses induced by cable misalignment and vibration. Both of these features were absent in the original stay system. Detailed requirements for the mechanical performance and corrosion protection system as specified on Penang Bridge have now been included in the UK National Annex to BS EN 1993-1-11. A typical new anchorage is shown in Figure 16.

7.2 Loading conditions for replacement

It was essential during construction that the load in the existing stays was not allowed to exceed their values in the permanent situation since their actual condition, and hence resistance, was unknown. The outer lane adjacent to the stays was permitted to be closed to traffic but the remaining two 3.65m wide traffic lanes had to remain open to traffic at all times. The trafficked lanes and the construction area had to be separated by a continuous concrete barrier. During construction, abnormal loading was restricted to the equivalent of 30 units of HB without prior agreement. Under these conditions, it was necessary to ensure that the construction loading in the site lane was limited such that the net loading did not exceed that in the permanent condition.

The new cables had to be designed to comply with the stress limits given in BS EN 1993-1-11 under the live loading given in section 4.1. The cables needed to be replaceable without restrictions to the traffic and needed to be adequate in the event of accidental removal of any one cable, including the dynamic effect of severance. Cables were required to be replaceable strand by strand or as an entire cable but the latter case would require additional temporary stays to support the deck since the deck itself had inadequate serviceability performance with a stay removed.

7.3 Replacement methodology

The replacement scheme was made very difficult for two principal reasons. First the existing anchorages were concreted into the deck edge beams and had to be cored out to remove them. Second, the deck could not sustain removal of a cable so needed to be supported by temporary stays during each replacement. Coring of the existing anchorages in particular was a very precise activity because it was necessary to leave the existing bursting reinforcement shown in Figure 17 intact; it was required to prevent bursting once the new anchorage was installed. The core therefore had to pass inside this reinforcement.

The details of the replacement scheme during construction will be the subject of a second paper. The typical extent of temporary works required to replace on set of cables is shown in Figure 18. Further broken couplers were discovered when the existing cables were removed.

7.4 Checking during construction

The bridge was checked globally at every stage of construction. There were also however a number of local checks necessitated by the temporary cable anchorage steelwork. Because of their greater economy compared to current British Standards, the Eurocodes were used extensively by both designers and checkers to verify that the existing structure was adequate during the replacement of the cables. Examples included:

- Verifying bearing stresses between deck concrete and temporary steelwork
- Verifying shear stresses in the hollow pylon concrete walls caused by the forces imparted by temporary steelwork anchoring the temporary cables. The strut and tie rules of clause 6.5 of BS EN 1992-1-1:2004 were used
- Designing temporary steelwork for high combined moment and shear. The rules in clause 7 of BS EN 1993-1-57 were used.

8. Conclusion

This paper has summarised the problems encountered with threaded bar systems and in particular the severe problems encountered when defects are suspected when cables have not be designed to be either inspected or replaced. In the case of Penang Bridge, the combination of high static cable stresses, observed vibration, poor fatigue classification and detected bar breaks in conjunction with a lack of inspectability and replaceability led to the need for wholesale cable replacement. This decision was the most economic in terms of construction and traffic disruption; piecemeal reactive replacement would have been expensive and potentially caused ongoing traffic delays for years.

This project reinforces the need to be very cautious when using threaded bar couplers in applications with fatigue loads and also the need to design for maintenance and replacement. It also illustrates the effectiveness of suitably chosen acoustic monitoring systems for detecting bar breaks.

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Lateral buckling of steel plate girders for bridges with flexible lateral restraints or torsional restraints

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Abstract

Since publication of BS 5400:Part 3:2000, the design check of paired plate girders during erection has become more onerous in the UK. This has led to increases in top flange size or the provision of plan bracing just for the erection condition where previously neither modification to the permanent design was likely to be needed. This change has been brought about by the addition of two main features in BS 5400:Part 3:2000; a change to the mode of buckling considered in deriving the girder slenderness and the addition of more conservative buckling curves when the effective length for buckling differs from the half wavelength of buckling. The latter change was incorporated because of concerns that the imperfection over the half wavelength was more relevant than that over the effective length which is implicit in traditional strut curves. BS EN 1993 Part 1-1 requires no such reduction in resistance and the work in this paper was prompted by a proposal to modify the rules of BS EN 1993-2 for bridges in the UK's National Annex to be more like those in BS 5400:Part 3. The authors believed this to be unnecessary.

This paper investigates buckling cases where the effective length for buckling is shorter than the half wavelength of buckling and demonstrates that the series of correction curves use in BS 5400:Part 3:2000 are unnecessary and that the BS EN 1993-1-1 method is satisfactory and slightly conservative. The paper also outlines the design process to BS EN 1993 using both elastic critical buckling analysis and non-linear analysis. The case studies considered are a simple pin-ended strut with intermediate restraints, a pair of braced girders prior to hardening of the deck slab and a half-through deck with discrete U-frame restraints. For the latter two cases, the results predicted by BS 5400:Part 3 and BS EN 1993-1-1 are compared with the results of non-linear finite element analyses.

1. Introduction

An important part of the design of a steel concrete composite bridge is the stability check of the girders during construction of the concrete slab and prior to it hardening, whereupon it provides continuous restraint to the top flange. This may be a critical check as the girders will often be most susceptible to lateral torsional buckling (LTB) failure when the deck slab is being poured. Beams are normally braced in pairs with discrete torsional restraints, often in the form of X bracing or K bracing (as shown in Figure 1), but for shallower girders single horizontal channels connecting the beams at mid-height is an economic, but less rigid, alternative.

Paired girders with torsional bracing generally fail by rotation of the braced pair over a span length as shown in Figure 2. With widely spaced torsional bracing, buckling of the compression flange between bracing points is also possible. It was once thought that torsional bracing was effective in limiting failure to occur by buckling of the compression flange between restraints but this is now known not to be so and is reflected in the calculation method given in BS 5400:Part 3:2000[1].

FIGURE 1. BRACED BEAMS IN PAIRS PRIOR TO CONCRETING

The previous incorrect approach however allowed girders to be constructed safely for many years, probably due to incidental bracing arising from frictional restraint of the formwork and because of partial factors used in design. The mode shown in Figure 2 is however prevented by adding plan bracing to the compression flange (which is effectively provided by the deck slab once it hardens) and the latter mode (buckling of the flange between bracings) then occurs. If the check of the paired beams during concreting suggests inadequacy, either the compression flange has to be increased in size or plan bracing added.

Plan bracing is not a popular choice with contractors in the UK. If the bracing is placed above the top flange for incorporation within the slab, it interferes with reinforcement fixing and the permanent formwork. If it is placed to the underside of the top flange, it presents both a long term maintenance liability and a short term health and safety hazard during its erection. It is often therefore preferred to increase the width of the top flange or provide more discrete torsional bracing.

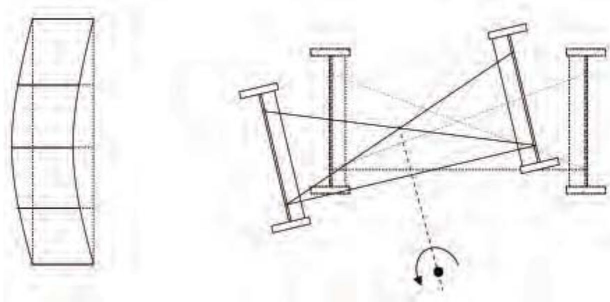

FIGURE 2. BUCKLING OF PAIRED BEAMS PRIOR TO CONCRETE HARDENING

The calculation of buckling resistance for the construction condition is currently both lengthy and conservative to BS 5400:Part 3:2000. This has the consequence that frequently the check is not carried out properly at the tender stage of a project. When the check is subsequently carried out at the detailed design stage, it is often found to require additional bracing or changes to plate thickness. One of the reasons for this is that the current BS 5400:Part 3 method is too conservative.

The design method for beams with discrete torsional restraints (the construction condition above) in BS 5400:Part 3:2000 (and BD 13/06[2]) is very conservative for a number of reasons as follows:

(i) The use of multiple strength-slenderness curves for different l_e/l_w ratios, which take the imperfection appropriate to the half wavelength of buckling, lw, (typically the span length) and apply it to the shorter effective length, l_e, is incorrect and this is demonstrated in the remainder of this paper.

(ii) The calculation of effective length for the true buckling mode is simplified and conservative. To overcome this, an elastic critical buckling analysis can be performed to determine the elastic critical buckling moment and hence slenderness. This technique is used later in this paper.

(iii) The curves provided to relate strength reduction factor to slenderness, which are derived for strut buckling, are slightly conservative for a mode of buckling where the paired girders buckle together by a combination of opposing bending vertically and lateral bending of the flanges.

(iv) Incidental frictional restraint from formwork is ignored.

The first of these issues is studied in the remainder of this paper and it is shown that the current multiple strength-slenderness curves for different l_e/l_w ratios in BS 5400:Part 3 are incorrect and overly conservative. This conclusion applies both to the construction condition above and to beams with U-frame support to the compression flange, since both cases produce an effective length that is less than the half wavelength of buckling. Non-linear analysis is used to illustrate this conclusion.

2. Buckling curves

The buckling curves in BS 5400:Part 3:2000 are based on a pin ended strut with the half wavelength of buckling equal to the effective length of the strut. The resistance is dependent on the initial geometric imperfection assumed and the residual stresses in the section. The equivalent geometric imperfection implicit in these equations is not constant but is slenderness-dependent (and therefore a function of effective length) in order to produce a good fit with test results.

The buckling curves in BS 5400:Part 3:2000 for beams with intermediate restraints are modified based on the ratio between the effective length, l_e, and the half wavelength

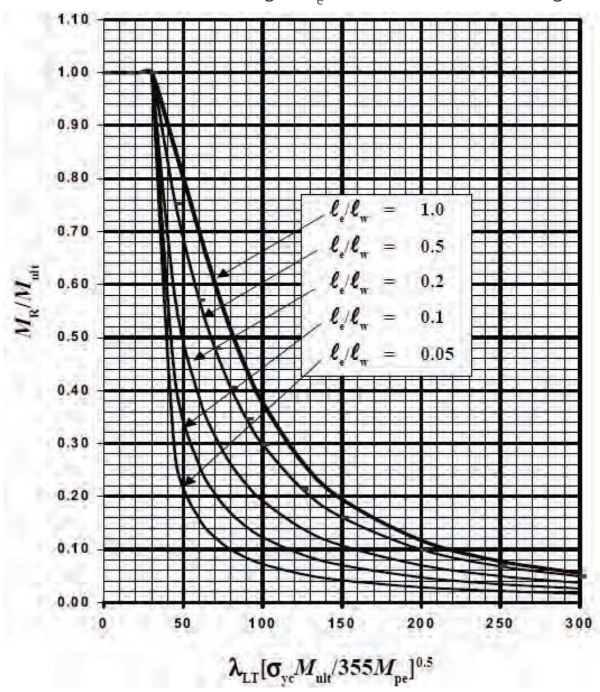

FIGURE 3. BD 13/06 BUCKLING CURVES (FOR WELDED MEMBERS)

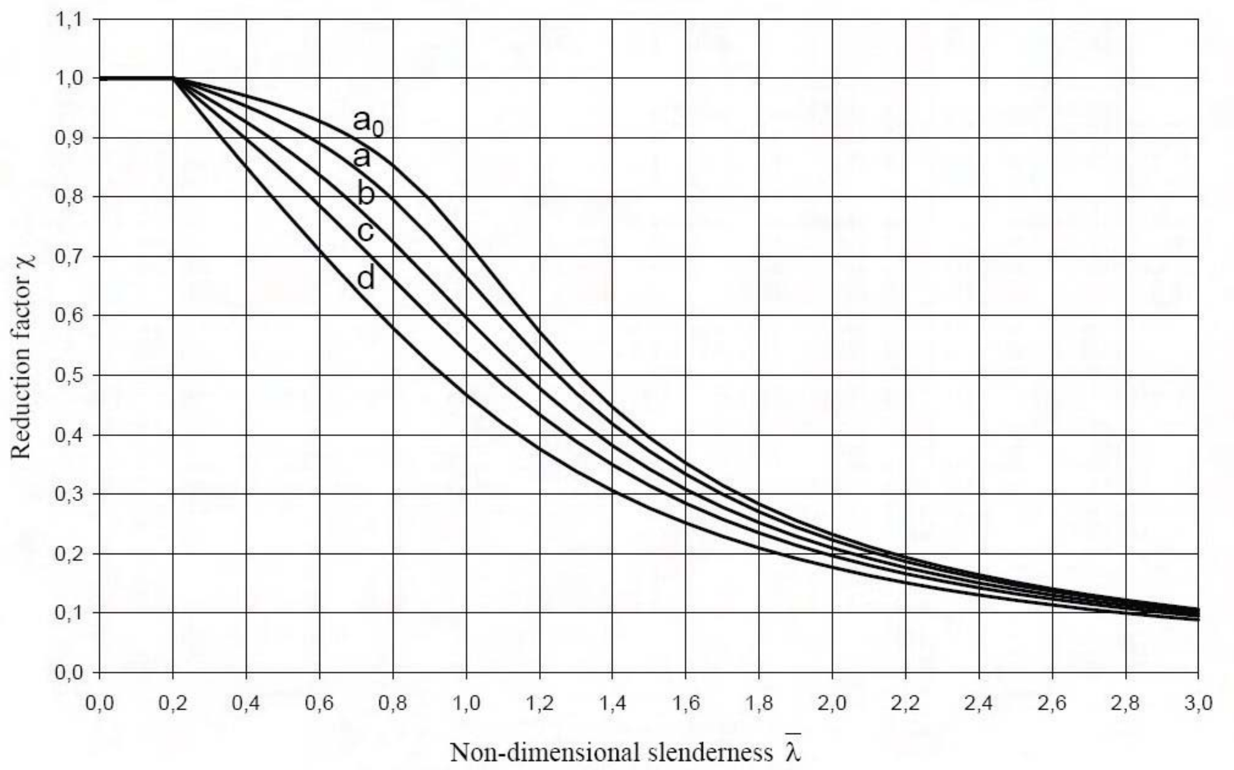

FIGURE 4. BS EN 1993-1-1 BUCKLING CURVES (A – D REFER TO THE FABRICATION METHOD)

of buckling, l_w. This was considered necessary in order to factor up the imperfection to be used in the buckling curve from that appropriate to the effective length to that appropriate to the half wavelength of buckling. BS 5400:Part 3 gives rules for calculating l_e where lateral restraints to the compression flange, torsional restraints, discrete U-frame restraints or restraint from the bridge deck are provided. Where lateral restraints to the compression flange are fully effective, l_e is taken as the span between restraints, and l_w will also be equal to this span. Where the restraints are not fully effective, l_e may be shorter than l. This is usually the case for beams relying on U-frame restraint or for paired beams with torsional bracing like that shown in Figure 1. BS 5400:Part 3 states that l_w is determined by taking L/l_w as the next integer below L/l_e where L is the span of the beam between supports. The rules in BS 5400:Part 3 are modified in BD 13/06 for use on Highways Agency projects. BD 13/06 introduces further conservatism for beams with torsional restraints by requiring lw to be taken as the full span. The buckling curves for welded members in BD13/06 are shown in Figure 3.

The buckling curves in BS EN 1993-1-1[3] (see Figure 4) have the same basis as, and are effectively the same as, those in BS 5400, but no adjustment is made for the ratio l_e/l. A non-dimensional presentation of slenderness is also used.

3. Buckling cases investigated

To investigate the validity of the BS 5400:Part 3 approach for beams with lateral restraints, a number of situations were considered. First, to clarify the principles involved, a pin-ended strut with springs providing lateral restraints was considered. The behaviour of this simple model, axially loaded and with an initial geometric imperfection, was compared to the behaviour of an equivalent strut with no lateral restraints but the same elastic critical buckling load. Second, two practical situations where a reduction in capacity to BS 5400:Part 3 would be required due to differences between the half wavelength of buckling and the effective length were considered. These were a typical half-through bridge with U-frames and a steel-concrete composite bridge during construction.

3.1 Simple strut model

Figure 5 illustrates two struts with identical elastic critical buckling loads; one with flexible intermediate transverse restraints and the other without. BS 5400:Part 3 and BD 13/06 would predict the case with intermediate restraints to have the lower ultimate resistance (as distinct from elastic critical buckling load) because it has a ratio of $l_e/l_w < 1.0$. The comparison of true ultimate strength in the two cases was examined.

A 10 m long strut in S355 steel with springs at 1m centres as shown in Figure 6 was considered. The strut was pinned at either end.

The cross section was square with 100 mm sides and the springs had a stiffness of 10 kNm[-1].

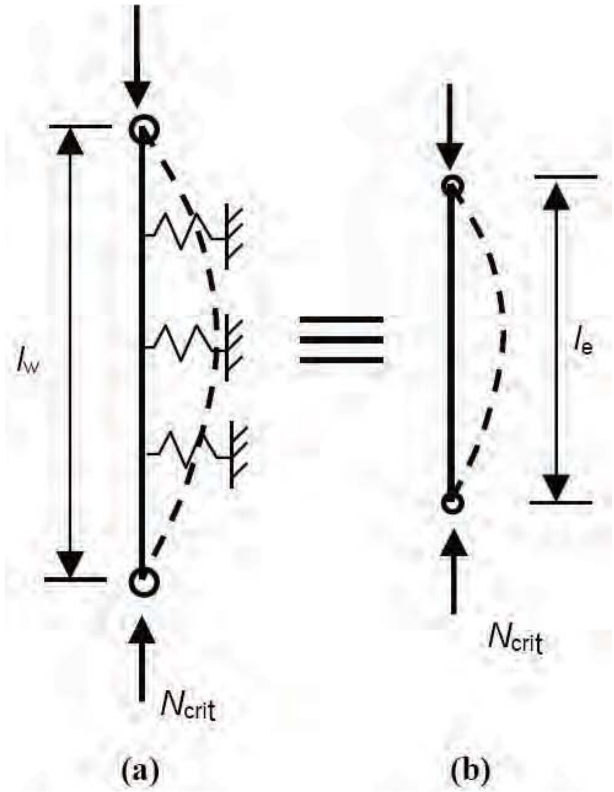

FIGURE 5. EQUIVALENCE OF STRUTS WITH AND WITHOUT FLEXIBLE RESTRAINTS IN TERMS OF ELASTIC CRITICAL BUCKLING LOAD

Without any lateral restraints, the elastic critical buckling load, Ncrit, of this strut is 173 kN. The presence of the springs increased this to 274 kN and the buckling mode remained in a single half-wavelength between end supports. From the Euler strut buckling equation, the effective strut length, l_e, to give this same value of N_{crit} with no lateral restraints is 7.94 m. A pin ended model of length 7.94 m with no springs was therefore also set up.

The two models were analysed with geometric non-linearity to obtain the axial load at which first yield occurred. An initial imperfection having the shape of the first mode of buckling was applied to the models. The deflections were scaled so that the maximum imperfection offset was equal to $l_w/250$ = 40 mm for model type (a) (Figure 5) with lateral restraints. L/250 is the imperfection recommended in BS EN 1993-1-1 for second order analysis of this particular strut geometry. Model type (b) was analysed twice, first with a maximum imperfection of $l_e/250$ = 31.8 mm and then with the same imperfection of 40 mm as used in model (a). The first case represents the Eurocode approach of using buckling curves based on equivalent geometric imperfections appropriate to the effective length and the second case represents the BS 5400 approach using an imperfection factor appropriate to the half wavelength of buckling.

TABLE 1. LOAD AT FIRST YIELD FOR MODELS (A) AND (B)

Model	Imperfection (mm)	N first yield (kN)
(a)	40.0	245
(b)	31.7	239
(b)	40.0	231

Table 1 shows the load at which the outermost fibre of the beam first yielded for each case. Model (a) represents the true resistance of the strut with intermediate restraints. It can be seen that the equivalent shorter strut without restraints represented by model (b) had a lower resistance even when the smaller imperfection based on le was used. This shows that the codified approach in BS EN 1993-1-1 is safe without the need to consider the ratio l_e/l_w.

It is easy to illustrate why model (a) produces the greatest resistance. The first order moment acting on model (b) is (N x a_0) where a_0 is the initial imperfection and N is the axial force. Where lateral restraints are present, the first order moment is lower than this because of the transverse resistance offered. A first order linear elastic analysis of model (a) gives M = (0.65 x N x a_0). In both models, the second order moment considering P-   effects can be obtained approximately by increasing the first order moment by a factor of

$$\frac{1}{1 - N/N_{crit}}$$

If a_0 is the same for both cases the smaller first order moment in the case with lateral restraints gives rise to a smaller second order moment and hence a higher ultimate buckling load. Using an imperfection based on the effective length in model (b) still gives a conservative buckling resistance as the first order moment (0.65 x N x a_0) for the restrained strut is still less than that for the effective length strut of (N x a_0 x L_e/L_w) = (N x a_0 x 0.794). This implies that actually the curves in BS EN 1993-1-1 become more conservative for cases of beams with intermediate restraint, rather than less conservative as implied by BS 5400:Part 3.

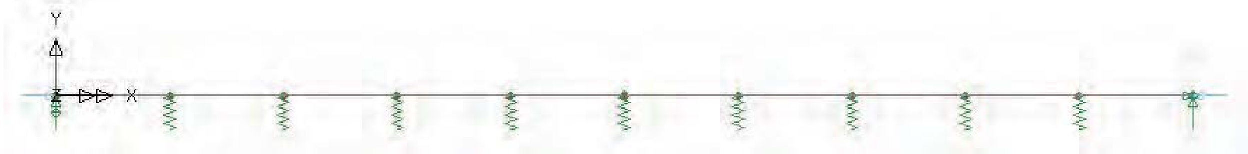

FIGURE 6. MODEL OF STRUT WITH LATERAL RESTRAINTS

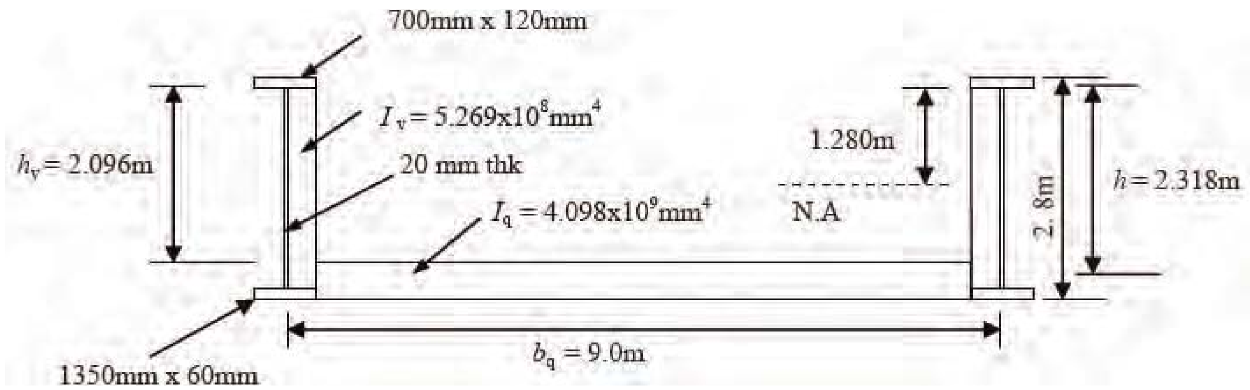

FIGURE 7. SKETCH OF MODEL REPRODUCED FROM DESIGNERS' GUIDE TO EN 1993-2

The above result can also be demonstrated more generally by solution of the governing differential equation for a curved beam on an elastic foundation with axial force, but the reader is spared the mathematics here.

3.2 Half-through bridge with U-frame restraint

A specific case of a half-through bridge in S355 steel was investigated. The bridge is simply supported with a 36 m span and cross girders at 3 m centres. The transverse web stiffeners coincide with the cross girders. The dimensions of the case considered are given in Figure 7 and is based on worked example 6.3-6 in the Designers' Guide to EN 1993-2[4].

The slender main girders are class 4 to EN 1993-1-1 (and non-compact to BS 5400:Part 3). The deck slab is composite with the cross girders but is not fixed to the main girders.

The bridge was modelled using shell elements in the finite element package LUSAS. The layout of the FE model is shown in Figure 8. For simplicity, the deck slab was not included explicitly in the model, other than in the rigidity of the cross members. To prevent relative longitudinal movement between main girders, plan bracing was added to the model at the ends and transverse supports were provided at each cross girder to prevent lateral buckling into the deck slab. Pinned vertical point supports were provided at the end of each girder and longitudinal movement was permitted at one end.

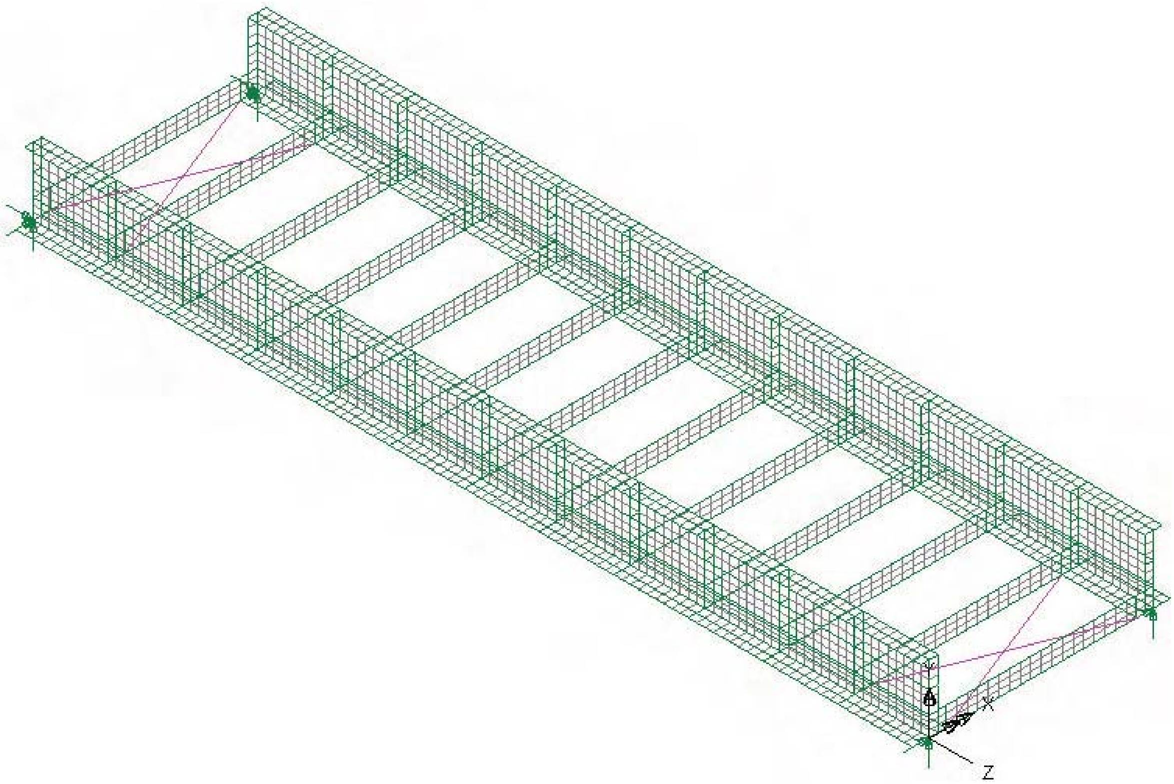

FIGURE 8. FINITE ELEMENT MODEL OF HALF-THROUGH BRIDGE

STRUCTURES

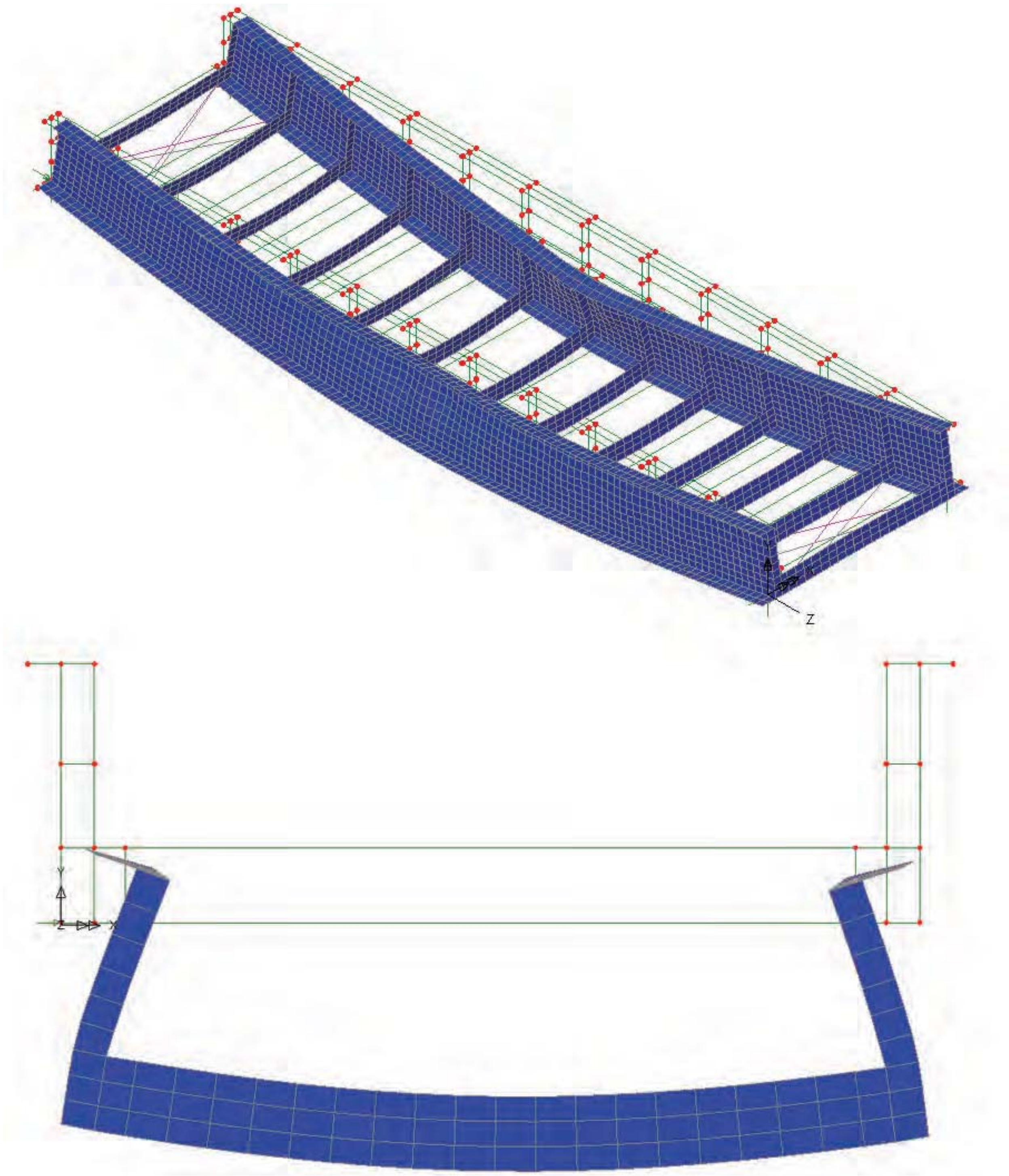

FIGURE 9. DEFLECTED SHAPE AT FAILURE

In order to prevent local failure of the model at the point supports, the stiffener and web plate thicknesses at the end of the model were increased locally. Loading was applied uniformly distributed along the top of the cross girders.

The model was first analysed linear elastically with vertical and lateral load cases to check that deflections and flexural stresses were as expected. The true resistance of the bridge to uniform vertical loading was then determined from a materially and geometrically non-linear analysis. The non-linear material properties used were based on a material model given in EN1993-1-5 Appendix C. In this model yield occurs at a stress of 335 MPa and a strain of 0.001595. To model limited strain hardening, the gradient of the stress-strain curve was then reduced from 210 GPa to 2.10 GPa up to an ultimate strain of 0.05.

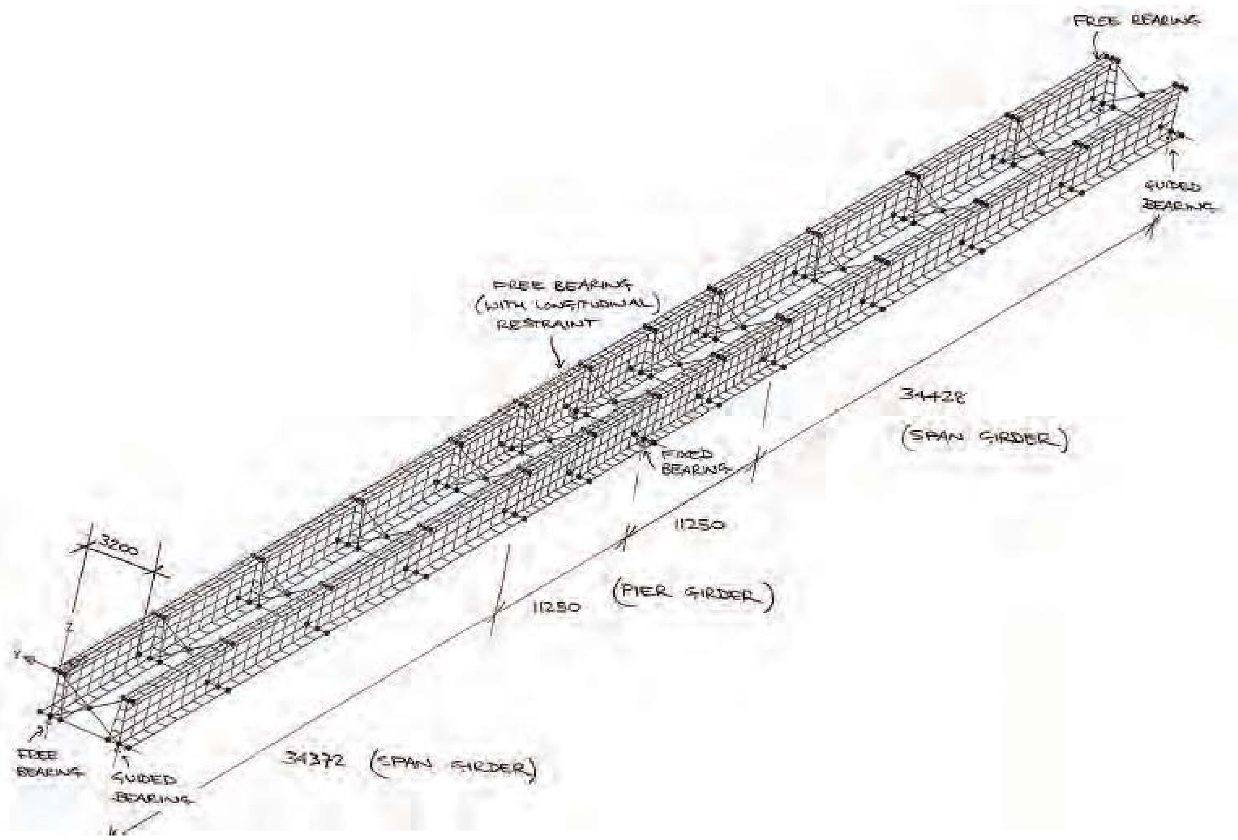

FIGURE 10. IDEALISATION OF TWO SPAN BRACED PLATE GIRDERS

Non-linear analysis

An initial deflected shape similar to the elastic buckling mode expected was generated by applying a point load to the top flanges of the main girders at midspan in a linear analysis. The deflected shape from this analysis was factored so that the inwards deflection at the start of the non-linear analysis had magnitude L/150 where L was taken as the distance between points of contraflexure in the flange. The value L/150 is taken from EN1993-1-1 Table 5.1 and accounts for allowable construction imperfections and residual stresses in the girders.

After the initial analysis to failure, shown in Figure 9, a second non-linear analysis was performed using a modified shape of initial imperfection based on a scaled version of the deflections at failure from the first analysis. The deformed shape was scaled so that the horizontal deflection at the top flange was again L/150. This gave a moment of resistance of 72340 kNm.

Hand calculations for buckling resistance in accordance with BS EN 1993-2[6] gave a buckling moment of resistance as 54360 kNm (without any partial factors). The non-linear FE analysis therefore gave 33% more resistance than the code calculations. For comparison, the elastic moment resistance of the girder was 77988 kNm and the plastic moment resistance of the girder was 86373 kNm.

In order to produce a more pronounced buckling failure, the cross bracing and stiffener spacing was increased to 6 m. The same analysis procedure as for the first model was repeated. The model failed at an ultimate applied moment of 62076 kNm. The hand calculations were repeated using the reduced stiffness of the U-frame. The new buckling moment was found to be 52252 kNm. This is 19% lower than that found in the non-linear analysis.

The analyses both show that the non-linear FE models demonstrate considerably more strength than is predicted by the method in BS EN 1993-2 using the buckling curves in BS EN 1993-1-1. The model with girders at 3 m centres gives 33% extra resistance and the model with girders at 6 m centres gives 19% extra resistance. Once again, the buckling curves of BS EN 1993-1-1 were found to be conservative and thus the BD 13/06 approach of using multiple curves to allow for the ratio l_e/l_w is unnecessary

There are several reasons why the non-linear FE model gave higher predicted strength than the calculations to EN 1993. These include:

1) The FE model shows partial plastification of the tension zone occurs, which gives extra resistance that is not accounted for in the hand calculations.
2) The strain hardening included in the material properties allows the stresses in the model to increase beyond yield (by roughly 7%).

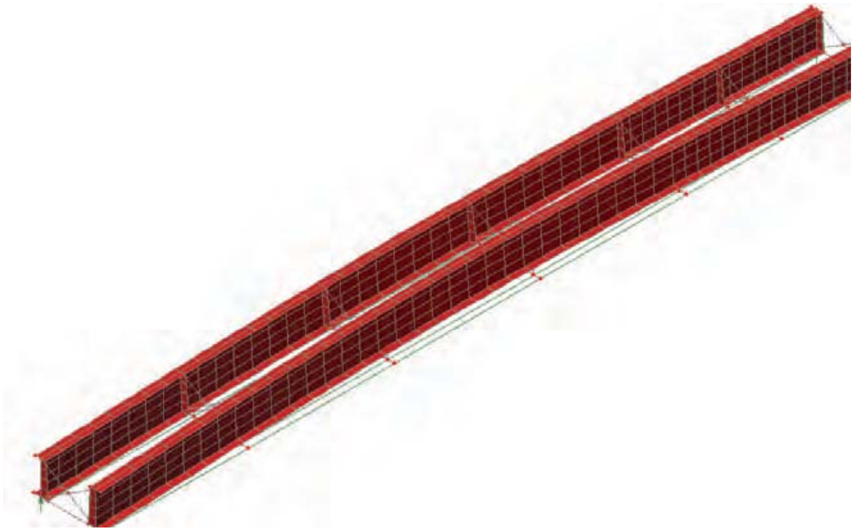

FIGURE 11. LOWEST GLOBAL MODE OF BUCKLING SINGLE SPAN BEAMS

3.3 Paired beams during construction

The composite bridge cases considered were a two span and a single span bridge with two steel plate girders braced together by cross bracing. The dimensions are representative of typical UK construction, being based on a recently constructed bridge. Figure 10 shows the geometry of the two span bridge and the FE model setup and Table 2 gives the dimensions of the girder. A uniformly distributed vertical load was applied to both girders in one span only, representing concreting of a single span.

TABLE 2. GIRDER MAKE-UP

SPAN GIRDER	Width	Depth	f_y (MPa)
Top Flange	600	40	345
Web	16	1942	355
Bottom Flange	810	33	345

Span Girder

PIER GIRDER	Width	Depth	f_y (MPa)
Top Flange	600	59	345
Web	16	1942	355
Bottom Flange	810	59	345

Pier Girder

The bridge layouts were checked for lateral torsional buckling during construction using four different approaches:

a) The standard method set out in BD 13/06 9.6.4.1.2 was followed to obtain the slenderness λ_{LT} and then the resistance moment from Figure 11

b) The alternative method permitted in clause 9.7.5 of BD13/06 was used to obtain λ_{LT} from a value of M_{cr} determined from an elastic critical buckling analysis using the FE model. The effective length was back-calculated from λ_{LT} using clause 9.7.2 and the resistance moment obtained from Figure 11 of BD 13/06

c) EN 1993-1-1 clause 6.3.2 was used to calculate the slenderness

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

(where M_{cr} was obtained from an elastic critical buckling analysis and M_y was the first yield moment)

d) Non-linear analysis

With a continuous bridge, redistribution of moment away from the span to the support is possible with a non-linear analysis when the mid-span region loses stiffness through buckling. This would make the conclusions from the comparison of ultimate load obtained for continuous spans with the code approaches in a) to c) inapplicable to simply supported spans where such redistribution could not occur. The above approaches were therefore repeated for a single span model, with the same dimensions as half the two span bridge, but with the span girder properties used throughout. For the single span bridge, load was applied to both girders over the whole span.

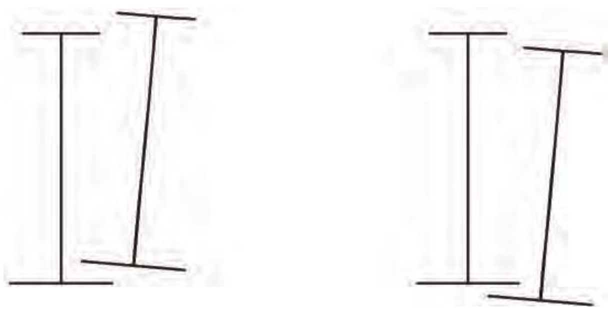

FIGURE 12. LOWEST GLOBAL MODE OF BUCKLING FOR SINGLE SPAN BEAMS – ROTATION OF CROSS-SECTION

STRUCTURES

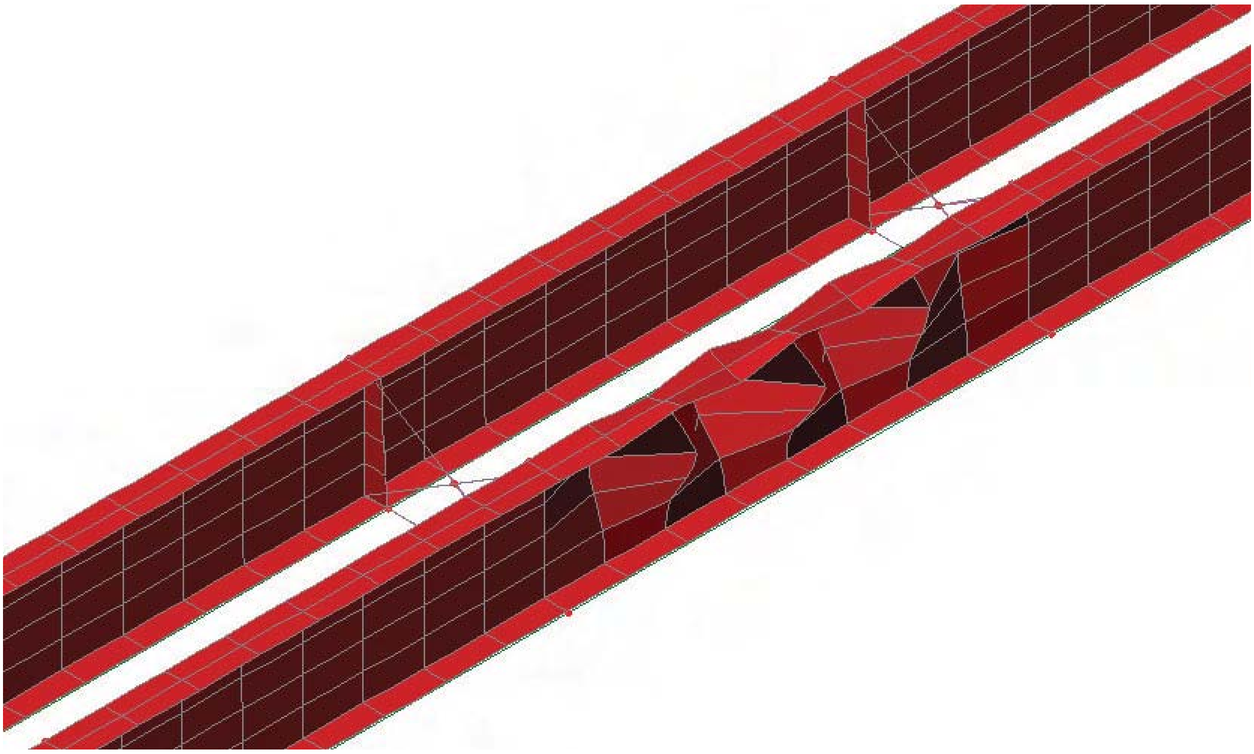

FIGURE 14. TYPICAL LOCAL
BUCKLING FOR SINGLE SPAN BEAMS

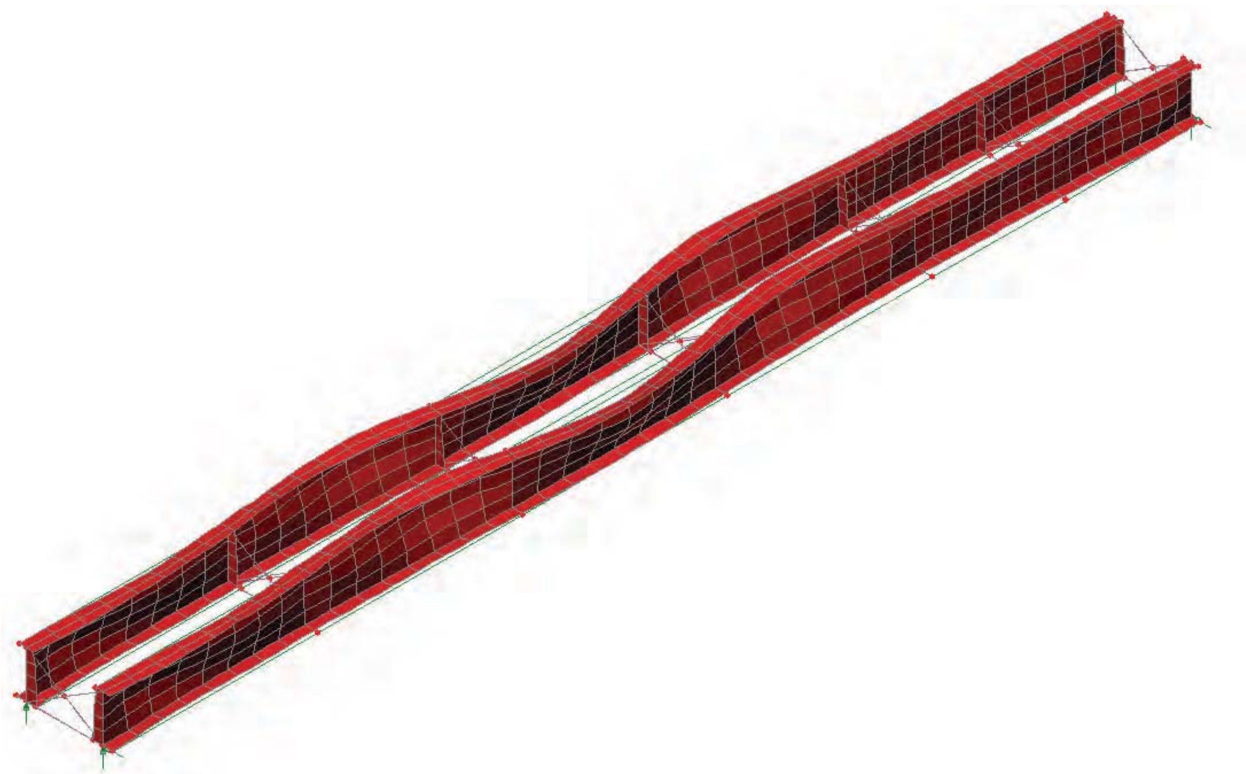

FIGURE 13. SECOND LOWEST GLOBAL MODE
OF BUCKLING FOR SINGLE SPAN BEAMS

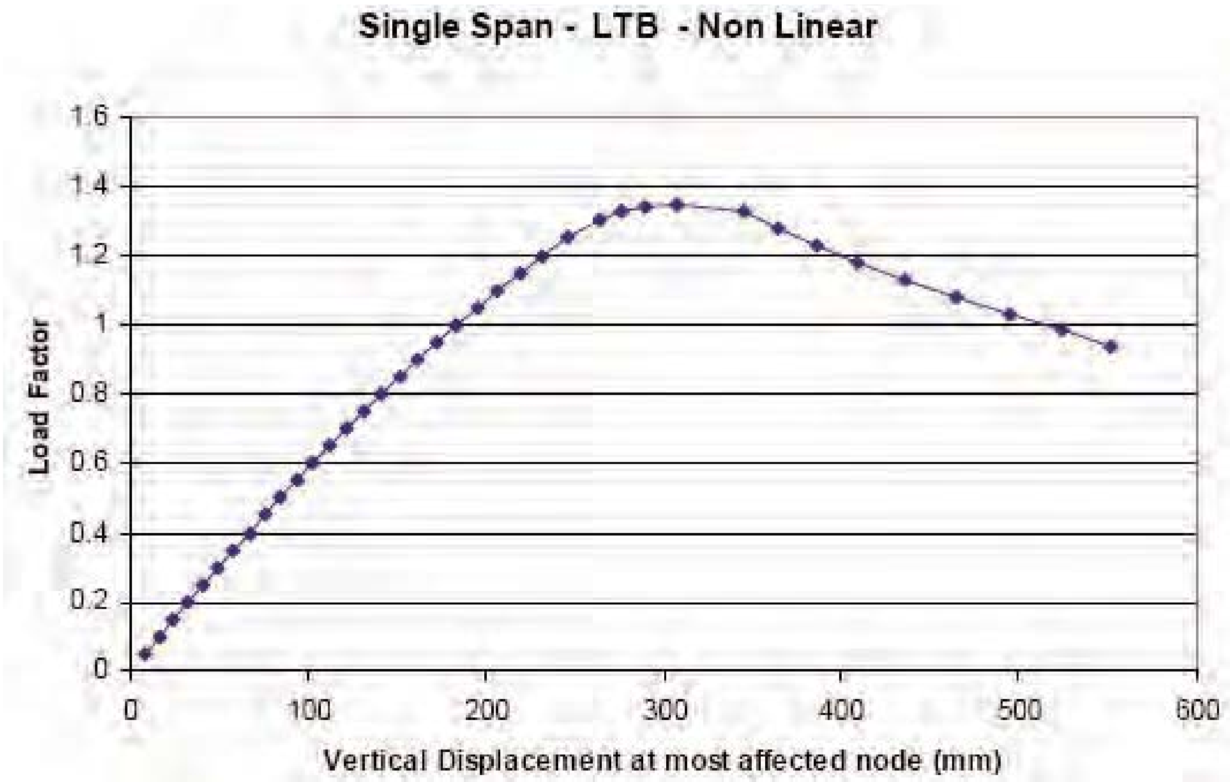

FIGURE 15. LOAD-DEFLECTION CURVE FOR NON-LINEAR ANALYSIS OF SINGLE SPAN MODEL

Elastic critical buckling analysis

The lowest global mode of buckling for the one span model, corresponding to the attainment of M_{cr}, is shown in Figures 11 and 12. The girder pair is seen to rotate together over the whole span as illustrated in Figure 2. The second lowest mode is shown in Figure 13 and corresponds to lateral buckling of the compression flange between braces. At lower load factors than either of these modes, a number of local buckling modes such as that shown in Figure 14 were found. These typically correspond to buckling of the top of the web plate in compression or potentially to torsional buckling of the top flange and may be ignored for the purposes of determining Mcr; these buckling effects are considered in the section properties in codified approaches to BS 5400:Part 3 and BS EN 1993. The values of M_{cr} determined were used to derive a total resistance moment for cases b) and c).

TABLE 3. BENDING RESISTANCES OBTAINED FOR THE DIFFERENT METHODS

Calculation Method	Design Resistance Moment at mid-span (Including $_{m}$ and $_{f3}$ on the resistance side) (kNm)	
	Two Span	Single Span
a) BD 13/06 9.6.4.1.2	6045 (l_e = 17.0m)	5260 (l_e = 18.9m)
b) BD 13/06 9.7.5 (with FE)	7330 (l_e = 13.5m)	6085 (l_e = 16.0m)
c) EN 1993-1-1 6.3.2 (with FE)	9231	7470
d) Non Linear FE (maximum M)	12000	9591
d) Non Linear FE (first yield)	9654	8170

Non-linear analysis

The same FE models for single span and two span cases were analysed considering non-linear material properties and geometry and including an initial deformation corresponding to the first global buckling mode. This was used to determine the collapse load. The magnitude of the initial deflection was taken as L/150 as required in EN 1993-1-1. The maximum moment reached and the moment at which first yield occurred were noted. Failure occurred by rotation of the braced pair over a span in the same shape as the elastic buckling mode of Figure 11. Figure 15 shows the load-deflection curve up to failure for the single span model. It indicates that the failure is reasonably ductile.

The design resistance moments for the two span and single span models are given in Table 3. The resistances are all factored up by the partial material factors $_{m}$ and $_{f3}$ in BD 13/06 so that they are all directly comparable and, for the two span beam non-linear cases, are based on the original elastic bending distribution along the paired beams without allowing for any moment redistribution away from the span. (This was achieved by using the load factor at collapse to scale up the original elastic moments.) The effective length used in the code calculations is also given. The ratio l_e/l_w lies between 0.3 and 0.5 and Figure 3 shows that this will give a significant reduction in strength in the BD 13/06 calculations.

The non-linear analysis gave higher resistance moments, thus showing that methods a) to c) were all conservative with the methods of a) and b) based on BD 13/06 being the most conservative. This again indicates that the BD 13/06 approach of using multiple curves to allow for the ratio l_e/l_w is unnecessary. Again, there were several reasons why the non-linear FE gave higher predicted strength than the other code methods:

1) Partial plastification of the tension zone is possible.
2) Strain hardening can occur.
3) For the two span model, redistribution of elastic moment is possible away from the span.

4. Conclusions

All the analyses demonstrate that the resistance curves and slenderness calculation used in Eurocode 3 are conservative. The analyses of the two simple strut cases show that applying the initial imperfection relevant to a length equal to the half wavelength of buckling, where this is greater than the effective length, is overly conservative. The curves in BD 13 and BS 5400 Part 3 therefore need to be revised to remove the curves below that for le/lw = 1.0. The curves for le/lw = 1.0 can always be used safely.

Non-linear analysis can be used to extract greater resistance from beams for a number of reasons which include benefit from:

- partial plastification of the tension zone in non-compact sections
- strain hardening
- moment redistribution in statically indeterminate structures

The case for including the multiple buckling curves in BS 5400: Part 3 and BD 13/06 was challenged by the authors previously and this eventually resulted in the Highways Agency and the British Standards Institutions (BSI) B/525/10 committee accepting that the lower l_e/l_w curves were not appropriate. The BSI does not intend to revise BS 5400: Part 3 but the B/525/10 committee agreed that the revision, removing the lower curves, can be published by the Steel Constructions Institute. The revision was recently published in the New Steel Construction as advisory desk not AD 326.[7]

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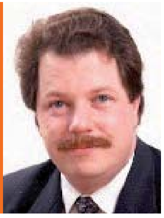
This paper will be published in the Proceeding of the ICE Bridge Engineering, March 2009

Civil Certification – The new approach to ensuring fitness for purpose for enforcement equipment in England and Wales



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Abstract

Traffic management regulations in the UK have been undergoing a transformation over the last four years. This has been caused by the realisation that the enforcement of some regulations is a safety issue (speeding and red-light running) whilst others are entirely a traffic management issue (parking, banned turns and driving in bus or HOV lanes). Principal among the changes is the transfer of certain types (not speeding or red-light-running) of traffic enforcement from the Police to appropriately authorised civil enforcement officers. In order to enable this change, the equipment to be used has to be approved by the Department of Transport (DfT).

This paper reports the process used to define the new standards for enforcement equipment, comments on the format of the new standard and finally looks to how to incorporate new technologies in the future.

1. Introduction

Recent changes in legislation in Great Britain have moved some aspects of traffic enforcement into the field of traffic management. As such, civil traffic enforcement is now not simply a tool for ensuring compliance, it has become an indispensable tool for extracting the maximum capacity out of the road network. In the coming years civil traffic enforcement will be seen as a key ITS technology along with SCOOT/SCATS and traffic surveillance systems.

In England and Wales, before a piece of equipment may be used to generate any penalty as a result of a contravention of traffic regulations, it must be certified by one of the Secretaries of State. In the past, this was solely the preserve of the Secretary of State for the Home Office. Recently, with the introduction of civil penalties for minor traffic contraventions, responsibility for certification of equipment to enforce certain contraventions has transferred to the Secretary of State for Transport. Where the regulations require that a criminal process is followed, (e.g. for speeding or red-light violation) equipment approval remains solely at the discretion of the Secretary of State for the Home Office. For a limited number of contravention types (e.g. bus lane violation), there is the option to enforce using either criminal or civil powers. In these cases, it is most likely that the civil powers will be used for the overwhelming majority of cases.

To add further to the confusion caused by the apparent duplication of legislation, the terminology differs between the criminal and civil processes. Table 1 provides a convenient look up between the terminology used in these two fields.

TABLE 1. LEGISLATIVE TERMINOLOGY

Criminal Term	Civil Term	Meaning
Type Approval	Certification	The approval given by the relevant Secretary of State to allow the equipment to be used on street.
Offence	Contravention	The event that gives rise to a penalty
Fixed Penalty Offer (FPO)	Penalty Charge Notice (PCN)	The document sent to the road user to notify them that a penalty is due. It should be noted that in the criminal domain there are alternative penalty paths that can be employed.

This paper describes the features of the new civil certification standard, its requirements and the way in which the standard was defined. We acknowledge the roles played by the following groups: The Department for Transport, who chaired the TWG, Transport for London (who for historical reasons chaired the Steering Committee) and the Vehicle Certification Agency (who are acting as executive agents of the DfT for the certification of enforcement equipment).

2. Developing the new standard

With the publication of the Traffic Management Act 2004, the UK Government introduced a new class of traffic enforcement to the majority of England and Wales. This built on the experience that London had under the Transport Act 2000 and its own location specific regulation in the form of the London Local Authorities and Transport for London Act 2003. The effect of this regulatory change was to move certain traffic contraventions from the criminal domain (i.e. enforced by the police) into the civil domain (i.e. enforced by an officer of the relevant local authority). More relevant for this paper, it also moved responsibility for the approval of some enforcement equipment from the Secretary of State for the Home Office to the Secretary of State for Transport. During the development of Transport for London's Digital Traffic Enforcement System project, it became clear that whilst the civil certification route was a theoretical possibility, it was not clear how approval could be obtained.

At this point, TfL (supported by the author) approached the DfT with a proposal to generate a coherent standard describing the technical requirements for enforcement equipment. DfT agreed that there was need for such a document and agreed to work with TfL to produce the certification standard This lead to an initial technical meeting, hosted by the Institution of Engineering and Technology (IET) at their Savoy Place headquarters, in mid October 2006. After the initial public event a steering committee was appointed. This committee agreed the format and scope of the standard and appointed a TWG to complete the detailed technical work. Both the steering committee and the TWG were formed from a cross-section of stakeholders drawn from the four main (legislative/ policy bodies, enforcing authorities, manufacturers and certifiers) stakeholders. In addition to the above, the steering committee had representation from the IET who provided expertise related to the publication of standards.

3. The new standard

Starting with a fundamental model of an enforcement system (shown in Figure 1), the technical working committee defined a number of key parameters that needed to be met in order to demonstrate that all components of any assessed system met the required standard. Initially, the TWG attempted to write two separate documents: one for fully automatic (or unattended systems as they are known in the document) systems and another for systems that are operated by a human operator (attended systems). It quickly became clear that there were significant areas of commonality between the two types of system that meant it would be better to combine the two into a single specification.

One of the principal areas of discussion within the TWG was the issue of what was acceptable image quality. Some members of the group (including the author) asserted that it was possible to set semi-objective standards for image quality similar to the "Double Stimulus Continuous Quality Scale (DSCQS) method" described in ITU-R test standard ITU-R BT.500-10. After a further investigation it was felt that the test method was too onerous and that nothing would be gained over an assessment of fitness for purpose by a competent person. Similar arguments were then applied to recording standards and data security. In a number of areas, it was possible to take requirements and test methods / limits from similar documents generated by the Home Office Scientific Development Branch for the criminal domain.

After a number of iterations, and a brief excursion into a pair of independent documents, the final version of the document has the following general format:

- Introductory chapters: describe the legal context, certification methodology and document structure
- Attended system requirements: a series of fitness for purpose requirements for systems to be operated by a human operator
- Attended systems sample fitness for purpose limits
- Unattended system requirements: a series of fitness for purpose requirements for systems to be operated by a human operator
- Unattended systems sample fitness for purpose limits

The heart of the requirements for certification is a Technical Construction File (TCF) that documents, in detail, the enforcement system to be certified. The key elements of the TCF are detailed specifications for the system and it components, test results and site drawings for all outstation equipment, listing of all software and version numbers and a detailed maintenance plan.

4. Public consultation

As with all elements of legislation (this document is specifically referenced by a new Statutory Instrument), this document had to be made available for public consultation. This was done by the DfT in early September 2007, and by the end of the consultation period in December twelve bodies had raised a total of approximately 150 comments. Most of these were straightforward to address, however, a small number of the comments caused the authors

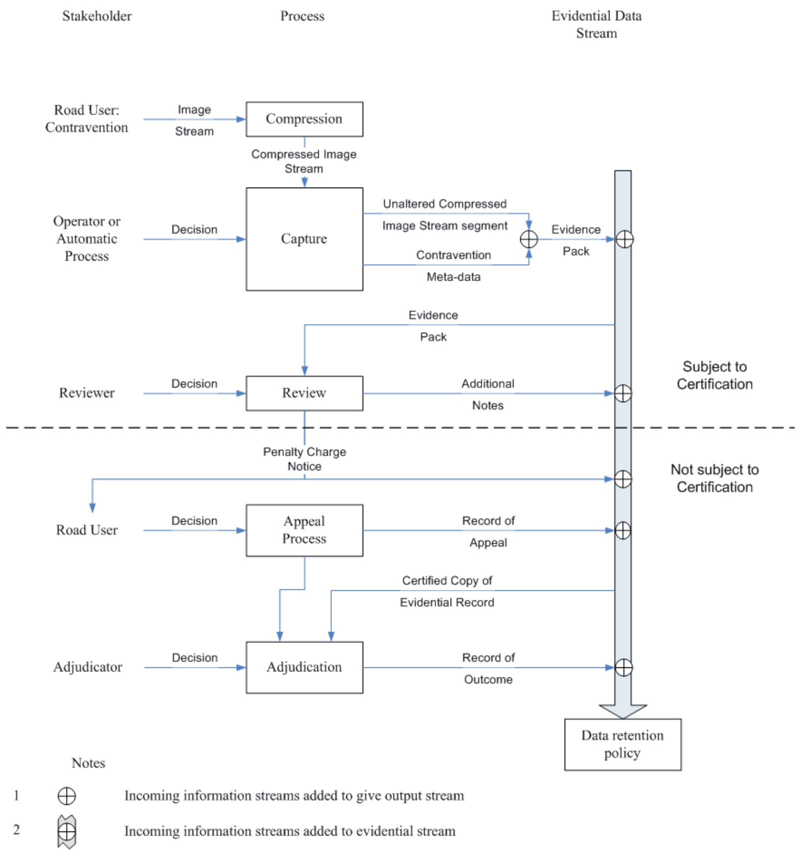

FIGURE 1. ENFORCEMENT SYSTEM DATA MODEL

to review the certification strategy. In one case a small change to methodology was made. The certification requirements[1] were finally issued in late February 2008.

5. Fresh challenges

The certification standard has been designed to be technology neutral and non-prescriptive as to method of operation, wherever possible, to enable innovation. However, new innovations are coming to the fore in enforcement technology that will blur the boundary between attended and unattended systems. The steering committee is now considering how to respond to these developments. It is not believed that a major change will be required for Version 2 of the standard.

6. Practical application

The new regulations for civil traffic enforcement require that systems currently in use for enforcement are certified by the Secretary of State for Transport. As a result, TfL had a requirement to obtain certification for their existing enforcement system. This is a large scale system consisting of the following:

- 60 CCTV operator workstations
- 1250 CCTV cameras
- 15 CCTV matrices spread around London (including the master 1024 x 512 matrix in Westminster)
- 55 static cameras
- A weight enforcement system on Tower Bridge
- A portable enforcement camera system
- A prototype vehicle mounted camera
- The existing back office with 40 operator positions

As the system has been operational for in excess of 10 years, it has proved quite difficult to provide the detailed specifications required for the TCF as most of the systems have been upgraded considerably since they were first procured 10 years ago. Similarly, it has proved a logistical challenge to provide all of the site information for the outstation equipment. In this case it is the sheer number of CCTV cameras that has proved the most difficult to manage. At the time of writing, certification has not been granted for the TfL legacy system but the first formal issue has been submitted to the Vehicle Certification Agency for their consideration.

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Previously published, 15th World Congress on Intelligent Transport Systems

Using ITS to deliver the network management duty of the traffic management act and improve journey time reliability for road users

024

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Abstract

This paper outlines the innovative work being undertaken to deliver improvements to road users across the road network, in the county of Essex. The paper shows the links that have been established with the private sector to make innovative use of data sources, and the work with the County Council's partnered consultant to develop sound statistical analysis of data and the development of interventions to control and manage the road network. These activities combine to form the basis of the delivery of targets agreed with central Government which, when achieved, will deliver over £3 million of reward funding to Essex County Council.

1. Background

Essex County Council

The County of Essex is set in the east of England. It is a diverse and growing authority area. The south of the county is relatively developed and includes the Thames Gateway development area, linking it to London. In the west, Stansted airport is growing and increasing passenger numbers, putting pressures on both the road and rail network as well as housing and commercial infrastructure.

To the north of the county is the Haven Gateway, a growth area incorporating the port of Harwich which brings a significant level of freight into and across the Essex road network. To the east, the county has a long coastline (over 300 miles) and is generally rural. The county is crossed by both the M25 and M11 motorways (Figure 1). Despite having significant urban areas of growth, over half the county is covered by farmland.

Essex County Council is a second tier authority, providing a wide range of services to over 1.3 million residents including, education services for children and adults, library services, social care for adults and children, and the management and development of the highway infrastructure. It is the second largest shire authority in England and Wales.

The Development, Highways and Transportation group is placed within the Environment, Sustainability and Transport directorate. It has a budget of over £150m per annum (capital and revenue) and almost 800 staff.

Essex has 7500 km of roads and 785,000 registered cars, travelling over 11 million vehicle kilometres annually. Traffic levels are growing by 2% per year. The gross domestic product of Essex (the total economic activity in the area) exceeds £19 billion annually, representing over 20% of the economic activity of the eastern region.

Intelligent Transport Systems in Essex

In June 2000, Essex County Council entered into a partnership style contract for its Intelligent Transport Services with SA2000, being a joint venture between Siemens and Atkins. This partnership continues and will last until at least 2013. In its early years, the value of this contract was approximately £1 million per annum. With the development of ITS and new approaches to the delivery of the Traffic Management Act and services this has now grown to over £10m/pa.

The Traffic Management Act 2004

The Traffic Management Act (TMA) establishes and defines a need for improved network management. The network management duty imposed in 2005 requires local authorities to keep traffic flowing and to co-operate with other local authorities to achieve this. All traffic authorities are required to appoint a Traffic Manager. In order to be able to deliver all aspects of network management an authority needs to able to monitor traffic in both real time and historically to identify congestion hotspots and to then identify schemes and intervention

FIGURE 1. MAP OF ESSEX'S STRATEGIC ROUTES

strategies to improve flow and reduce delays.

The Traffic Manager has the general powers to intervene in traffic operations that may affect network performance, TMA 2(23). To do this effectively, he or she needs an overall picture of network performance in order to assess how a potential change to the network may impact overall performance.

Putting aside the traffic management duties and target driven funding, network performance monitoring is an important tool in identifying where investment is needed to combat congestion. It is also critical in demonstrating how channelling money into the right schemes improves the network. However, this can only be carried out effectively if based on quantitative analysis, i.e. statistics of traffic flows, journey times, accident rates, etc.

So at a fundamental level, performance management revolves around monitoring long-term network statistics and comparing them to a goal or target. It is not about the day-to-day running of the network, which is the job of control centre operators and engineers, but relates more to an assessment of the health of the network. For example, what is the effect of incident response times on performance, or how effective is a new scheme in reducing congestion.

Local Area Agreements

In 2006 Essex County Council entered into a Local Area Agreement (LAA) with the Department for Transport to deliver improved journey time reliability across the Essex road network. These targets were defined as more stretching targets than those set in the Local Transport Plan.

This agreement included an element of pump-priming and, on successful achievement of the targets, significant reward funding of nearly £3m. The concept of the LAA was to set stretching, pioneering and innovative targets for local authorities, which often needed to be delivered in partnership with other organisations.

The LAA for improving journey time reliability contains two priority areas:

- Improve the reliability of journey times for car users (increase journey time reliability to 95%)
- Reduce average journey times (reduce journey times by 1% across the board)

This agreement, coupled with the requirements of the TMA and increased public expectation were the key objectives for this undertaking.

2. Data

Reporting overview

The first stage in setting the targets was to establish baseline data against which to compare future performance.

The baseline data was established using an innovative agreement between the private sector (the Norwich Union insurance company) and Essex County Council.

The source data is collected by Norwich Union as part of their Pay as You Drive (PAYD) insurance scheme. In this scheme, customers purchasing motoring insurance are charged according to how far they drive, where they drive and at what time of day.

In order to monitor and calculate the charge, each policy holder needs to have a GPS tracking system fitted to their vehicle. These systems collect positional information as the vehicle drives round the network, thus capturing a record of the distance travelled, where the journeys are made and at what time.

The data collected by the unit is uploaded to Norwich Union's servers overnight for permanent mass storage providing a bank of floating vehicle data from which journey times for links and between points can be calculated. This data is anonymised for use as floating vehicle data.

As well as using this data for calculating insurance payments, Norwich Union also supply the data (for a cost) to local authorities for measuring journey time performance. This data is provided as a list of records relating journey times to various sections, or links, of the road network.

Each record contains a journey time sample for a particular vehicle and has a unique vehicle identifier, the time the vehicle entered/exited a link, and the time to travel the link.

This data is gathered over a quarterly period and is passed to SA2000 (the Council's ITS consultant) for processing.

To establish a baseline, each route was assessed to determine an Allowed Travel Time (ATT) from the start and end points in both directions. This overall ATT is then broken down and a proportion allocated to each link. In this way, the time it takes to travel on each part of the route can be calculated. The actual time that the floating vehicle data reports current traffic as achieving is compared and contrasted with the ATT recorded for that route.

In addition, and to offer a more permanent and reliable method of measuring journey times on a wider network, the County Council has made a significant investment in a system of Automatic Number Plate Recognition cameras (ANPR) with over 80 cameras being installed to date. SA2000 has undertaken work to compare and validate the data from the two different sources, and has found a strong correlation between the two data sets.

Analysis

Given the sheer volumes of data collected by ITS systems, it is often difficult, if not impossible, to handle and process these with a simple set of spreadsheets. For example, an ATC collecting samples into five minute intervals will gather over half million samples in a year. Multiply this by a network of sensors and the amount of data being collected and stored balloons to tens of millions of samples.

Presented with such a huge amount of data, it has been necessary to employ a suitable database and reporting system to process these large quantities of data reliably. These systems are then able to present the information to the ITS engineers for interpretation to develop interventions and actions to better manage Essex County Council's road network.

At its most fundamental level, the reporting system extracts the historical data from a database and aggregates it according to a set of spatial references and time periods, which form the resulting key performance indicators.

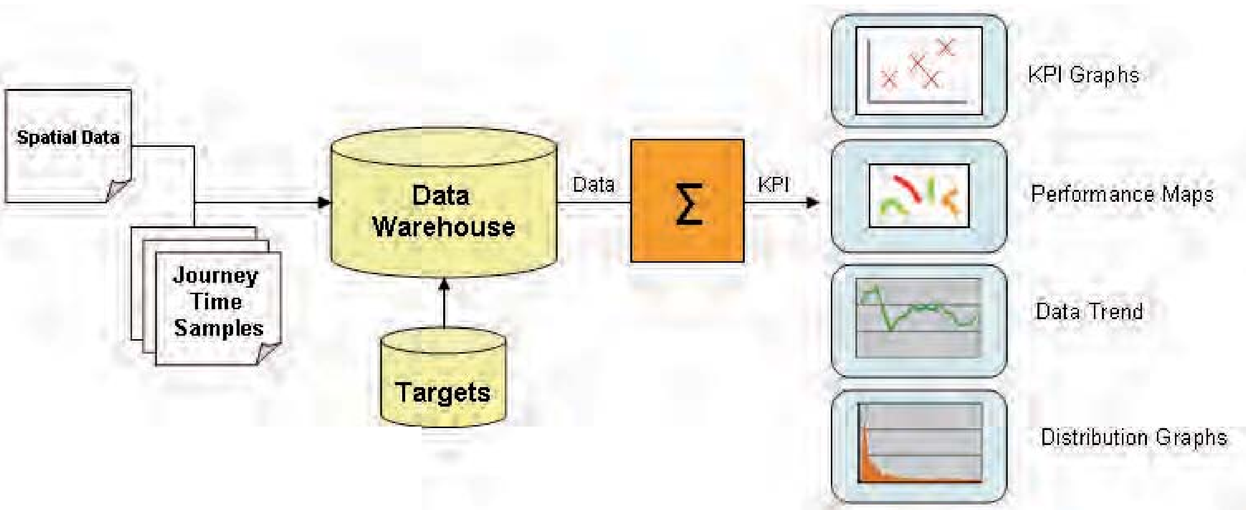

FIGURE 2. LOGICAL ARCHITECTURE

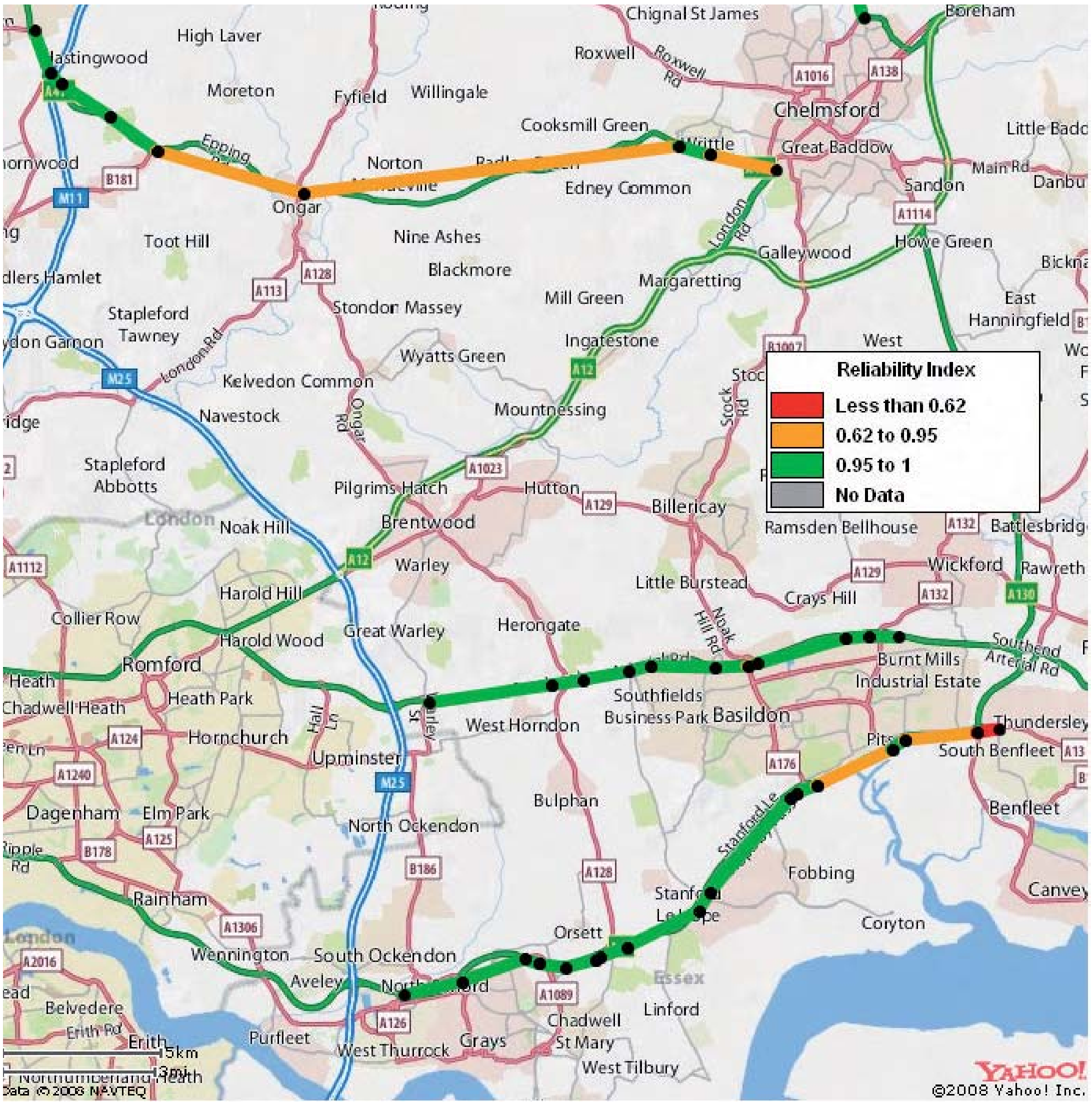

FIGURE 3. ANALYSIS SUMMARY RESULTS

Calculation method

Floating vehicle data sources are too sparse to analyse the network in terms of daily trends and delay. However, unlike ANPR cameras, they are able to provide extensive network coverage and quantify network performance for comparatively small cost. Through the continual calculation of average journey times, journey time variability and journey time reliability (For calculations see Appendix A.1-4) it is possible to identify network hotspots and analyse performance trends which enable targeted investment.

Figure 2, illustrates the logical architecture of the system used to manage, process and view the large quantities of ITS related data for Essex County Council.

A file of raw Norwich Union data is received each quarter and journey time samples are extracted, filtered and assigned to spatial identifiers before being loaded into a data warehouse. The warehouse is a historical repository for all valid journey time samples which can be called upon for trend analysis in the future.

The warehouse is structured in a way for it to accept journey time samples from a variety of data sources including Essex's own ANPR network of cameras.

The spatial identifiers are essentially the predefined links. However, the spatial identifiers also group these links into routes, grouped routes and grouped groups for the purposes of calculating indicators. This allows a unique set of aggregations to be defined for each group ultimately creating the high level, 'Grouped Group', indicators required by the Department for Transport.

Organising the data in this way allows the quick identification of under performing indicators along with the ability to drill down into the data to find out which sections of the road network are affecting its performance.

The last element of the system is able to present the data in a number of graphical forms to help communicate the network performance to relevant stakeholders.

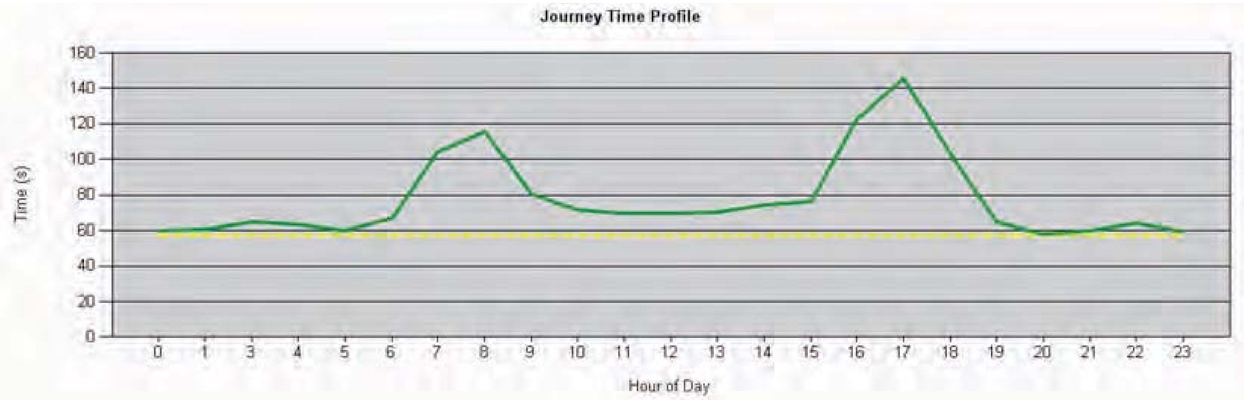

FIGURE 4. JOURNEY TIME PROFILE

FIGURE 5. JOURNEY TIME DISTRIBUTION

Presentation and interpretation

Once the data has been received it can then be processed, analysed and presented in a number of different ways:

- Spreadsheets
- Graphical representation
- Map-based information
- Summary information (highlights and lowlights)

For ease of use, each route is plotted over a Google Map and is displayed as a coloured line. The line colour depends on how well the route is performing against its target. Each link on the route shows up as either green if the link is meeting or exceeding its target, amber if it is performing between 62% and 95% of target, and red if it is performing below 62% of target (Figure 3).

This means that it is very easy to pinpoint where on the network the potential problem exists. Far more complicated is the identification of the nature of the problem. Reports and information can then be produced for each separate link on the network.

Data can be analysed and compared to norms for that time of day, clearly identifying the problems to be resolved. Figure 4 compares the actual journey time (green) for a section of the journey with the average target time (yellow). Clearly the peak hour traffic exceeds this figure and highlights the need for additional research and possible intervention.

Figure 5 details the journey time distribution for the section of the journey in Figure 4. Journeys below the target time (57 sec) are shown in green with those above being in red. It is clear on this link that approximately only 35% of vehicles are within the target time.

Using data for congestion management

There are three main factors that contribute to congestion on the road network:

- Volume of traffic - 65%
- Accidents and unplanned incidents – 25%
- Road works and planned events – 10%

Congestion itself is one of the factors that contributes to air quality issues through vehicle emissions. It has been shown that ITS has a role to play in reducing congestion (and therefore emissions) through:

- Reducing the impact of congestion through improved route guidance
- Reducing the causes of congestion – creating a smoother traffic flow and less standing traffic
- Reducing demand by altering driver behaviour

Over a protracted period it is possible to identify links that are consistently underperforming, and also to see where unexpected events have occurred at previously "on target" links. This now initiates the second section of the Performance Management process during which remedial interventions can be designed to address any identified issues.

Once a potential problem is flagged up by the analysis, the first step is to check that there are no obvious causes e.g. road works, road closures, lane closures, road accidents, signal failure and so forth. With those events discounted, the next step is to look at the link to see if there are any indicators why the link isn't performing to target. Factors to be considered range from the standard of driver behaviour on the link, to design and engineering issues affecting its reliability. Driver behaviour may be affected by inadequate or confusing signing and lining, lanes narrowed by vegetation, signals timing and position as well as the original design of the carriageway layout. If necessary, a manual traffic survey is undertaken at problem areas to gain a more detailed understanding of the cause and effect.

3. The Essex Traffic Control Centre

The innovative Essex Traffic Control Centre is the main focus for congestion management. The centre delivers two main functions, firstly acting as the control room for monitoring the network and implementing intervention strategies in response to planned and unplanned incidents, and secondly to provide all forms of travel information and advice to the public relating to journey planning advice and live information on the network, including: bus travel, car park information, roadworks information and the County Council's response to incidents and accidents. In the future it is intended that the Essex Traffic Control Centre will develop further and co-ordinate information relating to fault management, maintenance and streetworks.

The control centre monitors not only the Essex road network but also has strong links with neighbouring authorities and the Highways Agency to ensure that responses to incidents are co-ordinated and managed. Officers have worked with the Highways Agency to share access to new Variable Message Signs that the Highways Agency is installing on the A12 (a key HA route carrying significant traffic volumes through Essex) which will be used to advise drivers of alternative routes when there are incidents and delays on the A12. The possibility of using this infrastructure for CCTV cameras is also being discussed. Protocols are also being established with the Highways Agency to agree better management of diversion routes which may include signing and changes to signal timings.

4. Intervention and management

A number of different workstreams have been put in place to give a sound base from which to take action to achieve the targets set in the LAA for journey time reliability. The work undertaken so far falls into a number of different streams.

Intervention strategies

Over 100 intervention strategies have been specified and are being put into operation via the Essex Traffic Control Centre. A number of these interventions are automatically implemented through COMET, the Essex Traffic Control Centre operating system, but many can also be implemented and adjusted manually. The interventions are based on identifying specific links where incidents may occur and detailing the responses and actions that should be put in place, such as setting Variable Message Signs at key locations across the county network, broadcasting information via the media (currently the Essex Traffic Control Centre gives live travel broadcasts on local radio stations) and changing timings on signal controlled junctions. An example of an intervention strategy is that used at Junction 7 M11/A414 Harlow. This is successfully implemented via the Essex Traffic Control Centre to clear the circulatory carriageway when the roundabout "locks up" due to heavy volumes of traffic (the A414 route is also one of the defined LAA routes).

Congestion schemes

The intervention report identifies locations where congestion and delays are caused by capacity issues. At these locations, congestion schemes are implemented to make better use of roadspace, improve driver behaviour and allow intervention and control mechanisms through technology. This can also include enforcement of bus lanes and gates where contravention of such orders creates delay for passenger transport. Successful schemes include part-time peak hour signal operation, congestion responsive pre-signals, and more active use of SCOOT control.

Outcomes

This project has proved both challenging and demanding. In its simplest form it has allowed us to gain a much more accurate and detailed understanding of the performance of the network. As can be seen from Figure 6 and Figure 7, this technology and information has enabled informed strategies and decision making, which in turn has led to real outcomes being delivered.

FIGURE 6. ESSEX KPI REPORTING 06/07 JAN-MAR

KPI		Journey Time
LPT3		142.201
	Interurban Eastbound	70.279
	Interurban Westbound	71.922
LPT4		93.079
	Radials (Inbound)	47.98
	Radials (Outbound)	45.099
Total		235.28

FIGURE 7. ESSEX KPI REPORTING 07/08 JAN-MAR

KPI		Journey Time
LPT3		135.178
	Interurban Eastbound	68.904
	Interurban Westbound	66.274
LPT4		93.371
	Radials (Inbound)	50.332
	Radials (Outbound)	43.039
Total		228.549

With journey time savings of around 2.5%, the improvement is substantial. These savings equate to an economic benefit to Essex on excess of £20m per annum, with a CO^2 reduction of over 9,000 tonnes per annum. These improvements have helped improve customer satisfaction whilst providing a safer travelling experience. The increased network resilience has not only aided journey time reliability but has really begun to facilitate the ultimate aim of making best use of the network capacity, whilst enabling better journeys for our ultimate client, the travelling public.

Challenges

This project has explored a number of data sources for monitoring journey time reliability. The use of floating vehicle data has proven useful, but since it relies on a pool of tagged vehicles it has not proved to be consistent enough (in isolation) for key routes. A better approach in this context has been the use of ANPR data. The challenges of setting targets for national indicators and for LAA2 with the option of using floating vehicle data sets are now being investigated. It is likely that this data will be used for setting baseline data with ANPR being used to monitor and report.

5. Acknowledgements

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Appendices

A.1 Average journey time

Calculated by summing over linked averaged journey times

$$\mu_{jt} = \sum \overline{jt}$$

Where:

jt = Journey Time sample

A.2 Journey time standard error

Calculated by summing over the link standard error.

$$jt_{stderr} = \sqrt{\sum \frac{\left(\overline{jt^2} - \left(\overline{jt}\right)^2\right)}{N}}$$

Where:

jtstderr = Journey Time Standard Error

N = the Number of samples

jt = Journey Time sample

A.3 Journey time reliability index

$$RI = \frac{\sum N(jt \leq ATT)}{\sum N}$$

Where:

RI = Reliability Index

ATT = Allowed Travel Time

N = the Number of samples

jt = Journey Time sample

A.4 Journey time reliability standard error

$$RI_{stderr} = \sqrt{\sum \frac{0.0475}{N}}$$

Where:

N = the Number of samples

RIstder = Reliability Standard Error

What happens when the control centre is not there? Technology resilience for the Highways Agency regional control centres

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Abstract

Over the last 20 years Intelligent Transport Systems have performed increasingly important roles in the management of urban and interurban travel. The Control Centre has become the hub for the co-ordination of incident management strategies. Located within the Control Centre are key staff and key systems.

This paper poses the question "What happens when the Control Centre isn't there?" In particular it discusses the technology resilience provided for the Highways Agency Regional Control Centres in England in their support of the Traffic Officer Service.

1. Introduction and background

The introduction of the Regional Control Centres (RCCs) represents a profound change in the way that the motorway network in England is operated. Prior to these changes, the Highways Agency (HA) was responsible for the infrastructure and maintenance of England's motorway and trunk roads. Now the HA is also a Network Operator and responsible for actively managing traffic and optimising network performance. The HA's role as Network Operator places a far greater emphasis on the resilience of the systems that the agency uses to manage the road network. The implementation of the Traffic Officer Service delivers substantial support to the aims of the HA, which are "Safe roads, reliable journeys and informed travellers". Interruption to this essential service will impact delivery of these aims and may be detrimental to public perceptions of the service. Long-term disruption will have an impact on the ability to achieve congestion targets.

The main technologies deployed at the seven Regional Control Centres that support the Traffic Officer Service are:

- Command and Control System
- Integrated Communications and Control System
- Airwave Radio
- Operational Telephony
- Closed Circuit Television System (CCTV)
- Highways Agency Traffic Management System (HATMS)
- Dynamic Display System (DDS)

The technologies may be categorised as National and Regional. National technologies are so called because their functions and information are available at all the RCCs whilst the Regional technologies are currently only available at the host RCC. Some of the National technologies initially had restrictions on their use from a different RCC.

The strategy has been to remove these restrictions whilst attempting to move the Regional to National technology status. This has been achieved for the CCTV system with the advent of 2nd Generation CCTV and the National Roads Telecommunications Services (NRTS) project. The HA operates a dedicated telecommunications network that interconnects many thousands of roadside devices (telephones, cameras, signals, etc.) to the RCCs. This network is made up of fibre optic and copper cables that run along the length of the English motorways.

The RCCs are located close to the motorway and connected into the network via diverse routed cables. The NRTS project now allows these roadside devices to be connected to any RCC.

The seven RCCs together including the two data centres and intercommunication networks may be considered as a system. Many of the elements of this system have in-built resilience in their own right, (e.g. dual processors, uninterruptable power supplies, diverse cable routings). Also, alternative methods of achieving the desired customer perceived outcomes are possible, (e.g. mobile phones as an alternative to Airwave, paper and pencil as an alternative to a computer).

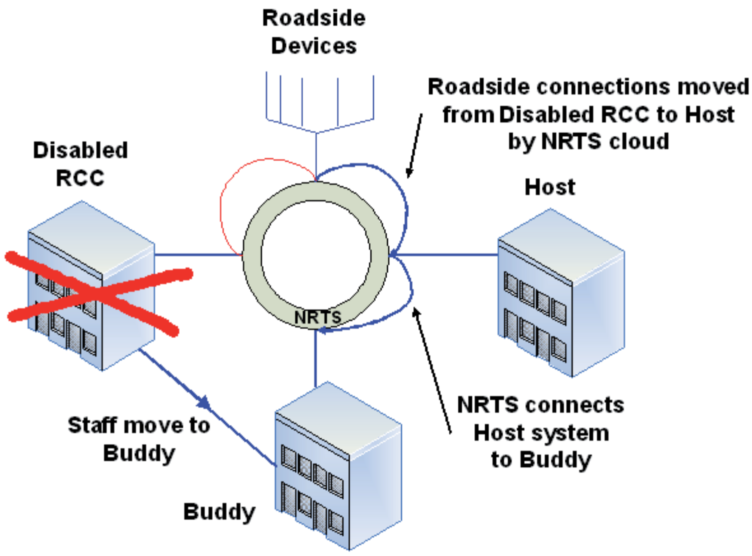

FIGURE 1. SIMPLIFIED DIAGRAM FOR DISASTER RECOVERY

The operational end-user requirements of the system were ascertained as functional requirements with proposed availability and time to fix figures. The next action was to undertake a gap analysis between the requirements and the existing systems. This identified that the RCC systems were largely compliant. The RCC Technology Team has been scoping the work required to provide technology solutions for the RCCs that will provide continuity of the service in the event of an RCC having to be evacuated or disabled.

The first step for the RCC technology team was to define a set of requirements in order to establish the range of systems that will have to be provided by a Disaster Recovery Facility. It was clear from the collated requirements and other information that there are significant operational and technical issues that will have to be resolved. This paper only deals with the technical issues. In addition, other areas of the HA are working to improve the RCC systems, including the updating of new technologies. These improvements will change the solutions required for Disaster Recovery: for example the introduction of IP based telephony will simplify those solutions as well as making feasible further improvements in resilience.

In practice, there are a spectrum of scenarios that may have to be addressed but the following three categories are a convenient means of discussing technology strategies:

- Support from a neighbouring RCC in the event of heavy operational workload
- Evacuation of the RCC while the Regional systems remain operational
- Evacuation with some or all of the RCC systems no longer operational

In this paper the former is termed "Buddying", the second is termed "Fallback" and the third is termed "Disaster Recovery". They are arranged in order of increasing impact but decreasing likelihood.

2. Buddying

A "Buddy" is an RCC with the ability to take over operations from another. This would allow: workload sharing during peaks; potential for resource optimisation and fallback for outages etc.

To achieve "seamless" Buddying, systems and data will need to be available to ensure:

- Swift, smooth transfer of operations from subject RCC to target RCC
- Continued operation of subject and target RCC for as long as is required
- Smooth transfer back of operations from target RCC to subject RCC

As noted above, most of the technology systems have achieved National Status and therefore Buddying is relatively straightforward. The resilience study assessed the HATMS difficulty of setting signs and signals and conceived a solution that used relatively "standard" network switching techniques across the NRTS network to establish remote operator positions to allow control of the subject HATMS from a target RCC. These proposals enhance the availability of RCCs and deliver a powerful ability not only to address discontinuities but also allow new methods of collaborative working between RCCs to enhance the productivity and efficiency of RCC operations. Two different forms of switching techniques have been proposed and these are being separately trialled, compared and the best solution will be rolled-out.

3. Fallback

In the event of an RCC being evacuated, access would currently be lost to the Regional Technology Systems even though they remain operational. The urgency of the need is different from that of Buddying in that it is required now rather than after negotiation. It is envisaged that, in the short term, the proposed Buddying HATMS method will allow the Buddy RCC(s) to provide support to the on-road Traffic Officers.

4. Disaster recovery

In the event of an RCC being disabled, whether as a result of a loss of power or communications or access, then a separate set of hardware needs to be available for the disabled RCC. Figure 1 shows the outline architecture and the key components discussed in this paper.

One possibility for the separate set of hardware is to use existing training centre hardware but this has obvious implications for the loss of training. It should be noted that training will be lost not only in the very rare case of a disaster (in the 30 year history of 30 Police Control Offices there is only one report of a disablement for any length of time) but also for testing purposes. An ongoing programme of testing with results audited to highlight and correct issues arising is essential to assure resilience. Given that there are seven RCCs and each needs to be tested at a reasonable interval, then this could result in a loss of many days of training per year. If resilience was the only consideration and cost was not an issue then each region could have its own spare RCC building and systems kept running and ready for instant occupation. However resilience is always a balance between business threats and the cost of defence.

5. Summary

A resilient operation is essential for delivering a successful service. Service continuity requires a resilient supporting infrastructure. Operation can be characterised as where people, process and technology come together. Technology will only be truly resilient when people and process are working together in harmony.

There are still several key decisions and operational issues to be resolved but we are confident that the resulting system will have an appropriate level of resilience.

6. Acknowledgements

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What happens when the control centre is not there? Technology resilience for the Highways Agency regional control centres

Technology resilience for Highways Agency network operations

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Abstract

The management of technology has changed on English motorways and this paper outlines some of the challenges that have been faced and overcome by the Highways Agency. With the advent of Regional Control Centres and recent announcements to roll out Active Traffic Management (ATM), the dependency on technology for effective network management has been highlighted. Fault conditions cannot be tolerated for long periods and fault resolution is a much more significant activity in achieving operational resilience.

The change in focus from technology support to operational support is being reinforced by the trial and implementation of a fault management system, a communications and escalation process and a technology resolution service. This paper outlines progress to date in pursuing this strategy and outlines future plans.

1. Introduction and background

Until 2004, the English motorway network was managed operationally by the police from nearly 30 control rooms. Since then, however, the Highways Agency has taken over operational responsibility. A Traffic Officer Service was established and seven Regional Control Centres (RCCs) were built to support the service. Although the Traffic Officers work closely with their police colleagues, the Highways Agency needed its own command and control and radio systems. These were added to their technology inventory that included VMS and matrix signal control, CCTV, traffic and environmental monitoring, transmission and emergency telephone systems. Combined with the need to equip people and vehicles with the necessary equipment, the level of dependence on technology in the Highways Agency took a step forward with their enhanced operational role.

More recently, The success of the Active Traffic Management trial on the M42 near Birmingham, UK, has encouraged a roll-out of this managed motorway approach to resolve some of the congestion problems on English motorways. The availability of technology is a key factor in this approach, so there is a greater sensitivity to system failure and to fault resolution.

Existing maintenance contractors integrated well into the new RCC arrangements. It was clear from the outset, however, that additional local support was needed, which was provided in the form of Technology Systems Support Engineers (TSSEs). TSSEs offer welcome and vital services to Traffic Officer staff in each RCC to resolve faults. Further lessons have been learned, however, in the years since launch and a review of the operational support indicated that additional measures was needed to provide the appropriate level of technology service for operational staff. Atkins was commissioned by the Highways Agency as part of the Tandem Consortium to examine the feasibility of implementing a National Technology Resolution Centre (NTRC). (Tandem consists of Atkins, Hyder, IPL and CCL. It provides technical support services to the Highways Agency for RCC technology.)

2. Problem definition

Technology is recognised as a key factor in the provision of an effective and efficient Traffic Officer Service for the management of the network. Business continuity demands that the availability of systems is as high as possible and there is recognition that although the current fault resolution service meets operational needs, opportunities for improvements are available. A small number of high impact technology problems have placed this issue under the spotlight.

It is apparent that there are particular difficulties in managing National Technology faults. Previous to the advent of the RCCs, technology could be managed within each police control office. Some of the Highways Agency's new systems, however, can only be procured, maintained and managed on a national basis (e.g. Command & Control and Radio).

Any reduction in availability of these "new" facilities has a wider impact on operational performance than had been experienced with previous approaches. This does not necessarily mean that such technologies are less reliable. It is more likely to indicate that the impact of a National Technology fault is greater. National Technology fault resolution is managed by suppliers and some co-ordination occurs, which can be improved with a managed approach. Fault management in RCCs is performed by the TSSE in the first instance, but this responsibility falls back to the operational staff outside normal working hours or when TSSEs are not available.

Resilience requirements have been determined for Business Continuity. This indicates that there is a much greater reliance on technology for RCC operation than there was under police management, which places a greater onus on the performance of systems. Specific targets for availability of systems were identified.

It is clear, then, that there is now a much greater reliance on technology to achieve operational targets than there used to be. This trend is likely to continue as new techniques for traffic management are deployed that rely on technology, rather than tarmac to increase the capacity. This means that technology support services need to be enhanced.

Improving system reliability is clearly one way of improving availability (i.e. increasing the Mean Time Between Failures– MTBF). In addition, however, availability can be improved through a reduction in the Mean Time to Repair (MTTR). The approach outlined here focuses on the MTTR to improve availability through the deployment of a National Technology Resolution Centre (NTRC).

3. Summary literature search

Most of the investigations and analysis in the development of the NTRC relied on internal HA consultation and information. Cox and Tait (1991) was used as a guide for general issues relating to reliability and availability. Service design was influenced by Fitzsimons & Fitzsimons (2000) and the "seven-S" model from Peters and Waterman (2004) was used as a basis to describe the organisation of the service.

4. Key issues

Stakeholders were identified and consulted to determine their needs. The key stakeholders were operational staff and technology managers over a range of levels. Their needs were for a single contact point that would receive fault reports and take ownership of resolution. 24/7 support was required and national coverage was desirable, particularly for national technology (i.e. systems that were implemented and managed on a national basis). Fault tracking and progress reporting needs to be tailored to the level at which the request is made, which means that national monitoring and reporting is required.

The concept of a service provided by the NTRC was established to meet the needs identified above. The NTRC will deliver improved effectiveness, efficiency and operational planning. It will make technology planning easier and it will reduce disruption and wasted effort caused by unnecessary escalation. A national technology fault resolution service would be a 24/7 operation that is implemented ideally through a support centre. It will manage all faults relating to national technology. Implementation of this service will reduce the need for operational staff to be involved in fault resolution for national technology and it will increase system availability. It will also reduce the risk of duplication of effort (particularly from technical support staff).

The NTRC therefore needs to provide improved efficiency and effectiveness by enabling operational staff, TSSEs and managers to focus on their primary responsibilities. It improves planning by providing greater certainty of resource availability and system availability.

5. Implementation

The NTRC is being implemented as a service through system deployment, process development (particularly for communication and escalation) and education of stakeholders. The key features of the NTRC are a commitment to providing operational integrity, 24 hour support for RCC technology, a support centre arrangement, a supportive, proactive, tenacious, communicative style, two to three full time skilled engineers with detailed knowledge of systems and a personable nature, a variety of tools and systems, including ICT, tracking, auditing, operational overview, information, knowledge, processes and fault resolution support.

6. Early results

NTRC implementation is progressing. An interim service was established on a trial basis in March 2008, focusing on Planned Engineering Works (PEW) and Permits to Access (PtA). The trial has been implemented without imposing additional burdens on suppliers and it relies on service centre staff to ensure that applications for PEWs are properly processed. A technical forum of experts provides assessment.

The interim NTRC service is starting to meet its objectives and in particular, to provide information relating to volumes of PEWs and PtAs. These are still ramping up as stakeholders gain confidence in the system and process and they are expected to stabilise later in 2008 to enable HA to specify a full NTRC system and migrate into full fault management.

The positive outcomes for the Traffic Officer Service are expected to be improved operational planning, increased effectiveness, reduced inefficiency, improved communications and improved confidence in technology availability. For TSSEs, the benefits will be evident in reduced duplication and inefficiency and centralised resolution of national technology. RCC technology team managers will be able to drive system availability improvements and they will be able to plan for system provision, upgrade and maintenance more easily. Senior management will spend less time and effort dealing with fault situations.

7. Conclusions

There are problems with the current RCC technology fault resolution arrangements that can be addressed by the NTRC. National technology system faults can have a significant impact on the performance of RCC operations and the current TSSEs service should be maintained and, if possible, enhanced.

NTRC operators must take ownership of fault resolution and be technically skilled, have appropriate systems and access to information and must be able to communicate with all relevant stakeholders.

The NTRC needs to refocus national technology fault resolution away from system support to operational support. The NTRC is well placed to manage the escalation and communication process.

Various options exist for a long term solution for the NTRC. The National Fault Data Base could provide a fault resolution support system for the NTRC, but this depends on negotiations with the NTRC supplier.

8. Acknowledgements

This paper is based on work undertaken in the Network Operations Directorate of the Highways Agency and is published with the permission of its director Derek Turner. The views presented in this paper are those of the author and do not necessarily represent those of the Highways Agency.

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Developments in highway design standards: Key issues for highway authorities

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Abstract

Recent innovations in the design of public realm schemes have opened up a wider debate on the role of highway design standards. The design of highways is becoming increasingly complicated as the use of innovation, particularly technological solutions gathers momentum. This is set alongside a background of balanced appraisal methods and a growing appreciation that highways are not just for motorised vehicles.

This paper discusses a number of current highway standards issues and why care needs to be taken in creating policy. Inappropriate use of standards may attract liability or be a contributory factor in the non-achievement of wider policy outcomes. The enactment of the Corporate Manslaughter and Corporate Homicide Act in April 2008 provides an opportunity to review current practice.

1. The current situation

Design standards are drafted to seek an appropriate balance across the recognised investment criteria of: safety/security, economy, integration, environment and accessibility.

There are many sources of highway design documentation. The main ones are:

Design Manual for Roads and Bridges (DMRB)

The DMRB is fully applicable to strategic highway networks across the UK and is the only structured comprehensive UK manual, making it a well respected set of documents. Although comprehensive in comparison to other discrete information sources, the DMRB does not deal with all issues and cannot respond quickly to changing circumstances.

Documents produced by individual Highway Authorities

The "Introduction to the DMRB" notes that

"The manual sets a standard of good practice that has been developed principally for trunk roads. It may also be applicable in part to other roads with similar characteristics. Where it is used for local road schemes, it is for the local highway authority to decide on the extent to which the documents in the manual are appropriate in any particular situation. While the requirements given in the manual may be the best guidance available to local authorities, such authorities should ensure that their application to local road schemes does not compromise safety, result in poor value for money, or have an unacceptable impact on the environment."

Therefore it is expected that individual authorities will formally confirm the applicability of DMRB and other source documents and produce their own documents, where appropriate. It is also expected that a system will be created for departing from the single common-base requirements.

In the course of developing a design Index for the London strategic highway network, Atkins revealed that few authorities seemed to have a comprehensive strategy for documenting their family of design documents. The most common replacements or additions to DMRB documents were associated with:

- Road safety audit
- Policies for skid resistance
- Residential street layouts and associated advice for developers
- Bus and tram layouts
- Streetscape, including mobility issues

Interim Advice Notes

The Highways Agency publishes Interim Advice Notes on its website. It is likely to be important that highway authorities can demonstrate that they make themselves aware of latest practice by having a formal system for reviewing emerging documents.

European Standards

Some European Standards will create a new challenge for highway authorities. The UK National Annex 1 for EN 12767 (the standard for passive products) uses a new form of language. It states that: 'The decision to specify products which conform to performance classes from BS EN 12767:2007 in a particular situation, rather than class 0 (products not certified as passively safe), is a matter for the road authority.' Authorities may be well advised to produce written policies on the use, non-use or partial use of the Annex so that their potential liability is reduced.

DfT documents

The DfT prepares general advice e.g. Local Transport Notes and Traffic Advisory Leaflets, which in some cases refer to requirements having a statutory backing.

Other documents

This category includes best practice guides issued by the Highway Agency, CSS and professional Institutions. It also includes more formal Codes of Practice issued by similar bodies

2. Legal status of standards

By and large the courts consider the wide family of standards as being of similar status, sometimes using a generic term "Code of Practice" (CoP). Indeed many highway liability cases explore the area of maintenance rather than specific design documents. "Well-maintained Highways Code of Practice for Highway Maintenance Management"[2] is an important CoP for maintenance activities.

Generally in attributing fault, a civil court is looking for evidence that an authority is not operating similarly to its peers, by acting contrary to established industry practice. However in the criminal courts, the test will be different and compliance against the requirements of specific legislation will be assessed.

Of particular concern are the Construction (Design and Management) Regulations[3]. An owner of standards (a "specification" in CDM parlance) is in effect acting as a designer and therefore attracts associated designer duties. Part 1.2 of the Regulations state:

"design" includes drawings, design details, specification ……prepared for the purpose of a design;

"designer" means any person (including a client, contractor or other person referred to in these Regulations) who in the course or furtherance of a business:

(a) prepares or modifies a design; or

(b) arranges for or instructs any person under his control to do so

The provision of facilities that do not allow future safe maintenance is an aspect that may attract attention in court. It is important to note that many DMRB documents rely on the designer's additional input. Recognising this potential gap in DMRB advice, the Highways Agency produced IAN 69/05[4]. Local authorities who simply rely on DMRB may be in a difficult situation. Progressive authorities may consider using IAN 69/05 or an equivalent document.

3. Defining standards

The absence of a formal policy that defines the standards to be used may create many problems, such as:

- Legal liability, particularly as DMRB specifically asks for such policies
- Extra costs in defending liability claims
- Slow design as designers familiarise themselves with the potential standards
- Increased or abortive design fees if inappropriate standards are used
- Increased construction costs, particularly in urban areas if rural standards used
- inappropriate allocation of highway space for different user groups
- solutions that fail to meet overall highway authority aspirations
- inconsistency for road users
- lack of a basis for the future improvement of standards
- lack of public information, against the spirit of the open-government agenda

Consistent with the DMRB recommendations, the process of identifying appropriate standards is a matter for each authority that reflects their local design needs and standards.

Transport for London recognised these potential weaknesses and through Atkins has now developed its own Highway Design Index which takes into account the many documents that were in use across its organisation, including the formalisation of which parts of DMRB were usable.

4. Design standards and the urban realm

The Manual for Streets (MfS) seeks to encourage designers to move away from a prescriptive approach and asks for active decisions, based on local conditions and risk assessment.

Any highway scheme, urban or rural, is a unique blend of circumstances, budgets and objectives. Therefore no standard, whether that be a more traditional standard or a guidance document like the MfS, can be used without intelligent application. Highway design engineers have an important job in seeking out the relevant standards, some of which will be unique to a particular city, and then applying them in such a way that the final product and highway outcomes are suitable for the client. Designers will often be faced with challenging the "perceived wisdom". Restrictive briefs should not automatically be taken at face value. As an example of good practice, Transport for London has produced an assessment tool to help with the difficult decision on whether to provide pedestrian guardrails.

It is increasingly necessary to think outside the box and develop local design documents that respond adequately to the challenges of local congestion targets and also the needs of walkers and cyclists. Local Transport Note 1/08[5] is a useful addition to designers guidance, but the collection of photographs serves to demonstrate a worrying variability of local practices. This emphasises the need for specific guidance to be developed at a local level for a number of towns and cities. LTN 1/08 itself states that designers 'may be unaware of the status and intended role of guidance documents and regulations, treating all as mandatory instruction'. It goes on to advise that 'Local authorities have considerable discretion in developing local policies and standards and should apply appropriate professional judgement to bear in their application'.

5. Corporate Manslaughter legislation

In April 2008 new legislation[6] came into force. Although this has to be tested in the courts, the main concern for highway authorities and their designers is probably whether design standards and subsequent designs comply with the CDM Regulations. Given the time since the first publication of the CDM Regulations (in 1994), it is possible that an inappropriate instruction to use standards that are non-compliant with those Regulations could be seen as a material fact, hence the possible importance of rectifying design standards piecemeal or by means of an overarching policy similar to IAN 69/05.

6. Departures from Standard

The Introduction to DMRB recommends:

"Users must ensure that any proposal involving a departure from the technical requirements is formally approved through the particular procedures of the relevant Overseeing Organisation prior to incorporation into the works."

Authorities that do not have a transparent departures system could:

- Attract potential legal liability
- Be implementing inconsistent designs
- Be incurring potentially higher construction costs or detriment to the environment by adopting "gold-plated" designs
- Be seen as risk averse

7. Standards for complex projects

I have been involved in supporting the DfT as an EU Road Safety Infrastructure Directive has progressed through legislative stages. Despite an earlier rejection, it now appears likely that an amended proposal will become law. At that point it will be compulsory, on the trans-European network, to undertake a number of road safety management procedures. For the UK, many of the required procedures already exist, but it is noticeable that the Directive calls for the safety impact of a new or improved road to be assessed. We have always done this in the UK, but in light of the exemplary work done on M42 ATM and other innovative projects perhaps it is likely that there will be greater rigour for all significant projects. The Highways Agency is already working up standards to define the most appropriate level of safety and design management for their individual projects.

8. Conclusions

Appropriate technical governance can reduce liabilities and improve highway performance. The adequate definition of standards may also become more important as outsourcing of services increases. Highway authorities may wish to consider the following:

- Identifying, in a formalised manner, the relevant standards used by the authority
- Developing local standards that respond to local needs, consistent with the advice of LTN 1/08
- In light of new corporate manslaughter legislation, consider technical governance issues generally, but particularly consider adopting a policy that reflects the contents of IAN 69/05
- Developing formal policy on passive street furniture on high speed and lower speed roads
- Adopting a departures from standards policy
- Arrangements for assessing safety impacts in line with the European Directive

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Atkins' Best Practice Guide for Integrated Data Management on Highways Projects: The first steps

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Abstract

As a company with the vision of becoming the world's leading infrastructure consultancy, we know that we are going to have to deal with large amounts of disparate data - and it therefore makes sense that we should be leading the way in implementing world class data management practices. In reality, we have some way to go in realising this vision. We DO have the knowledge, the expertise, AND the collaborative platform (in the form of the Atkins' Spatial Data Infrastructure) which can help us realise this vision. What we DON'T yet have is an established culture for accepting basic data standards and workflows across the company which embeds integrated data management into all our working practices.

Yet one of our largest costs is acquisition and maintenance of data: reducing this cost and increasing our return on investment from the data should be of major concern to all of us. This paper presents the first steps to achieving this goal.

1. Principles

The very nature of our work as an infrastructure consultancy is reliant upon having data that is:

- Accurate
- Consistent
- Accessible
- Readily presentable

In order to achieve the above requirements, a good data management strategy is necessary – and a key ingredient to this strategy is the development of a "Single Source of Truth". Our target is that this "Single Source of Truth" is not a central repository of data, but a collaborative platform, based on the Atkins' Spatial Data Infrastructure (SDI) - see Appendix for basic description, which provides access to all the available data and is able to extract and present the data in an integrated form.

As Network Chair for Geospatial and Integrated Digital Solutions, one of my key objectives is to establish Best Practice across the Atkins Group. I am, however, very clear that this Best Practice will come from working with all Businesses across the Group to determine what is already being achieved, where we have existing centres of excellence, and where, collaboratively, we judge we should improve. The purpose of this paper is to identify the key building blocks, and encourage you to engage with me to establish what will work best for Atkins.

The basic principles of data management have been used on a number of projects (e.g. A3 Hindhead, Olympics, Metronet), and a number of our clients and competitors (e.g. Yorkshire Water, Environment Agency, SOM, CH2M Hill, Arup, Halcrow) are embracing the use of Integrated Data Management Systems (IDMS) enabled by GIS technologies in contemporary large asset management projects as Best Practice, as reflected in BSI PAS 55:2008[1]. The use of Geographic Information Systems (GIS) as a technology enabler, reflects that most of the data we use can be linked to a geographic location or feature. Within Metronet it has been calculated that the absence of such an IDMS results in 2.5% inefficiency in knowledge activity[2]. Our competitors are adopting this "smart working" for competitive advantage.

Figure 1 illustrates how efficient data management supports the different project phases of a typical Highways DBFO project. The stepped red line illustrates how data used during each project phase is typically not carried over to the next phase where an information management strategy is not in place. At the end of each phase data there is an initial "start-up" period as information has to be recollected, which represents potential extra cost to the business. The orange line illustrates a continuous flow of data which continues through all the phases seamlessly. Efficient data management therefore

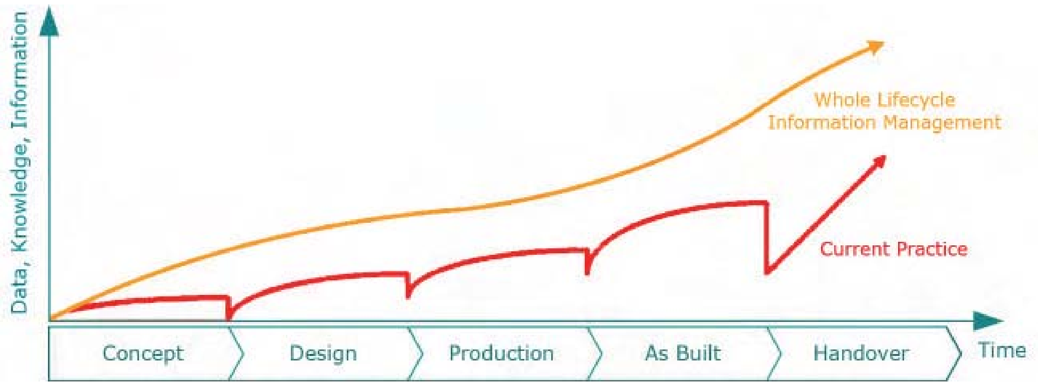

FIGURE 1. WHOLE PROJECT LIFECYCLE DATA MANAGEMENT

provides cost savings in the collection of data and the continuation of the use of data through project phases.

The Highways Agency (HA) has identified that the benefits of implementing such a data management strategy then encompass:

- Ready transfer of data to HA and/or Managing Agent asset management systems, post construction, leading to greater efficiencies and quality control
- Supports HA Business Initiatives and the delivery of the Ten Year Plan by facilitating:
  - Partnering
  - E-delivery
  - Asset knowledge
  - Best Practice
  - Development
  - Exchange

Investing in a data management strategy enabled by GIS provides returns throughout the project lifecycle and is relevant to all of the participating disciplines. Figure 2 illustrates the benefits in relation to all the project phases.

In Atkins’ case, we now have a project-ready infrastructure, in the form of the SDI, and expertise around the company to establish best practice data management, providing the key stakeholders within the project fully

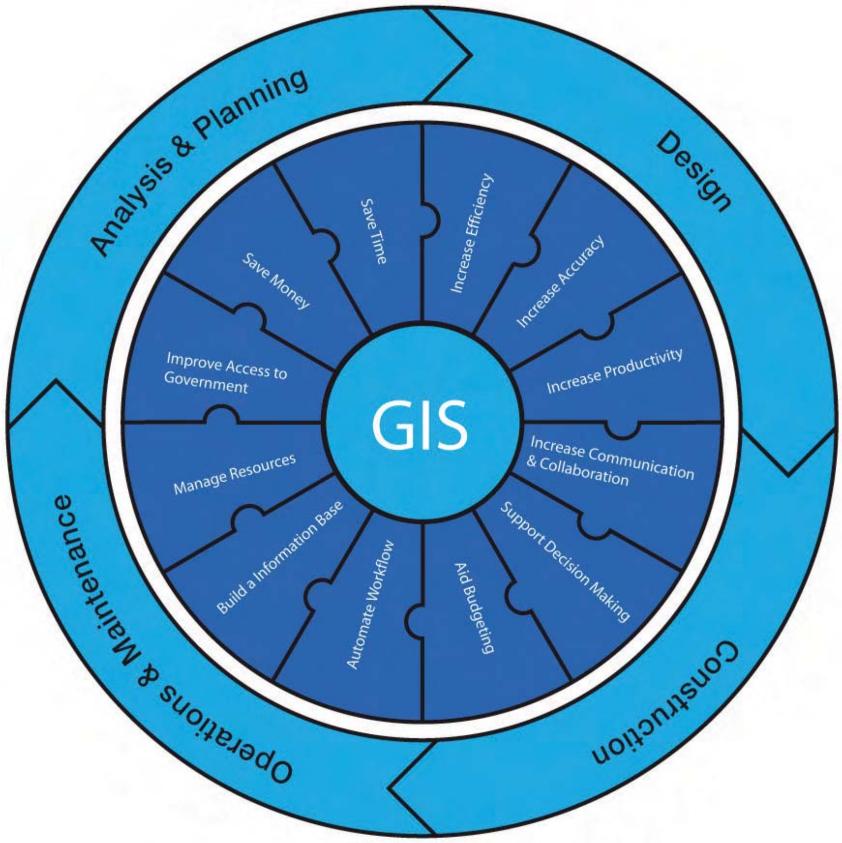

FIGURE 2. BENEFITS OF GIS-ENABLED DATA MANAGEMENT THROUGH THE PROJECT LIFE CYCLE

HIGHWAYS

support and champion the initiative. Early adoption maximises the return on investment to the business, and the probability of a successful outcome to the implementation. Experience on previous major projects, such as A3 Hindhead, show that delay can result in:

- Abortive work and hence delays due to working on out of date information
- Poor team relations and lowered morale
- Uninformed decisions and duplicated work

Our experience in Metronet, where we have developed and implemented an IDMS (analogous to our own SDI), demonstrates that once good working practices for data management are implemented, then the primary role of the SDI can be to facilitate data access and interpretation. To date, many companies, including Atkins, have been poor in the strategic use of the information they hold. On a corporate level, available software often is not, or can not be linked to provide management with "the whole picture". On a local level, departments and discipline groups have developed their own, informal datasets which are not shared with others. This situation can and does lead to duplication of effort, conflicting interpretation of information and the inefficient use of resources. By linking discrete datasets, the Metronet IDMS not only allows visibility of all the information from one portal, but allows the user to query the combined information, and use the results to inform better monitoring and planning. With the IDMS established as the single source of truth, further opportunities are being identified, such as supporting the Management Fault Control Centre, and use in other data hungry projects such as maintenance works, and portal for data collection from the new Asset Inspection Train. Early indications are that the same will happen with the Atkins' SDI as uptake gains momentum.

Due to the horizontal nature of the technology there are benefits to all disciplines. When building up the bid costs it is therefore important that this is taken into consideration so that costs are not duplicated. Some of the project business benefits can be divided into the following sub-headings:

Risk

- Reduce the use of poor quality data and potential misuse of information
- Reduce litigation risk because data is properly tracked and auditable
- Improve data security

Design

- Rapid easy access to multiple project wide data sets
- Combined access to different disciplines data
- Reduce duplication of data
- Geographically manage public involvement (e.g. complaints and communication)

Operation

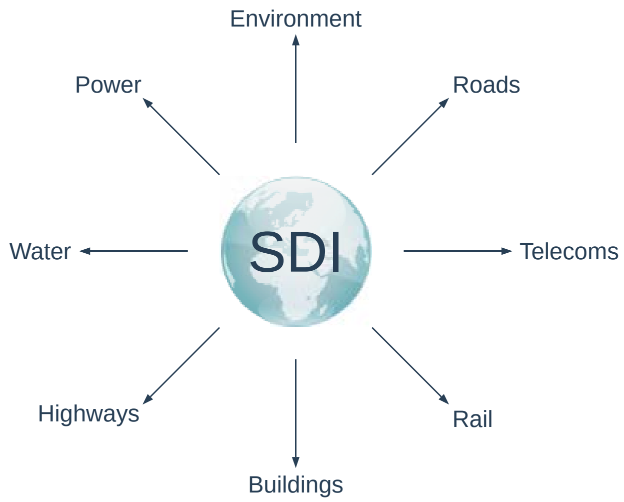

FIGURE 3. LINKING OF DIFFERENT DISCIPLINES VIA ATKINS' SDI

- Efficiently track and manage assets
- Efficiently plan future work and manage maintenance schedules

Cost

- Efficiently plan future surveys. Reduce chance of re-survey
- Collaborative works schedule
- Capitalise on data investment. Reuse data where appropriate

These benefits translate to a number of direct cost savings. These include:

- Less time spent searching for data. Organising existing and future data in a managed way (including collecting and storing metadata) will make it clear what data is available, where it came from and what it can be used for
- The same data can be used for a number of different applications, reducing the chance of multiple purchases
- Data delivery is more efficient reducing the need for costly desktop licences to view and interrogate data
- Information is shared and only needs to be captured once to provide benefit across the disciplines

2. Building blocks for Best Practice

From the discussion above, it should be clear that, in many Atkins' projects, the entire project lifecycle is dependent on having easy access to accurate data from many different disciplines. Managing data in a standard way benefits all the disciplines through all the project phases. Information can then be seamlessly used during all project phases and is easily accessible to all the different disciplines. This represents efficiencies and cost savings in a number of different areas.

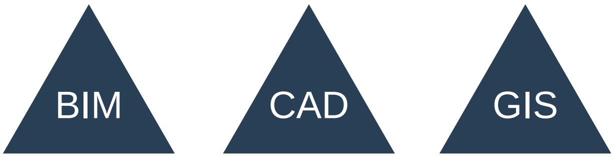

FIGURE 4. SILOS OF TECHNOLOGIES: STAFF, DATA, SOFTWARE, WORK FLOWS

To achieve Best Practice for Integrated Data Management in Atkins, it is proposed that the key building blocks should be:

- Establish the Strategy
- Adopt an Integrated Data Management Team
- Use the Atkins' SDI and compatible technologies

The Data Management Strategy is the key to ensuring the quality and consistency of data. It sets out management controls establishing the systems and software to be implemented, the "approved" sources of data and identifies data providers who are individual experts in their disciplines, and who will take ownership of the data, and the team who will be responsible for collating, organising, maintaining and publishing the assured datasets to the Atkins' SDI.

To establish the Integrated Data Management Strategy, it is vital to achieve the following:

- Buy-in and championing by the key stakeholders at all levels of the Data Management Strategy: Project Directors, Project Managers, Team Leaders, clients
- Collaboration of all Project Team members to engage a spirit of co-operation and sharing – and enlist "the speed of trust"
- Implementation of Integrated Data Management Team at the very start of the Project Life Cycle: integrate the CAD/GIS/Document Managers into one Team and fully integrate this Team with the Project Team – not as an aside

One of the biggest concerns is that we operate, not only in silos within our businesses, but within technologies as well. Design data (often in the form of CAD or – looking forward, BIM), spatial data (often in GIS), and descriptive data (often in spreadsheets or documents) mean that related information is locked in silos. The key to the success of sharing data is the business processes used to create, maintain and store data, and it is critical that the CAD standard and the GIS standard align. Within Atkins, there exists a strong CAD base-line for creating drawings and design concepts, but the CAD model is limited both for more analytical work or intelligent attribution, and for supporting future applications (e.g. interactive public consultation events, thematic mapping, and spatial analysis). The ultimate aim of the Atkins' SDI is to unlock the barriers, such that the strengths of each technology can be fully realised.

To overcome the existing silos, one of the key building blocks is to form an integrated data management team, who are an integral part of the project team. It is vital that the key stakeholders of the project support this team, and ensure that it is regarded as core to the project.

The Integrated Data Management team should comprise the following roles:

Integrated Data Manager

- To assist in formulating the scope of the Integrated Data Management and SDI, at bid stage, to include:
  - Configuration and implementation of the data store
  - Development and configuration of functionality for end users where necessary
- To agree and implement standards and procedures to facilitate data compatibility and interoperability between project information and management systems which may be required for a given project e.g. Arenium, BuildonLine, Business Collaborator, Sharepoint, iPronet, iCosNet. It is very important that careful consideration is taken of what systems are used at bid stage – integration with non-compatible systems can be costly and time-consuming, and should be avoided
- To own and manage the integrated data strategy
- To collaborate with the Document, GIS and CAD Managers to develop procedures and workflow for Data Management
- To liaise with data providers and ensure that the most up to date data is stored within the SDI
- To manage the provision of data to the Integrated Data Management team, agreeing schedule and procedures for data uploads with project managers and discipline team leaders, and report on progress to the Project Director/Manager
- To ensure that all relevant spatial and other data can be integrated and linked to the SDI and displayed on the Project's Web Portal (as provided by the SDI) as appropriate
- To train staff in the use of systems, standards and procedures
- To collaborate with information managers from other organisations involved in the project to provide a smooth workflow

CAD Manager

- To manage and maintain the CAD data repository, ensure that standards are adhered to.
- To process and load data as required.

GIS Manager

- To manage and maintain the GIS data repository, ensure that standards are adhered to.
- To process and load data as supplied.

Document Manager

- To manage and maintain the document repository.
- To process and load data as required.

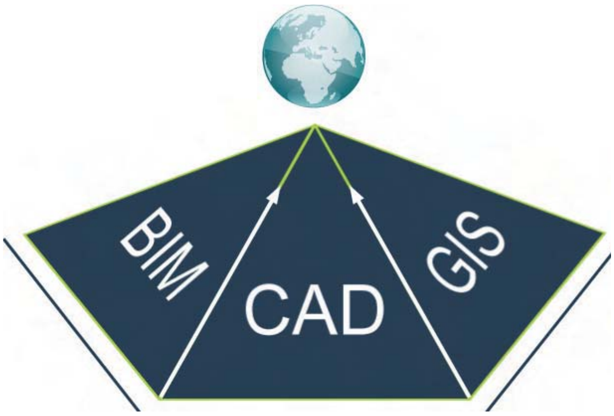

FIGURE 5. INTEGRATED DATA MANAGEMENT: ALIGNED STAFF, SOFTWARE, DATA, STANDARDS AND WORKFLOWS

Software Developer

- To liaise with the project stakeholders, and ascertain business requirements.
- To undertake customisation of the SDI, if required and as appropriate for the project.
- To enable access from relevant parties within the project and wider.

It is critical that the CAD standard and the GIS standard align. Previous experience has highlighted major issues where interoperability between CAD and GIS is not investigated. A lack of consideration creates problems which hinder both technologies.

It is vital that careful consideration is taken of what systems and software are used on the project at bid stage. Integration with non-compatible systems can be costly and time-consuming, and should either be avoided or taken into adequate account in costing the proposal. The Atkins SDI has been specifically developed to allow integration with the widest range of systems using open standards, but there can be cost implications in customising this integration.

The size of the team, and nature of the roles required will vary according to the size and complexity of the project. Some of the roles above may be combined, or may be taken by other project team members, if appropriate staff and capacity are identified. This should be established at bid stage, and a review should be undertaken to ascertain the ongoing operational needs of the project team through the life cycle of the project.

It is very important to realise that data management is an ongoing service requirement, which should be embedded as part of the overall management of the project, as mandated by the Project Director and controlled by the Project Manager.

3. Initial roll-out of Atkins' SDI

The objective of these first steps towards Best Practice in Integrated Data Management is to achieve minimum disruption of the project team, while achieving optimum gain. At this stage, many of the traditional project activities, including CAD and design, will continue as before, but the standards will be adjusted to allow alignment between the information and work flows as required. These adjustments will be established by the Integrated Data Management Team and approved by the Project Manager and Director.

The infrastructure supporting the Atkins SDI is an externally hosted, secure GIS platform running Autodesk MapGuide Enterprise on top of an Oracle Enterprise database. It is serviced by a fully robust and expandable infrastructure that can be easily extended to maintain high levels of performance as usage and data volumes increase. It has been specifically designed to use Internet technologies that allow secure collaboration across corporate boundaries.

The main benefits of using the Atkins SDI are:

- It supports the key architectural features required for the basic building blocks of integrated data management in an infrastructure consultancy
- It is an established infrastructure that requires minimal configuration for basic data management within a project
- It is fully supported by Atkins Geospatial as a strategic corporate system
- It is an enterprise system designed specifically to support key projects, clients and partners

More information about the Atkins' SDI and its future potential is provided in Appendix 1.

In these first steps towards Best Practice for Integrated Data Management, however, the Atkins' SDI will allow and control access to the project's centralised data store. This will improve the understanding of the spatially orientated features, clashes and restrictions which, in turn, can facilitate improved co-ordination between the many disciplinary aspects of the project. Only those users authorised to view specific data sets will be able to access them.

The use of the Atkins' SDI will also facilitate data held on network drives to be linked to spatial locations, thereby allowing data owners to manage and publish their data to known controlled locations, enabling the sharing and collaboration of their data with other disciplines. The information management strategy will dictate the processes required for the publication of data to the SDI.

This will enable disciplines to continue to manage and store their data in similar ways to the way they do currently, with the added benefit of providing their data to other interested parties.

In the future, the SDI can facilitate further steps to smarter working, including full integration of CAD and GIS, 3D Design and Analysis, and Visualisation. For instance, a range of user specific tools such as Environmental Information System (EnVIS) or Drainage Information System can be developed. Both systems store HA asset data for the environment and drainage disciplines respectively, according to a specification developed by the Highways Agency for recording and managing any changes to environmental or drainage data during any highways scheme. The development of an EnVIS tool using Topobase would allow for online editing of environmental attributes and spatial objects according to EnVIS standards within a controlled environment which would reduce duplications, errors and additional processing to convert from and to an EnVIS format. The data would be managed within an EnVIS environment using web editing tools.

4. Summary

- All of our work is dependent on access to up to date and accurate information, most of which can be linked to a geographical location or feature
- The implementation of a Data Management Strategy enables data to be managed according to standards and facilitates improved collaboration and data sharing across disciplines
- GIS is a technology enabler to efficient management and analysis of spatial data, and can therefore provide the supporting technology for the implementation of an data management strategy
- Development of a Data Management strategy supported by a GIS infrastructure, should be regarded as Best Practice, as reflected in BSI PAS 55:2008, to help structure the vast amount of data accumulated during highways projects, and to streamline project management, access to data and spatial analysis
- The Atkins SDI has been developed to support this vision
- Toolkits can be developed to help automate repetitive processes such as analysis and reporting

FIGURE 6.

This streamlines working processes and can reduce the risk of human error in the analysis of data.

- The Atkins SDI can link to compatible technologies to facilitate smarter working such as 3D Design
- Investing in data management now will provide benefits during the entire project lifecycle

In summary, the first steps to achieving the goal of Best Practice of Integrated Data Management, then, are to ensure costing for Integrated Data Management at bid stage, establishing the Strategy and adopting the Data Management team at the very beginning of a project, with responsibility for the management and dissemination of information for all parts of the project, through the use of the Atkins' SDI and compatible technologies.

Appendix

What is the Atkins' SDI?

Imagine seeing the Zayed City project in a Google Earth environment, with the ability to fly around the project area, looking at the project against base mapping, aerial photography or satellite imagery, and then zooming in, clicking on a building and linking to existing and active spreadsheets, asset databases, photographs, live feeds, such as CCTV, or pedestrian counts, associated with that building. The SDI can provide a fully integrated web-based information service and is the means to deliver integrated information management. It can also represent a major gear change through CAD/GIS integration.

It is very important to understand that the GIS in the form of the SDI does not merely involve the IT infrastructure – critical to the success of the implementation are the people delivering and using it and the applications which hang from it. Data management, workflow analysis, cultural change management and training are all important elements of effective implementation. Although the SDI can be a central repository for information, its key strength is that it can realise the "single point of truth" through its ability to link and integrate with the existing data stores.

SDI capabilities

The SDI allows the establishment of a central repository for spatially related data with a link to other data and information stores such as document management systems.

These could include the following data:

- Base topographic mapping and aerial photography
- Engineering, environmental and geotechnical baseline datasets
- Project specific investigation datasets
- Project specific design information
- Project specific construction records

HIGHWAYS

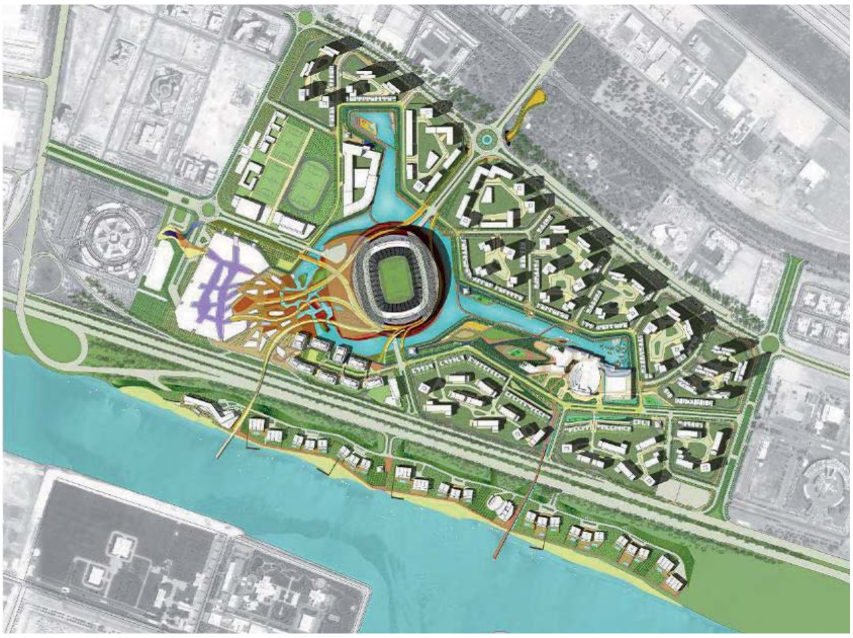

FIGURE 7.

- Project specific as-built records

The SDI allows a visual, synoptic view of spatial data including a visual index to key documents leading to:

- Better understanding of data held
- Easier identification of data gaps
- Understanding of data inter-relationships

The benefits of the Atkins Spatial Data Infrastructure are:

- Data management - The SDI data management service offers users the ability to store and access definitive copies of their geospatial data. Important metadata (or data about the data – for example source, usage restrictions) can also be captured
- The primary benefit is that a definitive copy may be stored once and used many times on different projects – increasing collaboration and reducing storage costs.
- Data discovery - The SDI data discovery services help users to understand what data is already available – reducing possible duplication of purchases, and increasing data reuse
- Web services and delivery - The SDI web services offers many reusable web services, such as geocoding, and data export. Furthermore, it can incorporate data into other business applications and can integrate data sets with Google Earth, Google Maps, and Microsoft Virtual Earth
- The SDI web delivery service allows data to be published on internal or external web-site thereby reducing the need to publish data to CAD files or PDFs to share and collaborate with clients and colleagues
- Enterprise applications - A large opportunity has been identified in rolling out integrated CAD/GIS via the SDI, to streamline the work processes of large multi-disciplinary projects. CAD and GIS users will be able to collaborate and share data, whilst simultaneously sharing the data with clients and colleagues

Better surveying, design and analyis

From surveying to design and advanced analysis, Atkins' SDI can streamline and accelerate workflows by giving you access to packages which offer purpose-built tools for automating time-consuming tasks and predicting project performance, while maintaining the link with the "Single Point of Truth".

Better co-ordination

Atkins' SDI, by linking its Single Point of Truth Portal with industry-leading software, can enable your entire team to work from the same consistent, up-to-date model so they stay co-ordinated throughout all the phases of the projects, from survey to construction documentation. But it goes further than this – it can then pass this entire information package across to the Operations and Maintenance Teams. By providing a scalable approach to data management and team co-ordination, the Atkins' SDI aims to address the widest needs of the organisation, from individual projects, through business units to corporate and client requirements, regardless of size or organisational structure.

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A new approach to legally protected species: Ecology mitigation for transport schemes that need not cost the earth

029

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Abstract

In 2007, the UK amended its legislation with regard to European protected species, such as great crested newts, otters and bats. This strengthened the protection of these species, with the potential for more on erous demands on a wide range of schemes, which in turn has the potential for higher capital costs and significant time delays. This article describes Atkins' approach to this new, stronger legislation, with refer ence to specific schemes, and seeks to demonstrate that, with a fresh look at the legislation and a pragmatic approach, taking account of legally protected species in any development scheme need not cost the earth.

1. Introduction

In 2007, the UK was obliged by Europe to amend its legislation with regard to European protected species, such as great crested newts, otters and bats. It will not come as news to anyone working with the built environment to read that these amendments to the Conservation Regulations strengthened the protection of these species, with the potential for more onerous demands for mitigation on a wide range of schemes. Such demands may result in higher capital costs than originally forecast and may require licensing, which can result in significant time delays and lead to further increases in cost.

Atkins has studied the legislation carefully, in consultation with an environmental lawyer. Whilst it is clear that the law is now stronger, in England and Wales the amendments also allow a distinction between high and low level disturbance and place greater emphasis on the overall maintenance of the habitat quality and the local species population than the original legislation. In Scotland, however, no such distinction is made and the full force of the European protection is felt. Therefore, across the UK, this requires a different interpretation of the law and a new approach to dealing with species protection.

Our friend the great crested newt is protected by this legislation; seemingly ubiquitous in England and Wales, this species is rare in Scotland and across the rest of Europe. Hence we have the sometimes dubious pleasure of being a stronghold for this species and are responsible for the maintenance of its population in Europe.

FIGURE 1. GREAT CRESTED NEWT

Small newts; big bill

There are numerous examples of the high cost of taking account of great crested newts in transport schemes, for example, the A47 by-pass in Leicestershire cost an extra £1.2 million and found 20 great crested newts. The A1157 improvement project in Cheshire cost an additional £300,000 in fencing, trapping and monitoring and found 15 great crested newts. At the every least, this is thought provoking.

However, Atkins has recently completed a number of schemes where a combination of careful consideration of the legislation, a sound understanding of the development proposals and knowledge of species ecology has resulted in a more rapid start to the project and less capital mitigation costs. We have been able to devise appropriate mitigation schemes that have not needed licensing, thereby saving time and money.

FIGURE 2. A66 POND

For example, we avoided the need for a licence for a new VMS scheme in East Anglia. Although the removal of suitable habitat was required, our assessment indicated that if certain measures were carried out, the works would be reasonably unlikely to cause an offence. Therefore, instead of a licence application and the associated delays, a combination of hand searches and appropriate methods of site clearance allowed the works to progress on programme and without significant extra expenditure. This approach has also been used on other maintenance schemes and widening schemes.

On a more strategic note, this scheme did not require hundreds of metres of plastic fencing, plastic bucket traps or daily visits for 30 days, thereby reducing the amount of materials and carbon expended in the implementation of the mitigation scheme.

Other European protected species that can be affected by road schemes include otters, bats and dormice. Atkins' ecologists have worked on mitigation for these species for a number of road schemes and aim to provide robust solutions that are proportionate to the specific scheme and that will stand the test of the legislation.

Ecology in Atkins

The Atkins ecology team has over 70 staff, based in offices around the country. We have delivered many challenging projects that range from small scale highway maintenance works to major construction schemes for government agencies and for local authorities. Our team has extensive experience in helping our clients to fully understand and comply with this legislation, providing specialist advice to deliver appropriate ecological solutions.

We pride ourselves on our pragmatic client focused approach in the balancing act between ecology and development. We are always delighted to hear from colleagues, to hear of any problems you may be experiencing with your schemes and to discuss potential solutions.

FIGURE 3. REEDMACE IN POND

Puny Drain diversion, King's Lynn, UK

030

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Abstract

The Puny Drain is an Internal Drainage Board main drain which collects highland water and agricultural runoff from an area of some 13.5 km2 to the south of King's Lynn, Norfolk. Until recently the existing main outfall for the Puny Drain was via an open channel and tidal sluice gate situated within the South Lynn urban area of King's Lynn. The operation of the existing tidal outfall would have been subject to sea level rise reducing by degrees, the quantity of water which could be discharged through each tidal cycle. In addition the South Lynn area is subject to major redevelopment increasing the amount of surface water runoff to the Puny Drain further reducing available capacity.

The paper details the planning, designing and construction of the Puny Drain Diversion.

1. Planning

To address these issues it was necessary to develop proposals for an alternative drainage system. The prefered scheme comprised a new open cut watercourse and outfall for the Puny Drain south of King's Lynn, which included a new pumping station to give a totally pumped drainage system. In order to assess the current drainage capacity of the Puny Drain and to develop an alternative system a detailed hydraulic model of the watercourse was constructed using the MIKE 11 hydrodynamic modelling software and the hydrology of the catchment assessed. For the hydrological assessment, flows were generated using the Flood Estimation Handbook (FEH). Using this model plus the IDB's local knowledge of the area, the interaction between the Puny Drain, the River Great Ouse and the Ouse Flood Relief Channel was established. This information was then used to model the current situation and a proposal for the Puny Drain to assess the benefits.

Following the detailed modelling work an option appraisal of the proposed solution was undertaken in consultation with the client, the East of Ouse Internal Drainage Board, The Middle Level Commissioners (IDB consulting engineers) and key stakeholders Network Rail, the Environment Agency, Norfolk Wildlife Trust and Natural England. The consultation enabled this process to determine the optimum solution for diverting the watercourse in terms of location of a new channel and how to overcome the technical challenges of crossing major rail infrastructure, the River Nar and local roads.

There were also significant environmental challenges to the proposals as the Puny Drain crosses a county wildlife site and there were recorded populations of nationally protected species, including badgers and water voles. The River Nar is also classified as a SSSI. The study sought to mitigate against negative impacts to these environmental receptors and to identify areas of potential enhancement to the system.

2. Design

Following the outline design and agreement of a preferred solution a full detailed design of the scheme was undertaken. The pumping station comprised 3 suspended bowl pumps, 1 duty and 2 standby pumps, giving a total discharge capacity of 4m3/s against a maximum static head of 5.4m. The pumping station was also designed to be fully automated and telemetry controlled and included a standby diesel generator for emergency power supply.

The design of the pumping station was optimised using the relationship between the pump discharge capacity and the available storage within the IDB drainage network. The total discharge capacity was reduced by increasing available storage within the new watercourse and through channel improvements to the existing Puny Drain. The final achieved standard of flood protection for the Puny

WATER & ENVIRONMENT

FIGURE 1.

FIGURE 2.

Drain catchment equalled a 1 in 50 year (2% annual probability) for the arable catchment and a 1 in 100 year (1% annual probability) for the urban areas, including allowances for climate change. During the design the local development plan for the study area was also used to identify future development within the Puny Drain catchment so that suitable allowances could be included.

As previously stated there were also several major obstacles present along the chosen route to the new diversion scheme. These comprised the main King's Lynn to London railway line and the River Nar which had to be crossed by the new channel. To achieve this siphon structures were designed to allow flows to pass beneath these obstacles.

The most challenging of these obstacles was the railway line crossing. The design process had to include production of a comprehensive Asset Management Plan and had to pass through Network Rail approvals process which informed the construction method for the crossing. For the railway crossing the siphon structure was designed based on using a "no-dig" construction method to ensure no disruption to the railway. The final agreed solution used a tunnel boring machine and pipe-jack to install the twin 1.5m diameter pipes 12m beneath the railway line. The design of the railway crossing was developed to maximise the use of the permanent structure for the pipe-jacking method thereby reducing construction temporary works. This was achieved through designing "header" and "reception" shafts to form the upstream and downstream vertical arms of the siphon. The pumping station discharged water into the upstream "header" shaft until a sufficient head of water was created to drive the siphon, thereby transferring water through the twin pipes to the "reception" shaft and out to the receiving watercourse. These shafts were increased in diameter to 8m for the "header" shaft and 6m for the "reception" shaft to allow their use as the drive pit and reception pit for the tunnel boring and pipe jack operation thereby removing any requirement for a separate 12m deep temporary pit. The shafts were also located outside of the railway line zone of influence so that only the tunnelling operation would impact the railway reducing the Network Rail approvals process.

Alongside the design a detailed Environmental Scoping report was produced for consultation with the statutory consultees and key stakeholders. As part of the scoping report comprehensive environmental site surveys were undertaken to determine the baseline environment of the area.

The design was appraised for impacts on the local ecology and to identify possible enhancement opportunities. Potential impacts on the local wildlife were identified and mitigation measures incorporated into the design of the scheme. The work formed the basis of a full Environmental Impact Assessment of the scheme which accompanied a full planning application.

3. Construction

Following a successful planning application, construction of the diversion scheme commenced in September 2006.

The construction of the new cut channel consisted of 1.6km of new open watercourse and also 2km of channel improvements to widen and deepen the existing Puny Drain. It was felt that if not managed well these improvement works could give rise to significant environmental risks. In essence it was necessary to maintain a live watercourse for drainage and flood risk management purposes as well as protecting existing environmental features and nationally protected species throughout the duration of the works. This was managed through the implementation of an Environmental Action Plan and Ecological Mitigation Plan developed during the detailed design and adopted by the Principle Contractor, May Gurney.

These detailed plans allowed May Gurney to develop a sectional construction process whereby short sections of the drain were dammed and overpumped to allow works to proceed whilst the rest of the drain was left undisturbed. May Gurney employed an Environmental

FIGURE 3.

Permit to Work system which involved undertaking comprehensive ecological surveys to ensure the dammed section was clear of environmental hazards and signed off before works could commence. The works progressed along the drain with one section being completed and fully reinstated before commencing on the next section. This ensured that environmental mitigation and protection measures could be implemented before construction works were undertaken.

In contrast to the railway crossing which utilised a "no-dig" construction method, the River Nar siphon was installed in open cut using a cofferdam to cross the river channel in four separate stages and maintain river flows and flood protection at all times. Stage 1 started on the eastern bank of the River Nar and laid the single 2.2m diameter pipe beneath the eastern River Nar flood embankment. For this phase the flood embankment was realigned around the cofferdam to maintain the flood defence protection. Once stage 1 was complete the cofferdam was removed and the flood embankment re-established on its original alignment. Stage 2 of the works continued installation of the pipe up to the eastern bank of the river channel.

Stage 3 of the work was the crossing of the River Nar main channel. During this stage a cofferdam was constructed across the river and a temporary bypass channel was excavated in the eastern flood berm over the top of the pipe section installed in stage 2. This ensured that the River flow was maintained whilst work was carried out in the river channel. During stage 3 the western flood embankment was also realigned so the pipe could be continued into the land behind the original flood defence line. Stage 4 of the works saw the removal of the stage 3 cofferdam, the reinstatement of the River Nar main channel and removal of the temporary bypass channel. Following realignment of the western flood embankment back to its original alignment the remaining length of pipe was completed to the outlet headwall structure into the receiving new cut watercourse.

As the River Nar was designated as an SSSI in this location, close co-operation was required with both the Environment Agency and Natural England over the construction methodology to develop an agreed approach to the river crossing from both flood defence and environmental impact considerations.

FIGURE 4.

FIGURE 5.

FIGURE 6.

This process again utilised the detailed Environmental Action Plan and Ecological Mitigation Plan to manage and control the environmental risks associated with crossing the River Nar.

The construction phase was managed under the NEC Engineering and Construction Contract Option A with a total value of £5.5M. The project was successfully completed and the pumping station commissioned in March 2008. A presentation of this scheme was given by the author at the Catchment 08 Exhibition on 17 September 2008.

The paper was published in the Association of Drainage Authorities Gazette, winter 2008.

Technical report by Traffic Wales into cause and subsequent analysis of the reasons for the failure of the tunnel fire main at the east portal of Conwy Tunnel

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Abstract

Following the maintenance replacement of several water main isolation valves and the addition of a non re turn valve in the Conwy tunnel fire main in 2007, several failures occurred whereby large quantities of water were discharged into the tunnel mid river sump causing increased pumping rates. The paper investigates the cause and details the action taken to alleviate any re occurrence. The paper highlights the fact that the failures were due to "water hammer", the result of which was compounded by the inclusion of the non return valve.

1. Background

The existing fire main serving the Conwy Tunnel was installed as part of the civil engineering works for the project and consists of ductile iron piping and eight associated sluice valves for system isolation. The pipe work and valves are connected using proprietary Viking Johnson (VJ) type couplings whilst at the tunnel element joint locations, flexible "Andre" bellows are fitted to cater for any movement of the tunnel elements. The fire main is supplied from and branched off the Welsh Water Authority (WWA) potable water main which runs along the Causeway and is sourced from the Cowlyd reservoir in the Conwy valley.

The tunnel point of supply is a WWA isolation valve located adjacent the Causeway and feeds the Conwy Tunnel fire main system a 250mm diameter pipe. Each tunnel bore is then supplied via isolation valves by two sub main pipes each of 200mm diameter. The general layout is shown in Figure 1.

The arrangement forms a closed loop which allows for flexibility in carrying out isolation of sections of the fire main in emergencies or as a result of maintenance requirements. Hydrants tapped off the tunnel fire main are located within carriageway hydrant chambers and are fitted with standard screwed couplings for the use of the Fire and Rescue Service.

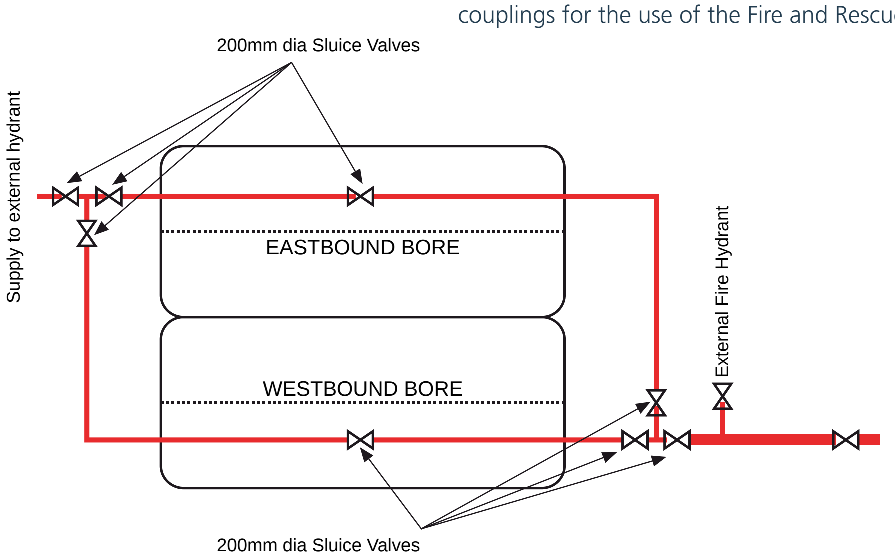

FIGURE 1. SCHEMATIC LAYOUT OF FIRE MAIN PRE-FEBRUARY 2007

WATER & ENVIRONMENT

Annual flow and static pressure measurements are taken at each hydrant outlet to ensure compliance with BA72/03. Static water pressures in excess of 20bar are common in the water main and problems with hydrant leaks have been experienced since the tunnel was built. However, additional problems with the installation were experienced following the installation of replacement valves on the fire main in February 2007.

2. Introduction

Problems were identified by the RMC in late 2006 with two of the isolation valves, one at the east portal and one at the west portal. Due to the lack of access to maintain the isolation valves at both the east and west portals, works were carried out to replace the complete valve sets (3 at each portal) and locate them in new chambers for improved maintenance access. An additional non-return valve at the east portal was also added to reduce the chance of back flow into the WWA main, as shown in Figure 1.

Work at the east portal was carried out in February 2007 whilst the west portal was carried out in June 2007. The chambers were constructed and pipe modifications carried out to accommodate the new valve layouts. The arrangement at the east was improved so as to provide access to the valves without the need for TM. The non-return valve was located at the position shown above.

As a part of the NWTRA commission to ensure compliance with the Road Tunnel Safety Regulations 2007, it became apparent, from data collected for compilation of the Drainage Study, that the number of pump operations associated with the Conwy Tunnel mid river Sump 6 and Sump 5 which are located at the eastern end of the tunnel had increased substantially over the previous twelve months. Data indicated that over the period the number of Sump 6 pump operations had increased from what was a normal rate of approximately 6 occasions per day to 48 and Sump 5 had increased from nine to thirty three occasions. Traffic Wales were made aware of and shown a draft copy of the study report on the morning of Friday 11th April 2008 by the author and were surprised at what the data indicated.

Following verification of the present pumping rates, Traffic Wales contacted the Deputy Tunnel Manager to inform him of the findings. Investigation as to why both sumps were being affected was carried out by first checking the east portal drainage system drawings for the possible location of a source that was common to the collection area of both affected sumps.

The drawings clarified the issue in that both sumps could be affected by large volumes of water emanating from the area around the east portal westbound carriageway.

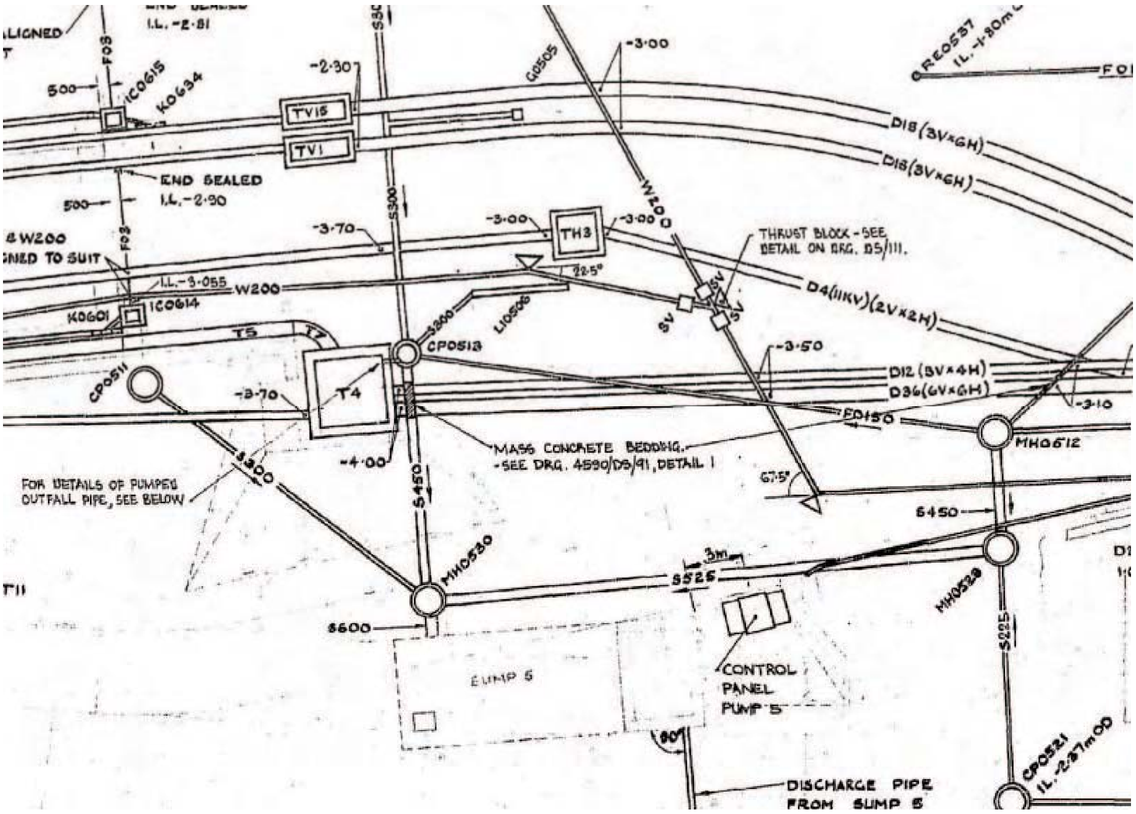

FIGURE 2. DRAINAGE LAYOUT AT EAST PORTAL

3. Summary of events

11.00hrs Friday 11 April, 2008

Site investigation by Traffic Wales and the author of the study report revealed what appeared to be unexpected flows of water emanating from a french drain which emptied into a drainage chamber at the portal area. This then discharged to Sump 5. A formation drain running across the carriageway at the east tunnel portal which drained into Sump 6 would be expected to intercept underground accumulations of water on the westbound tunnel approach. Clearly, from the data that had been presented in terms of pump activity in Sump 6, this pointed to the fact that water was leaking into the subsoil in the area local to the westbound bore at the east portal.

As works had been carried out on the tunnel fire main in that vicinity approximately 12 months previously, it was assumed that a problem resulting from those works may have been the cause of the leak, possibly due to a leaking joint.

14.30hrs Friday 11 April, 2008

The RMC were contacted at 14.30hrs and requested by Traffic Wales to attend the portal location where the tunnel fire main isolating valves are located and were instructed to listen to the valves using a "listening stick" for any audible indication that there was a flow running through the main. Confirmation was received that there was flow through the pipe. As the fire main is a dead main (no flow during normal operation) no flow should have been detected.

The RMC were then instructed to shut off the main and re-assess any flows. They confirmed that flow had stopped thus confirming that there was a leak on the fire main downstream of the point of isolation.

In reality the leak could have been at any "Andre" joint location, fire hydrant, west portal isolating valves or at any other joint along the fire main piping system – approximately 2km of pipe. This was discounted on the basis that if any leaks were present at locations other than buried sections within the tunnel they would have been visible on the carriageway particularly at the leakage rates that have been estimated.

FIGURE 3. SUSPECTED LOCATION OF LEAK - WORKS CARRIED OUT IN FEBRUARY 2007 SHOWING PIPING AND VALVE LAYOUTS AT THE EAST PORTAL

As extensive work had been carried out on the fire main isolation valve arrangements at the east portal fourteen months previously it was presumed that a connection on the new piping arrangement was the probable cause of the leak. (See Figure 3)

16.30hrs Friday 11 April, 2008

The sub-contractors (Daniels Ltd) who carried out the fire main piping modifications in February 2007 on behalf of the RMC were contacted and promptly attended site. They subsequently confirmed that the fire main was leaking but its cause was yet to be determined. Discussions on site between NWTRA/Traffic Wales/Daniels Ltd revolved around the immediate actions to be taken, the result of which was the carrying out of a dynamic site risk assessment. The outcome of the latter was to leave the fire main energised over the weekend until all parties involved could meet on Monday 14th April to agree a way forward.

A meeting was arranged for 10.00hrs on the following Monday at the NWTRA offices in St. Asaph between NWTRA, Traffic Wales, Daniels Ltd and the RMC.

10.00hrs Monday 14 April, 2008

NWTRA, Traffic Wales, Daniels Ltd and the RMC were represented at the meeting; however, a representative of Conwy County Borough Council traffic management division tendered his apologies. The background to the increased pumping at both the affected sumps, the available data to support the latter, the layout of the drainage and supporting photographs of the installation prior to reinstatement in February 2007 was explained to all by Wyn Roberts of Traffic Wales.

Having inspected the data, drainage layout and the results of the findings on site on the previous Friday, Keith Pickering (Daniels Ltd) intimated that the most likely cause of the leak was a displaced Viking Johnson (VJ) coupling adjacent a point where the new piping was coupled to the existing fire main. Its failure, if it was found to be the cause of the leak, was more than likely to be the result of water hammer.

It was accepted by all that the remedial works to fix the leak should be carried out as a matter of urgency. Daniels Ltd stated that they could commence works as soon as suitable TM could be provided. Unfortunately the DLO was not represented although keen to progress the issue. The A55 Route Manager joined the discussion in their absence to provide a way forward with the provision of TM. Due to a planned tunnel closure at Pen y Clip on the 15th – 18th April it was stated that TM could not be provided during that week and cyclic maintenance planned for the following week would only allow road space on the 24th April.

Due to the prolonged period of time up to when road space was to be made available, it was agreed that the safest option was to isolate the fire main until the 24th April. This resulted in actions being placed on Daniels Ltd and Traffic Wales; the former were asked to provide a large water bowser to be located at the West TSB compound and to isolate the fire main at the east portal. The latter were to inform the Fire and Rescue Service that the main would be isolated until works were complete. This arrangement allowed for a supply of water for the Fire and Rescue Service to fill up their tenders at the west of the tunnel and would leave a single fire hydrant available outside the east TSB for the Fire and Rescue Service to fill up their tenders at the east end of the tunnel. This was an arrangement that had been agreed with the Fire and Rescue Service during the works carried out in February 2007 which required similar isolation of the fire main.

10.00hrs Tuesday 15 April, 2008

A water bowser was located at the West TSB, the fire main isolated at the east portal and the Fire and Rescue Service informed of the status of the fire main via the NWTMC Control Room. The effect of isolating the fire main was monitored closely over the next 24 hours and it became immediately apparent via trending information on SCADA that the pump operations for Sump 5 and Sump 6 had returned to "normal" (equal to the number of pump operations pre the fire main works carried out in February 2007).

10.30hrs Thursday 22 April, 2008

Traffic management was set up to close lane 1 westbound and works commenced to excavate the area immediately above the location of the suspected failure point.

The source of the leak was located and was confirmed to be a leaking flanged joint on the westbound bore fire main.

FIGURE 4. SOURCE OF LEAK - FAILED GASKET

This was replaced and the fire main re-pressurised to check the joint for tightness. The repair held the pressure and was left overnight until carriageway reinstatement could be carried out in the morning. During the period when these works were being carried out, the valve bonnet cover gasket failed on the main isolating valve as seen below.

The reason for this cannot be attributed to the repair works but opinions are that it could have been caused by excessive air pressure resident within the fire main. This is due to the fact that the pressure of any air in the system would rise to a value in excess of the water pressure at the time due to its compressive nature.

Friday 23 April, 2008

Following inspection of the repaired joint, the excavation was reinstated and TM removed. A short meeting was held between Daniels Ltd and Traffic Wales to discuss the issues arising from the pipe joint failure.

FIGURE 5.

Following discussions with Daniels Ltd, who had taken expert advice on the probable failure mode, Traffic Wales were advised that the most likely cause was either air entrapment within the pipe and / or the effects of water hammer on the system. This confirmed the Traffic Wales assumption. It was agreed that the non-return valve upstream of the main isolating valve that was fitted as part of the works carried out in February 2007 should be removed as it appeared to be compounding the problem with the water hammer effects in such a way that any hammer pulses from the tunnel sent up the pipe were probably being reflected back off the non-return valve. It also had the effect of stopping any trapped air from migrating upstream to the WWA air valve on the causeway. To engineer out these problems Traffic Wales asked Daniels Ltd to provide a bobbin piece with a welded boss to which an air valve would be fitted to replace the N/R valve. This was to allow a release point for any accumulated air in that part of the system and allow for adjustments to make up any shortfall in the pipe lengths.

It was also agreed that the above works would be carried out as soon as the parts were available along with renewing the gasket on the main valve bonnet. The fire main was left isolated but the water bowser was still left in place at the West TSB and the single fire hydrant was available outside the east TSB for the use of the Fire and Rescue Service. The NWTMC control room were advised that the fire main was available in emergency situations and arrangements were made with the RMC on call engineer to be called if required to energise the main. The Fire and Rescue Service were informed of the situation. Traffic Wales and the RMC were informed on the 28th April that the works would proceed on the 8th May to repair the valve, remove the non-return valve and replace it with the new bobbin piece and associated air valve. An additional water bowser would be provided at the east TSB layby for the duration of those works (estimated at 2-3hours) when the fire main would have to be isolated.

4. Analysis of the effect of the fire main failure

The available data supports the fact that the average number of pump operations associated with Sumps 5 and 6 had increased during the period from February 2007 to April 2008. Sump 6 pump running occasions had increased by a dramatic 800% whilst Sump 5 had increased by 300%, this equates to a rise from 6 pumping events/day to 48 pumping events/day for Sump 6 and a rise from 10 pumping events/day to 30 pumping events/day for Sump 5. This can clearly be seen on the graph overleaf.

Calculations indicated that the inflow into the sumps was 6 litres/sec equating to a volume of 500m^3 / day. This has had the effect of increasing the Sump 6 running time by a factor of 12. Similar increase has been observed at Sump 5. Whilst no discernible effect

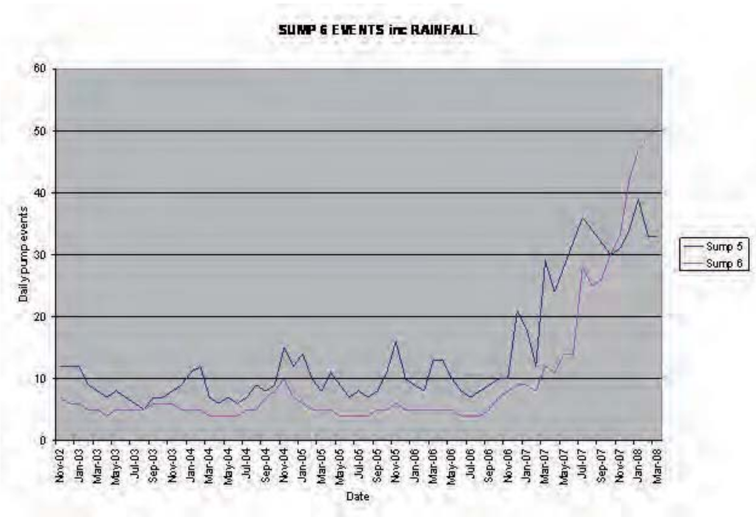

FIGURE 6. SUMPS 5 & 6 PUMPING EVENTS FOR THE PERIOD NOV 02 – MARCH 08

WATER & ENVIRONMENT

has been observed on the condition of the pumps or the associated electrical equipment, the running hours had increased over the period increasing the amount of power used and hence utility costs.

The cost of electricity associated with the increased running has been estimated to be in the region of £3,500+/year. Clearly this also has an environmental consequence in that on average over the year 300m^3/day of potable water has been wasted.

During the period between February 2007 and April 2008 routine tunnel fire hydrant flow and pressure tests were carried out without the knowledge of the leak as part of the planned maintenance and investigation of the recorded results indicate that hydrant outlet flows were within the requirements of BD 78/99. Static pressure readings were also at 14 bars. In hindsight the resultant leak therefore did not on its own compromise the fire main hydrant performance.

The number of pump running occasions and the correct sequencing of the pumps are recorded and filed daily by Traffic Wales control room operators. Whilst the increase in the pump running occasions is visible over the long term, the average increase in total pump occasions recorded per week over the year was discernible. To operators working various shift patterns the increase in pump activity from week to week in all probability was imperceptible. However, the data collected for the study was collated from the information available via the Traffic Wales shift reports.

In the event that both pumps were unavailable at Sump 6 and due to the additional flows from the leak, the containment period available at the tunnel mid river sump from the flows associated with the leak alone would allow for an impounding period of between 3 hours and 1¼ hours depending on the state of fill of the sump before road level would be reached. These periods would in reality be less due to the additional normal drainage flows into the sump.

5. Conclusion

Cause of failure

The consensus of opinion as to the cause of the various failures of elements of the water main experienced since the modifications were carried out is that attributed to water hammer and the possible presence of air in the system. The inclusion in February 2007 of a non-return valve in the system may have contributed to the problems being experienced.

A water-hammer (or, more generally, fluid hammer) is the repeated reflection of a pressure surge or wave caused by the kinetic energy of a fluid in motion when it is caused to stop or change direction suddenly.

The movement of liquid mass in a pipe is kinetic energy, which is proportional to the mass of liquid times the square of the velocity. For this reason, most pipe sizing charts recommend keeping the flow velocity at or below 5 ft/s (1.5 m/s). Typically, in a hard walled pipe the pressure wave of water hammer travels to reflection point and back at a velocity in the order of 1440 metres per second.

The size of the water hammer pulse can be estimated from Joukowsky's equation

P = a C (Equation 1) [1]

Where P is the magnitude of the pressure wave (Pa), is the density of the fluid (kgm^{-3}),

a is the speed of sound in the fluid (ms^{-1}), and

 C is the change in the fluid's velocity (ms^{-1}).

Note: The speed of sound in a fluid will depend on the fluid compressibility.

Using Equation (1) above:

Where = 1000 kgm^{-3}

a =1497 ms^{-1}

 C = 1.5 ms^{-1}

(assumed from flow rate of 33litres/second from one hydrant available from TW maintenance data)

P = 1000 x 1497 x 1.5

= 22.5bar @ velocity change of 1.5m/s – (more likely value)

Flow rates in excess of the above may cause a serious breach of the integrity of the system as shown below:

P = 1000 x 1497 x 2.0

= 30.0bar @ velocity change of 2.0m/s

P = 1000 x 1497 x 2.5

= 37.5bar @ velocity change of 2.5m/s

Using the following equation:

v = 4660/(1+KD/t) (Equation 2)

Where D = pipe diameter (200mm)

t = pipe wall thickness (5mm)

K = 0.01(for steel)

Then v= 4660/(1+0.01x200/5) = 3328m/s

The velocity of the pressure wave to travel from the furthest hydrant on the system to the valve cluster at the east portal would be 3328m/s and the resulting time for pressure wave to travel the length of the tunnel = 1000/3328 = 0.3 seconds

The origin of the hammer effect in this instance could be attributed to the rapid shutting of the hydrants within the tunnel with the rapid reduction in flow sending a shock wave up the pipe and then being reflected back off the non-return valve, ultimately causing a failure at the weakest point. The non-return valve in question has been removed and an automatic air valve fitted in its place as a means of displacing any accumulated air in the system.

Air could be resident within the system in two ways:

i. Air drawn in to the piping system following any draining of the main for valve maintenance

ii. Air in suspension in the water

The effects of air within high pressure water mains are well documented and the damage it can cause is clearly visible in this case. Quite simply, a pocket of air sandwiched between water within a pipe will be compressed if the water pressure rises. Static water pressure readings within the tunnel have been recorded between 14 - 20+ bar, the pressure of any trapped air would be in excess of this due to its compressive properties and more than likely in excess of the ratings of valves and fittings.

The effects of water hammer and air in the fire main could be mitigated by the following:

Lower fluid velocities – these are inherent due to the design of the system

Slow closing valves – change existing procedures

High pipeline pressure rating (expensive) – limited by existing installation

Inclusion of air vent valves – modification to system

Data provided as a result of the Drainage Study carried out by Atkins detailing the number of sump pump events for periods without rainfall supports the assumption that water hammer is causing the failures as the rate of change in the number of pump events changes in relation to the periods when tunnel closures took place. As the system was not drained during the previous 12 month period, the introduction of any air into the fire main can largely be discounted (any air resident in the system from previous occasions cannot be quantified). The only other cause to the failures can be attributed to the operation of hydrants by the tunnel washing contractors when filling up their water storage tanks.

Recommendations

1) It is proposed to modify the existing SCADA systems in the NWTMC control room to allow for automatic counting of pump running occasions (including correct sequencing) for each sump and provide suitable alarms on detection of excessive pump running events to aid the operator to take appropriate action. In the interim the control room operators require to be made aware of the consequences of increased pump running events and operational procedures updated. The existing SCADA caters for 14 days of displayed historical data. However, data is available via log files held within the ECS server which requires manual manipulation to access. Proposals require to be drawn up to make access to such files easier to access.
2) The planned installation of a pressure reduction valve on the customer side of the point of water supply on the Conwy Causeway will reduce the static pressure in the fire main on the customer side to more acceptable levels (6-8 bar) and will provide an opportunity to incorporate flow and/or pressure transducers for data input into the SCADA. This has yet to be discussed with Atkins/NWTRA. It is recommended that additional automatic air valves be installed at the valve location. Due to the high pressures experienced in the water main over a number of years and the recent incidents it is recommended that the works should progress as soon as possible.
3) The contractors carrying out the tunnel wall washing need to be instructed into the correct procedure for operating hydrants on high pressure water mains and their procedures amended to suit. The RMC was instructed that in future the hydrant located outside the east TSB was to be used for filling up purposes where practicable as this would reduce any adverse effects on the water main system.

Other observations

1) Repairs to the fire main were successfully carried out and the carriageway was reinstated on Friday 23rd April. Removal of the non-return valve, installation of the air valve and repair of the valve bonnet gasket were carried out. Traffic Wales are closely monitoring the situation in terms of further increases in pump running frequencies.
2) Traffic Wales made clear at the meeting of Monday 14th April the urgency to repair the leak due to its effects on the integrity of the pumping system. However, it transpired that the availability of road space dictated when the leak investigation and repairs could be effected.
3) Clearly the events detailed within this report support the requirements to design and operate high pressure water mains to current standards. The existing installation is not in compliance with the requirements of BD 78/99 and specific reference to the effects of air and water hammer within such systems is not documented within the latter. Amendments to BD 78/99 should be considered to reflect the hazards involved.
4) The results of the calculations carried out in this report indicate possible pressure surges within the system in the region of 22.5 bar which is in excess of what is understood to be the working pressure rating of the original pipe fittings and hydrants by 40%.

   Note: The new isolation valves that were fitted in 2007 were specified as 20 bar working pressure.
5) No similar failures have been recorded at the Pen y Clip or Penmaenbach tunnels which have similar fire mains installed. No specific research has been carried out to elucidate why the operation at those locations are different, but the incorporation of automatic air valves as part of those systems may be an attributing factor.

The original intent of this report was to provide a short brief to the Tunnel Manager and Tunnel Safety Officer into the cause and effect of recent instances of failure of certain components of the Conwy Tunnel fire main (which have not been separately identified as a "significant incidents"). The report still fulfils that provision, however, in the process of carrying out the technical research into the likely causes of the component failures it became apparent that the subject area warranted additional investigation and a comprehensive report. Clearly the subject area may be of interest to all tunnel operators who manage tunnels with high pressure water mains sourced directly from Water Authority supplies.

References:

1. TIJSSELING A S, ANDERSON A (2002). The Joukowsky equation for fluids and solids.
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Composites methods development under the next generation composite wing (NGCW) research programme

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Abstract

The Next Generation Composite Wing (NGCW) project was launched at an event at the Institution of Civil Engineers, London in May 2008 and has over £103 million of funding allocated to it by its various industry partners, and the Government's Technology Strategy Board, Regional Development Agencies and Devolved Administrations. The project is led by Airbus in the UK, with Atkins, as one of the industry partners, committing the efforts of three full-time engineers over a period of three years.

1. Introduction to the NGCW Programme

The project is part of Airbus' response to the ACARE (Advisory Council for Aeronautical Research in Europe) 2020 targets for a halving of civil aircraft CO2 emissions by that year relative to the standards of 2000. The introduction of carbon fibre reinforced plastic (CFRP) materials into the primary structure of the wing box is seen as a vital component of this response, as composites offer the potential for significantly reduced airframe weight when compared with more traditional aluminium alloys. The NGCW investment will enable aerospace engineers to develop techniques and tools that will give a better understanding of how they can use composite materials to best advantage in the wing of a civil transport. By developing standard analysis tools, our engineers also hope to open the doorway to the increased use of composites in other industries.

The NGCW project is divided into four components that focus on critical aspects of wing design and manufacture. The multi-disciplinary optimised wing (MDOW) component is the one on which Atkins has focussed its efforts, this package having the aim of producing a wing structure that is optimised to account for all aspects of wing design: weight, aerodynamics and systems integration. The remainder of the NGCW programme comprises HiVol (high volume production), which seeks to minimise the manufacture and assembly costs of composite wings, IntEq (integrated equipment), which examines the optimisation of fuel, hydraulic and electrical systems within the

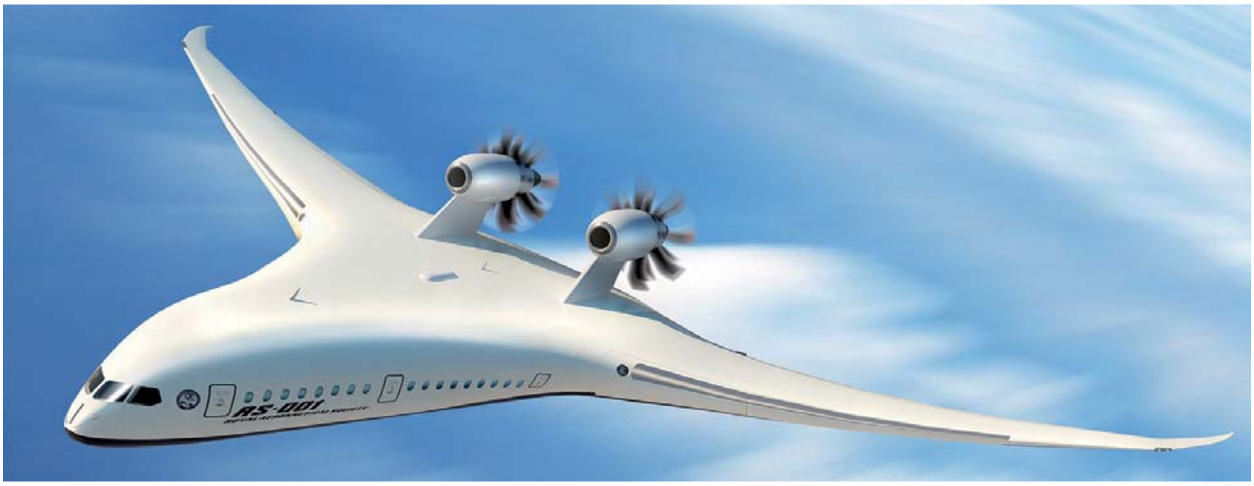

FIGURE 1. A CONCEPT FOR A FUTURE GENERATION CIVIL AIRCRAFT
(DIGITAL IMAGE BY KAKTUS DIGITAL (WWW.KAKTUSDIGITAL.COM) FROM ORIGINAL CONCEPT BY RAES

AEROSPACE

wing, and MINT (multi-disciplinary integration), which integrates the output from the other three studies into an analysis system that will deliver a highly efficient and cost-effective CFRP wing within the ACARE time scales.

2. Carbon fibre reinforced materials

CFRP combines high strength carbon fibres embedded in an epoxy matrix. When applied to aircraft structures, carbon composites are generally deployed as laminates: a stack of plies, or laminas, each ~0.125 – 0.25mm thick. A ply is supplied in uni-directional (UD) form: a sheet or tape of parallel fibres that has been pre-impregnated with resin that has yet to be cured. This form of the material is ideal for the manufacture of thin plates that are used so extensively in airframe structures. Manufacturers use tape-laying machines to lay down plies, one on top of another, to form single piece sub-components. Modern tape-laying machines can fabricate an entire wing skin in one piece, eliminating the fasteners that are routinely used in metallic designs and thus saving manufacturing time and cost, and component weight. To complete the manufacturing process, the component is cured within an 'autoclave', which subjects the component to pressure at an elevated temperature to consolidate and harden the layers of plies into a single monolith of carbon/epoxy laminate.

It is the fibres that give composite materials their potentially decisive strength advantages. By way of comparison for static strength, the ultimate strength of aerospace grade aluminium alloys is typically 450 megapascals, whilst that of a carbon fibre would be five times that value. In addition, fibrous composites are virtually immune from fatigue under in-plane loading, as the fibres arrest the growth of local defects before they can propagate. Again, aluminium alloys exhibit no endurance limit, so fatigue in metallic structures is an ever-present threat. As carbon composites are, additionally, only 60% of the density of aluminium, the potential for weight reduction in an airframe application is obvious.

To obtain the required multi-axial stiffness and strength within the structure, successive UD plies are orientated at a variety of angles – normally at angular increments of 45°. By judicious choice of angular distribution, the stiffness and strength of the final laminate may be chosen to match the demands of the local structure. For example, if high direct stiffness/strength is required in the 0° direction, some shear capability also (+/-45° direction), but not very much in the 90° direction, a lay-up of 50% 0s, 40% +/- 45s and 10% 90s might be selected (50/40/10).

Thus, carbon composites provide engineers with a means of reducing both manufacturing cost and structure weight. Clearly, however, the additional design freedoms extended to the engineers bring with them a more onerous analysis task, so that the optimal use of the material is more complex than in the case of metallic design.

3. Atkins' commitment to NGCW

Atkins is a core partner in the MDOW component of the NGCW programme, and has targeted two aspects of that study as its key contributions: investigation of the performance of novel laminates; and the rapid sizing of composite wing box components. These two technologies are aimed at the heart of optimised CFRP wing structures: how can the fibres best be deployed within a structure, and how can structural analysis be accelerated to allow investigation of the increased variety of structural options that the use of CFRP presents?

In terms of novel laminates, the Atkins work to date has concentrated on aspects of fibre angle, the distribution of ply angles, and a variety of aspects of inter-lamina strength (inter-lamina strength is a key weakness in composite materials). Rapid sizing techniques have concentrated on improving the idealisations applied to the analysis of wing box components and the introduction of composite integrity assessment techniques so that accurate sizes may be derived without the need to resort to time-consuming finite element analysis.

The target from the MDOW developments is the implementation of a process that will allow a complete sizing iteration for a wing configuration to be performed in less than 24 hours. Rapid sizing iteration is seen as a fundamental requirement of the goal of simultaneous multi-disciplinary optimisation that was a key message in the ACARE report of 2002.

4. Ply angle study

Traditionally, laminated composite materials have deployed UD plies in stacks with four associated orientation angles: 0°, +45°, -45° and 90°. These orientations are selected to generate high strength and stiffness under direct loading in the principal (0° and 90°) directions, whilst simultaneously maintaining mechanical properties under shear loading (+45° and -45° directions). Typical composite wing box skin (top and bottom aerodynamic surfaces) laminates, for example, comprise plies orientated in these 0°/+-45o/90° directions in the proportions 50%/40%/10%.

It is generally true that the chord-wise stiffness (Ey) and strength of the skins is of small significance. Much more important is the span-wise stiffness (Ex)/strength and the shear stiffness (Gxy)/strength, since the principal loads in the wing box (see Figure 2) comprise bending and torsion. Atkins has been studying the effects of narrowing the angle of the "angle plies" – those oriented at +45° and -45°. The potential benefit of such a revision is illustrated in Figure 3. Reduction of the 45° angle promises a significant increase in span-wise stiffness and strength, whilst having a negligible impact on shear performance.

The studies by Atkins have concentrated on laminate

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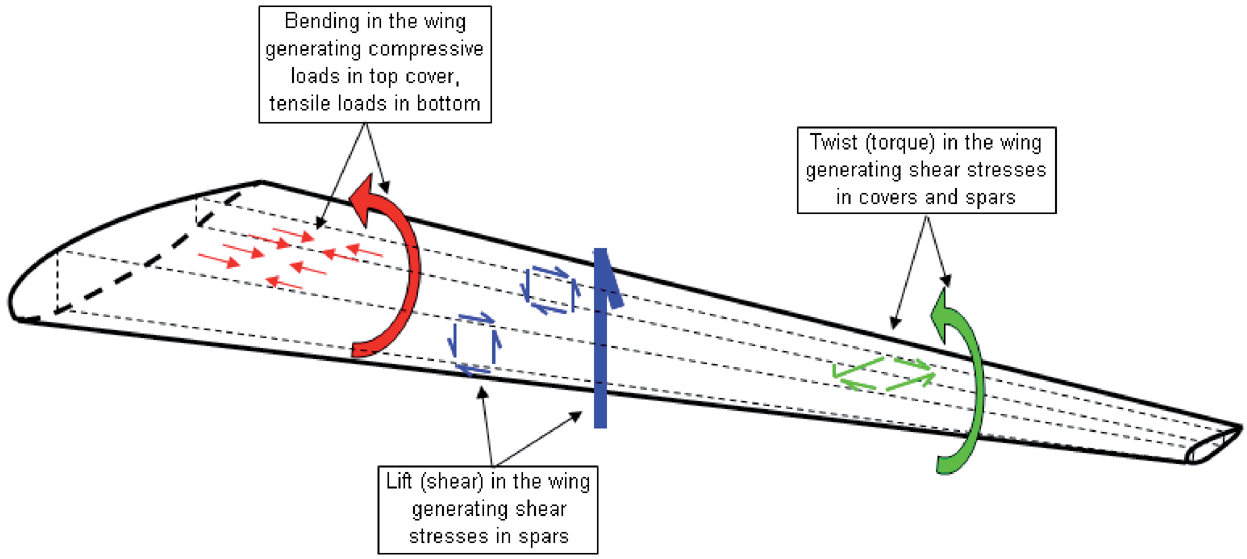

FIGURE 2. LOADING ON A WING BOX

strength under span-wise and shear loading and have looked at angles ranging between 30° and 45°. The studies have been confined to skin laminates and stringers - the stiffeners that resist buckling of the skins. These studies have shown that under predominantly tensile/shear loading there is a potential benefit of 7% in stiffness and strength for an angle of 35°. However, under compressive/shear loading, the benefits are less clear-cut, and strength can reduce at such angles. Thus, the use of these revised angles is likely to be suitable for the lower skin, for which the highest loads are tensile, but less so for the upper skin, which supports principally compressive load.

Similar benefits have been identified for the stringers for which, because they carry only small shear loads, a benefit both on the top and the bottom wing surfaces can be realised.

Further, a fringe benefit has been found to be reduced inter-lamina stresses at the bond between the stringers and the skins, alleviating a chronic concern regarding de-lamination at these susceptible joints.

From these studies we have been able to recommend initiation of a test programme to examine the strength of laminates with the angle of the angle plies ranging between 35° and 40°. These tests will assess damage tolerant residual strength (tensile and compressive strength following an impact) and bolted joint strength. Our recommendations are being taken up by Airbus as part of a wide-ranging test programme to assess the performance of different materials and lay-ups. Should this test programme confirm the suitability of such laminates, a 2-3% reduction in wing box structure weight is feasible.

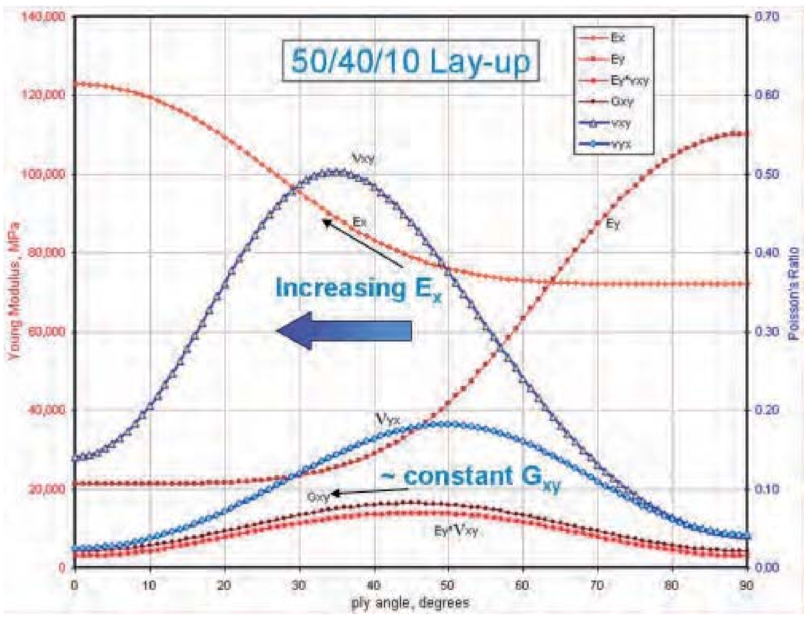

FIGURE 3. THE EFFECT OF REDUCING THE ANGLE OF THE ANGLE PLIES

5. Aero-elastic tailoring

Aircraft wings are flexible, and since their precise shape has a rôle in determining the aerodynamic loads applied to the wing, the control of the stiffness characteristics of the wing structure can be used to influence the externally applied loads, and hence the internal stress distribution. The adjustment of the stiffness characteristics of a structure to influence the magnitude and distribution of external and internal loads is termed "aero-elastic tailoring".

Carbon composite structures are particularly amenable to aero-elastic tailoring because the engineer can choose the orientation of fibres within the major components of the structure, thus strongly influencing the stiffness characteristics. In order to extract the greatest benefit from the use of carbon fibres for a wing structure, the adoption of aero-elastic tailoring should be considered with a view to minimising peak external loads and hence peak internal stresses. The reduction of the internal stresses and strains can be used to reduce structure weight. An extreme example of internal wing structure load relief is illustrated in Figure 4: the X29A forward swept wing demonstrator aircraft needed aero-elastic tailoring to mitigate the "divergent bending moment" that is inherent in a forward swept configuration. For a high aspect ratio (span/chord) wing, such as would be used for the NGCW, the greatest benefit may be accrued by a reduction in the bending moment near the wing root at high gust and manoeuvre loads. This reduction will result in a corresponding relief of span-wise stresses in the top and bottom skins, allowing either a decrease in the local skin thickness (and hence weight), or a decrease in overall wing thickness and sweep angle (again with reduced weight).

NASA Dryden Flight Research Center Photo Collection
http://www.dfrc.nasa.gov/Gallery/Photo/index.html
NASA Photo: EC87–0182–14 Date: July 24, 1987 Photo By: NASA

X–29 in Banked Flight

FIGURE 4. AERO-ELASTIC TAILORING MADE THE X29A FORWARD SWEPT WING STABLE

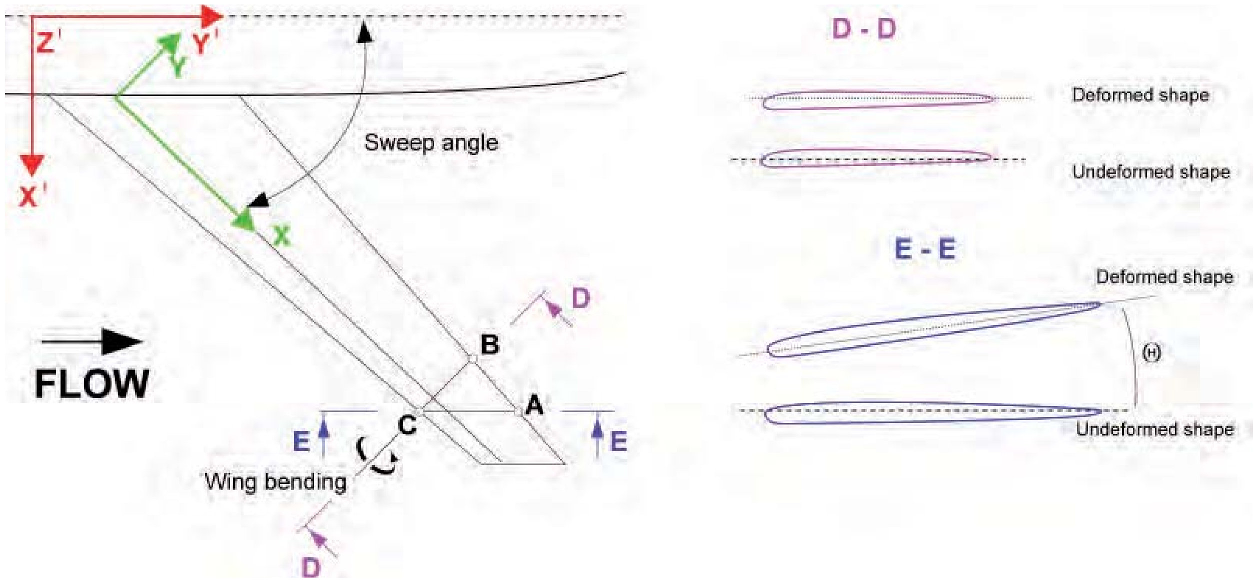

FIGURE 5. WASH-OUT OF LIFT AT THE TIP OF AN AFT SWEPT WING

The reduction of wing bending moment can be achieved by allowing lift to "wash-out" at the tip at high loads. This characteristic is achieved by introducing a coupling between wing bend and wing twist, and is one shared by aft swept wings, as shown in Figure 5. As composite wings are twice as stiff as their metallic counterparts, however, they "need some help" to achieve the same degree of wash-out and hence equivalent load relief. Atkins has been investigating the use of "unbalanced" lay-ups to achieve this goal.

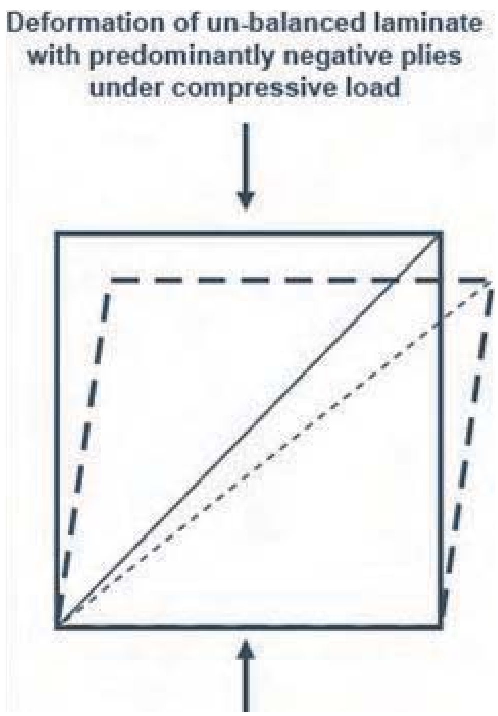

FIGURE 6. RESPONSE OF UNBALANCED LAY-UPS TO LOAD

By this means, there is an imbalance between the number of +45o and -45º plies in the skin lay-ups, leading to an imposed shear strain in the skins when a direct load is applied, as illustrated in Figure 6. Through the use of unbalanced laminates, compression in the top cover under an up-bending load can be arranged to generate shear in the cover, whilst tension in the bottom cover can generate shear in the opposite sense. These shears manifest themselves as twist of the overall box section. Therefore, the use of unbalanced laminates in the covers can be arranged to lead to bend/twist coupling in the wing box and hence the level of tip lift wash-out required to match or surpass that of an equivalent metal wing.

The Atkins work has shown that lay-ups with a moderate degree of imbalance can achieve twist/bend coupling that will result in significant increase in lift wash-out at the tip – equivalent to an additional 5º of wing sweep. Current analysis suggests, however, that the lay-ups should be used judiciously, as plain laminate strength calculations indicate that unbalanced lay-ups can have reduced strength characteristics under certain loadings. As is always the case with laminated composite aircraft structures, residual strength following damage and bolted joint strength drive the integrity of the structure, so Atkins has proposed unbalanced lay-ups as further additions to the Airbus materials test programme.

Although it is outside of the scope of our current remit, it is hoped that future work will allow us to assess the load relief that will be experienced across the wing by the use of these lay-ups. Further, the application of these techniques can be extended to aero/inertial-elastic tailoring – the control of an aerodynamic shape under combined aero and inertia loads. An application for this technology might be aero gas turbine fan blades, which could be designed to resist un-twisting across the running range of the fan.

6. Other studies

The examples that have been described derive from the novel laminates work programme, but a range of other developments are being carried forward both in this programme and in the rapid sizing arena. Typical examples of current studies are:

- Inter-lamina stress calculation to assess composite panel post-buckling and ramp rates used in varying thickness of laminate;
- Optimisation of wing box internal rib spacing and stiffener distribution;
- Rapid estimation of wing box internal loads and deflection;
- Simplified assessment of external wing box loads to support our aero-elastic tailoring studies.

These studies have been chosen to reinforce our knowledge and capability in areas of composite wing box design that represent a particular challenge.

7. Conclusions

Atkins' investment in the MDOW project has allowed us to explore facets of laminated composite design not readily investigated in the course of our normal work diet. The engineers involved in the project have had their eyes opened to the more esoteric possibilities that these advanced materials present. However, there is much to be done before the end of the NGCW programme, and we must maintain in our minds the critical words in that acronym: new technologies must be consistent with the next generation of civil aircraft.

Applications of Virtual Reality in transportation projects

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Abstract

Interactive Virtual Reality (VR) modelling gives opportunities to experience a proposed engineering project before it is constructed. Atkins has over 10 years experience in designing more than 60 VR models for a range of purposes and audiences. This paper describes the application areas Atkins has addressed so far, illustrating them with examples from real projects.

1. Introduction

Virtual reality combines 3D computer modelling with images and software to create a navigable world. Many of the modelling techniques are similar to those used in the creation of highly realistic presentational fly-throughs. These take many hours of processing time to generate frame by frame. But VR sacrifices some of this absolute realism for immediacy of interaction; it generates its animation in real time in response to the actions of the user. The technology is directly comparable to computer games rather than CGI special effects in films.

The game-like association can lead people to assume that VR might be both expensive and gimmicky. It is neither. Continuous improvements by Atkins over the years have repeatedly reduced the time and cost of converting project designs into VR models. The remainder of this paper will look at the practical benefits that flow from this modelling

2. Applications

Explaining proposals to the public

Many lay people gain very little from the plans, cross sections and elevations that technical professions read effortlessly. Even fundamentals like whether a road passes over or under a bridge are missed by a significant number of people. However, VR presents the information in a way that is intuitively understood because it is sufficiently like looking at the real world.

Atkins has created VR models for a large number of public exhibitions for highway projects. Supplementing the usual variety of drawings, text and graphics, a VR model can help to explain specific points of concern to individuals: the views from their houses, the change to their route to work, how environmental provisions would soften the impact and so on.

Clear explanation removes doubt. Hopefully most people would be reassured that their fears were unnecessary; but it might instead serve to focus precisely the points of objection, and perhaps lead to amendments satisfying all parties. Reducing the number of statutory objections shortens the Public Inquiry process, potentially saving considerable time and money.

FIGURE 1. A14 BAR HILL JUNCTION

At the feasibility stage, the VR model may show different route options. Atkins model of the A477 in South Wales allows switching between the existing road and alternative routes bypassing the village of Llanddowror to the north or to the south (refer to Figures 2 to 3).

FIGURE 2. A477 NORTHERN ROUTE

FIGURE 3. A477 SOUTHERN ROUTE

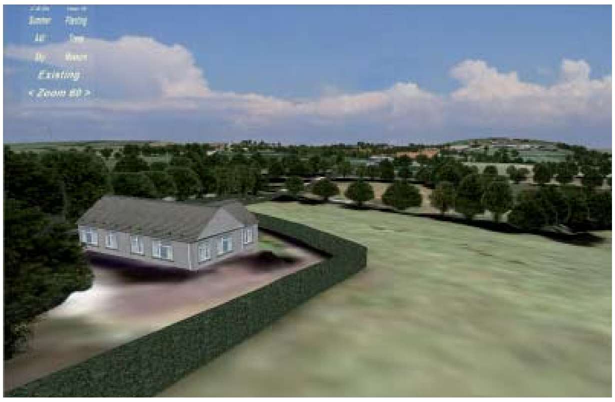

FIGURE 4. A40 PRE CONSTRUCTION

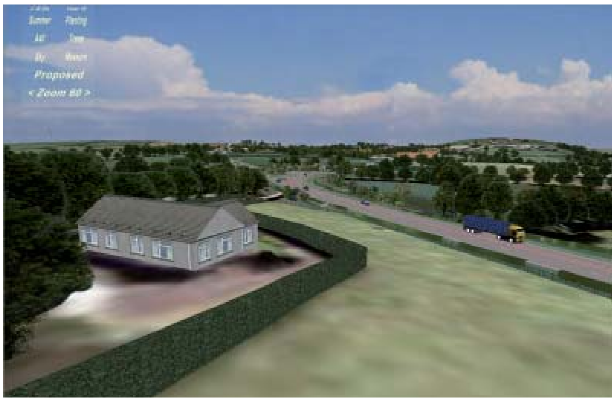

FIGURE 5. A40 AFTER CONSTRUCTION

Including the existing conditions is standard in most of Atkins VR models. People can readily orientate themselves in a recognisable environment. The instant comparison on the same screen, from the same viewpoint, of existing and proposed options is a highly effective way to get the message across. The change that will be experienced can be measured and explained with a clarity that static plans or models cannot give (refer to Figures 4 to 5).

A switch between options is perhaps the simplest example of the software that can be incorporated in a model. More sophisticated features that routinely enhance Atkins models are traffic and signals. A range of road and rail vehicles can drive realistically around the virtual world, using the correct wheelbases and articulation to achieve accurate swept paths from first principles. Locating viewpoints within the vehicles gives drivers' eye views appropriate to those specific vehicles. The signs and signals help to complete these views (refer to Figure 6).

As well as freedom to move around in space, the model can also step through time. Atkins model of the replacement of Burn Closes Bridge in Wallsend explained the whole process to the residents, including how access would be maintained to schools etc. by a temporary footbridge (refer to Figures 7 to 11).

Explaining to stakeholders

Stakeholders readily understand the aspects of a project that relate to their area of expertise. VR models

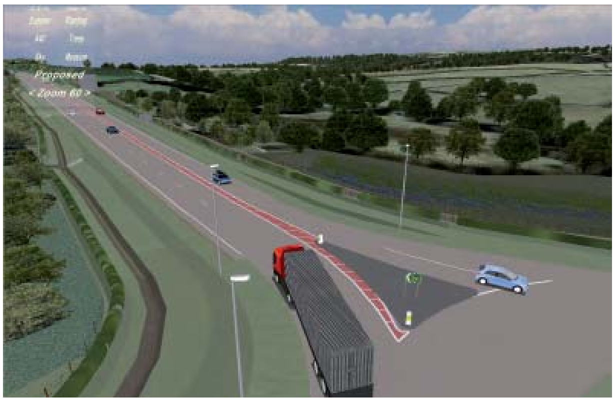

FIGURE 6. VEHICLE ARTICULATION

can include considerable detail about operation and maintenance, emergency services facilities etc. One of the first VR models Atkins produced was of the variable message signing for the A11 Green Man Tunnel, including normal operation, planned maintenance, and emergency closure (refer to Figure 12).

More recently Atkins has designed strengthening works for the Forth Road Bridge. There are several options to replace the bearings on the approach viaducts, but all involve changing the appearance of this listed structure to some extent. A VR model proved a powerful way to explain these options to the client, FETA, and to Historic Scotland (refer to Figures 13 to 14).

FIGURE 7. ORIGINAL BURN CLOSES BRIDGE

FIGURE 8. BURN CLOSES DEMOLITION

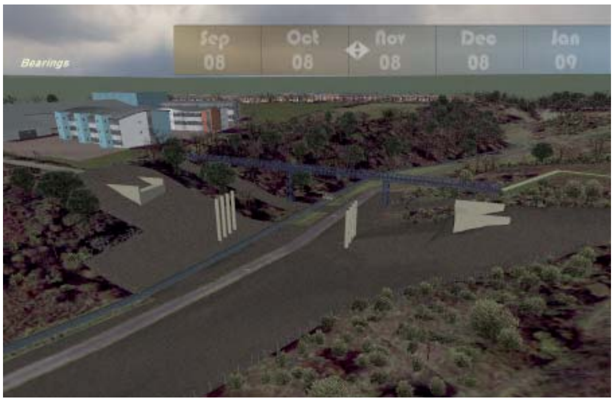

FIGURE 9. BURN CLOSES : ABUTMENTS & PIERS

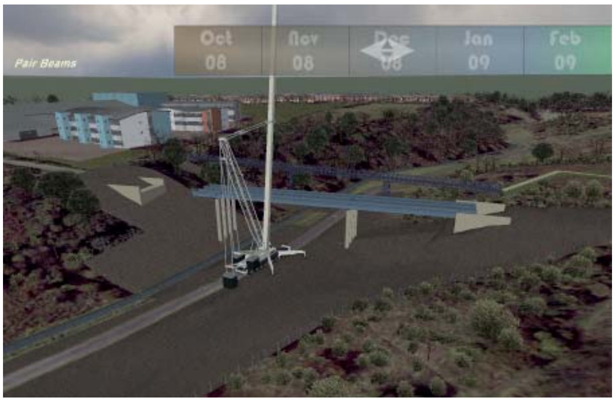

FIGURE 10. BURN CLOSES BEAM INSTALLATION

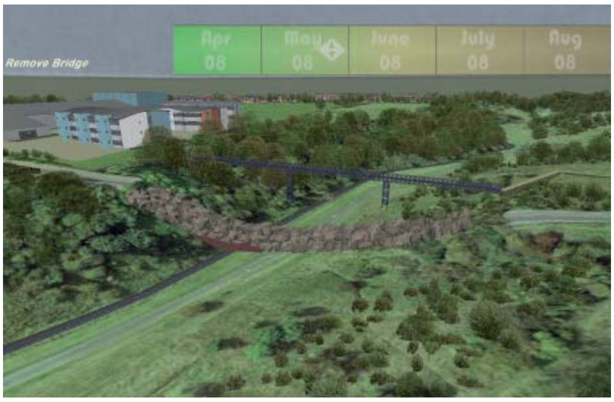

FIGURE 11. BURN CLOSES COMPLETE

FIGURE 12. GREEN MAN TUNNEL CLOSED

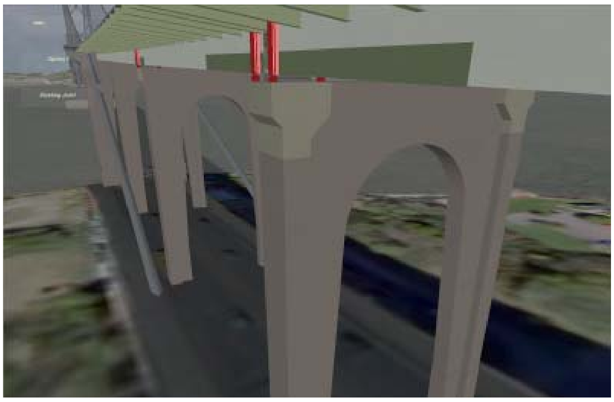

FIGURE 13. FORTH ROAD BRIDGE
BEARING REPLACEMENT OPTION 4

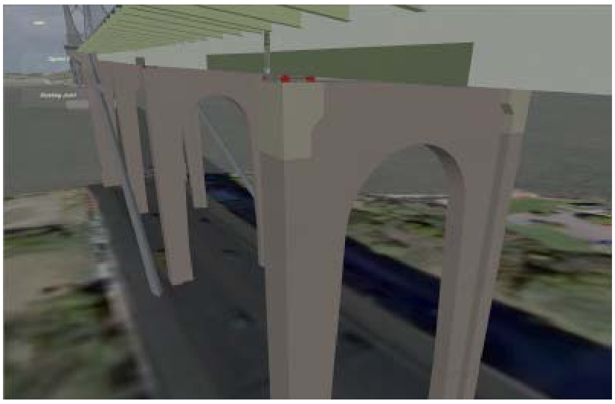

FIGURE 14. FORTH ROAD BRIDGE
BEARING REPLACEMENT OPTION 5

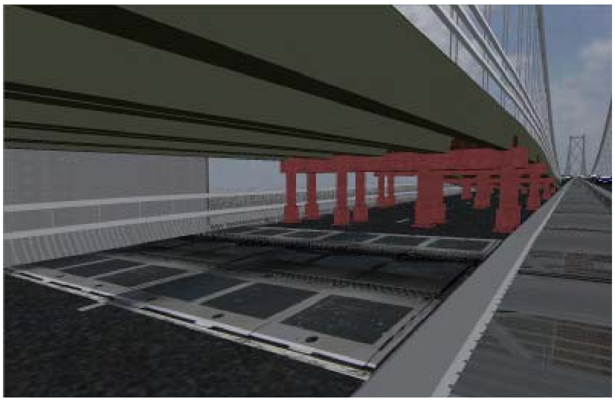

FIGURE 15. FORTH EXPANSION JOINTS WITH TEMPORARY FLYOVER

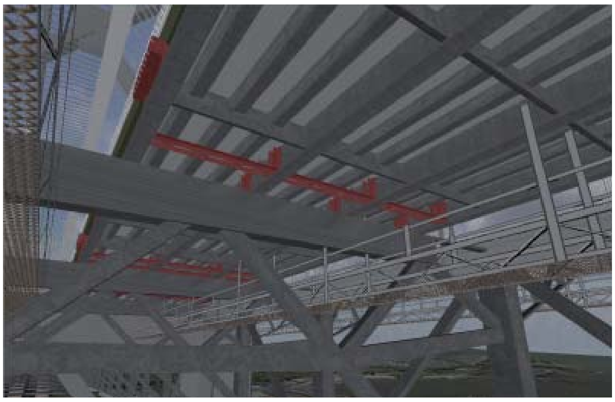

FIGURE 16. STRENGTHENING BENEATH THE DECK

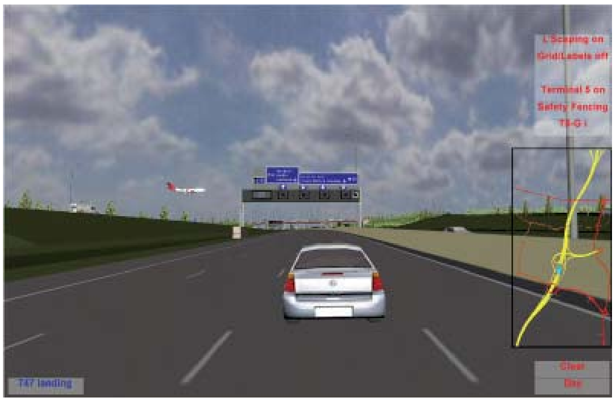

FIGURE 17. M25 HEATHROW T5 SPUR

FIGURE 18. M25 DETAILED MODELLING

Further proposed works to replace the expansion joints at the towers involve careful traffic management and temporary flyovers to keep traffic running while work is in progress. The VR model of this aspect was used to brief bidders as well as for public information. Images from it appeared on the BBC News Scotland web site (refer to Figures 15 to 16).

3. Evaluation of the design by the client

The opportunity to inspect a design before it has been built means that the client's teams may evaluate how it satisfies their requirements. Atkins was commissioned to model the M25 Widening near Heathrow Airport twice. First, before the contract was let, the joint clients Highways Agency and BAA wanted to assess the potential distraction to drivers on the T5 Spur from planes taking off and landing on the northern runway. Two years later the VR model was updated to reflect the D&B contractor's final design. Every sign, marking, gantry, lighting column, and safety fence was positioned accurately. This gave the clients a tool to assess the possible actions of drivers trying to find their way to the correct place in the airport. It also provided a view of the entry to one of Europe's busiest motorways as faced by a jet-lagged driver in an unfamiliar right-hand-drive car (refer to Figures 17 to 18).

"Getting Northampton to Work" was the title of an ambitious project to improve traffic flows on Northampton's inner ring road. Atkins' VR model switched between the existing and proposed states. However, seeing the stark impact of a roundabout with trees replaced by a multi-lane light-controlled junction was enough to convince the councillors to cancel the project. Saving the wrong scheme from being built made this VR model very cost-effective (refer to Figures 19 to 20).

Wolverhampton Interchange Project was an initiative by Wolverhampton City Council and Centro, the West Midlands Public Transport Authority. It set out to offer "seamless journeys" to passengers transferring between mainline rail, buses and coaches, the Metro trams, and taxis and private cars. It aimed to provide best practice travel information of various kinds throughout the extended transport interchange between the rail and bus stations. An "effectiveness test" was required as a condition of the European funding through the LiRa2 Light Rail Cities project: Atkins VR model fulfilled this very well. Throughout this model all the variable message signing for the many modes of transport is driven by a single consistent, correct set of timetable data: even the bus numbers match the stand letters. Some travel scenarios were posed by the client, and were demonstrated using the model (refer to Figures 21 to 22).

Getting agreement to the layout of station control rooms was a significant bottleneck in the remodelling of

FIGURE 19. NORTHAMPTON EXISTING

FIGURE 20. NORTHAMPTON PROPOSED

FIGURE 21. WOLVERHAMPTON OVERVIEW

FIGURE 22. WOLVERHAMPTON DEPARTURES

FIGURE 23. LONDON UNDERGROUND WHITE CITY STATION

London Underground stations. Atkins most interactive VR application to date embedded the VR viewer in a Windows application. It was intended to be used by Human Factors specialists working with the control room staff to achieve an ergonomic working environment quickly. The application allowed pieces of equipment to be added, deleted, moved and rotated directly on the walls and work surfaces in the model. The ownership hierarchy was also adjusted, for example if a telephone was placed on a desk, it would thereafter move with the desk unless reassigned elsewhere. In the control rooms there are minimum distances such as between the operators' chairs and the wall or obstacle behind. Direct measurement tools were therefore developed for this project (and feature in most subsequent ones). They give accurate 3D distances between snap codes: vertex, edge, midpoint, normal, vertical, and eyepoint. At any time during the session, the current state could be saved as a timestamped version with a matching inventory of equipment with its locations. These would then be used to automatically update the CAD drawings (refer to Figure 23).

4. Virtual prototyping

VRML's extensible design makes it ideally suited to test new concepts. Atkins Intelligent Transport Systems came up with an idea to increase safety in traffic management schemes by using synchronised flashing lights on traffic cones. These lights would pulse along the route to indicate both the speed and the 2-second separation distance that vehicles should take. Travelling too fast would make the flashes loom towards the driver, whereas travelling too slowly would make the flashes overtake the vehicle. The separations would be effected by driving at a comfortable distance from the flashes. A real prototype of the concept would have been very expensive, but a virtual one was a few days work. It showed that the spacing of the lights would have to be half that of the original design in order for the flashing to convey a sense of forward movement. With this change the design worked very well (refer to Figures 24 to 25).

FIGURE 24. INTELLIGENT TRANSPORT SYSTEMS PROTOTYPE

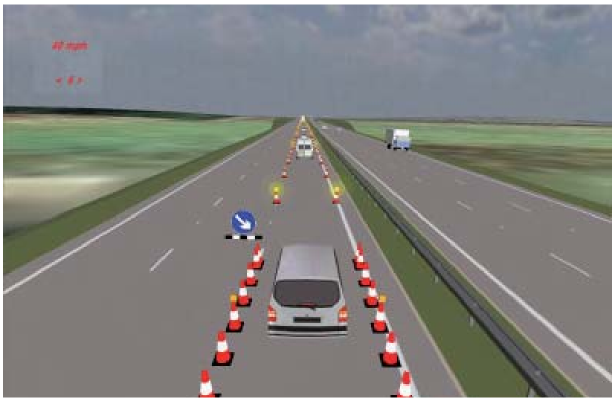

FIGURE 25. INTELLIGENT TRANSPORT SYSTEMS PROTOTYPE

5. Meeting CDM obligations

Traditionally the designer's suggested safe sequence of operations would be explained on a drawing or sequence of drawings. However, where the sequence is highly complex, there is a significant risk of a mistake in interpretation with potentially serious consequences.

For the strengthening works on the seven box girders of the M60 Irwell Bridge near Manchester there were over 500 steps: jacking, cutting, welding, bolting etc. Atkins chose to present the safe sequence through a Windows program that combined a spreadsheet with two VR windows for overview and detail, and a text description. The program took the operations defined in the spreadsheet, sorted them into time order, and presented them graphically. Each component was given a unique label reflecting its girder number, end identifier and face of diaphragm. The detail window jumped to a close-up of the component, while the overview window showed the locations of similar operations that could take place in the same time step (refer to Figure 26).

As well as presenting the sequence unambiguously, this VR application acted as a fast and safe means of project induction to the design team and on site. The contractor was very positive about the tool, using it also for logistics planning of component delivery to the restricted site.

6. Design improvement

Bringing together the outputs of many design teams in a VR model gives an early opportunity to correct clashes and errors that might not otherwise be discovered until the construction phase. By then options are restricted; costs may be high and delays serious.

The visibility of signals was a significant issue in the remodelling of Rugby Station on the West Coast Main Line. Traditionally signals are initially positioned from experience, and then checked on site using physical targets. This

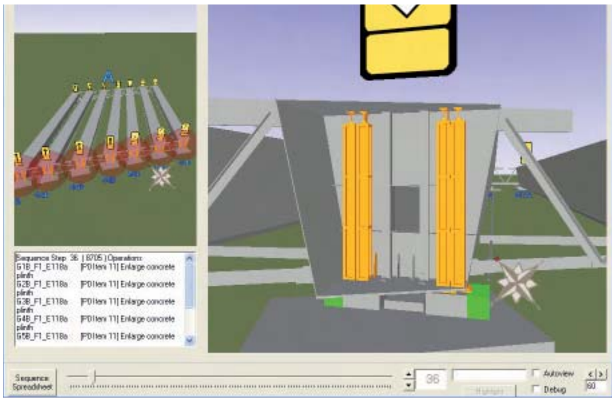

FIGURE 26. IRWELL STRENGTHENING CONSTRUCTION SEQUENCE

is impossible where the lines do not yet exist and their locations are among other existing live railway lines and infrastructure. The VR model allows the driver's view to be assessed on any route at the fastest theoretical speed profile for specific types of rolling stock. Distance and time from the next signal are continuously updated on the screen, with slow motion options. The precise timings and durations of obscuration by OLE equipment or any other infrastructure can be easily assessed. (refer to Figure 27).

A design consideration for the proposed new bridge at Walton on Thames was visibility to swans flying along the river. It was necessary to assess the contrast of the bridge against the sky in different weather conditions. The appearance also has to be attractive to humans. Atkins VR model therefore includes the facility to drag and drop materials and paint colours onto the various components of the bridge. Representatives of design disciplines and other stakeholders can thus work together around the VR model to reach agreement more quickly than by any other means (refer to Figures 28 to 30).

Road Safety Audits (RSA) are carried out on all new road schemes. The Stage 1 Preliminary Design and Stage 2 Detailed Design audits are carried out before construction. Both can be far more effective by auditing a realistic 3D VR world that includes the different disciplines' work than by mentally picturing the contents of their separate 2D

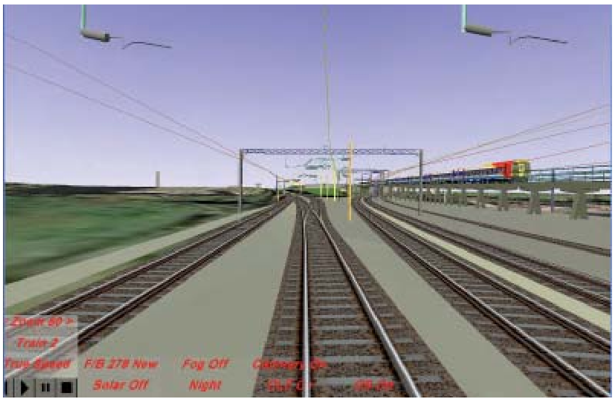

FIGURE 27. RUGBY STATION REMODELLING

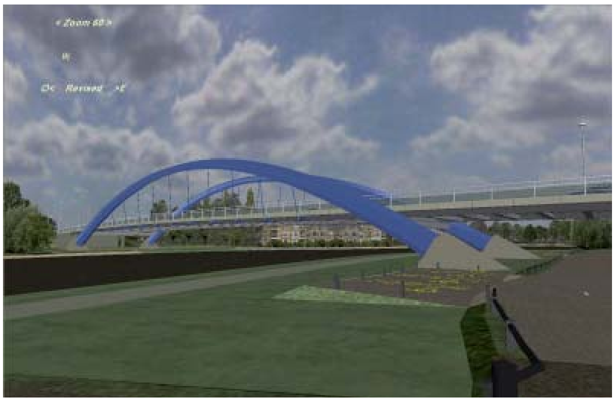

FIGURE 28. WALTON BRIDGE

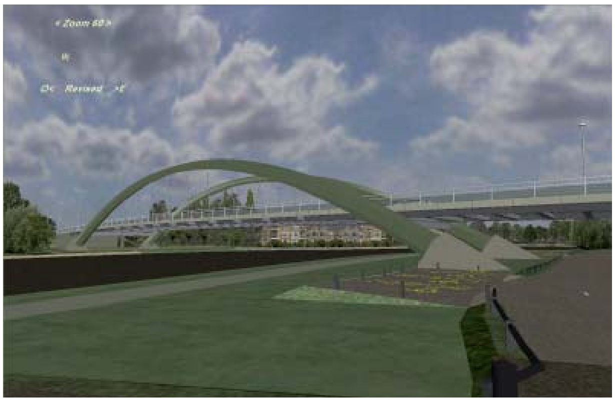

FIGURE 29. WALTON BRIDGE

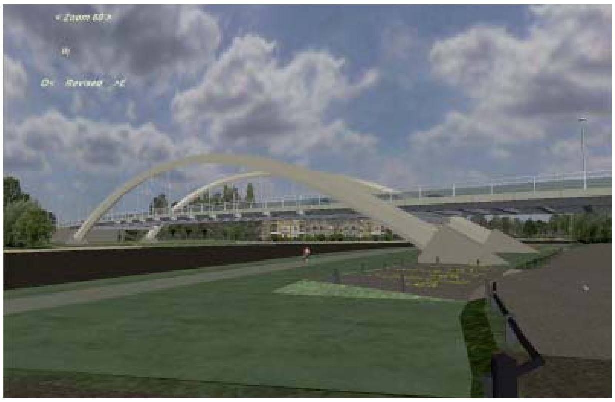

FIGURE 30. WALTON BRIDGE

FIGURE 31. SLOUGH SAFER ROUTES TO SCHOOL

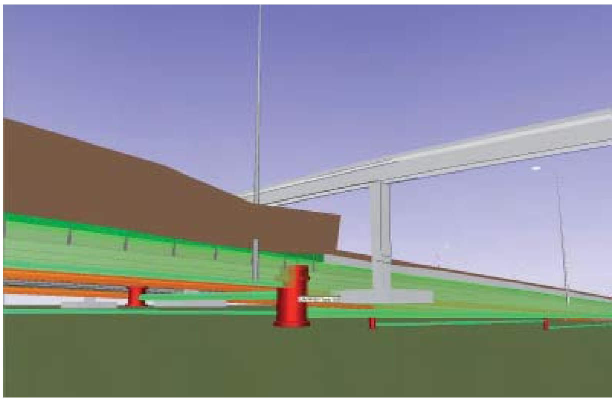

FIGURE 32. M25 DBFO UNDERGROUND INFRASTRUCTURE

drawings layered together. The ability to assess driver/vehicle interaction with the design at the design stage rather than during or after construction in the Stage 3 Audit avoids alterations to signs, markings and roadside features after they have been built. For complex junctions this is particularly powerful as both design team and safety auditors can engage with the design while there is still scope for improvement (refer to Figure 31).

The largest example of this approach to date is the M25 DBFO, in which Atkins is part of the Connect Plus consortium, currently Preferred Bidder for the project. The widening is to be accommodated within the existing land take, meaning that many services are jostling for severely restricted verge width. The immediate demand for the 3D model is thus to detect and avoid clashes. In-house software has been developed to create VR models automatically for drainage networks, communications ducts and chambers, safety fences, and lighting. Bridge foundations, gantries and retaining walls will also be modelled to give a complete picture of the infrastructure under the ground as well as above (refer to Figure 32).

7. Virtual construction

A further application of VR is in virtual construction. The very process of building the VR model highlights issues with the design. In the visual world the detection of missing, overlapping and incorrect components is easily carried out whereas in the 2D drawing world details that may span several drawings and schedules are not so easily checked. Road alignments that look odd because of the interaction of horizontal and vertical alignments are easily detected in a driver's eye "drive through", but may be missed in traditional plan and long sections.

8. Conclusion

The VR technology currently available in Atkins is under constant development. Being independent of the development schedules or markets of an external vendor enables Atkins to deliver improvements and new features quickly in response to clients' needs.

VR has proven benefits throughout the design process from explaining scheme proposals at preliminary design stage, to assessing the safety of detail designs and through to exploring the construction sequence with the Contractor. It can enhance the quality of design and save time and money for clients, contractors and designers.

Appendix

Equipment

Almost any modern PC has the processing power to display a convincing model of a small highway project such as a small bypass or improvement scheme. Navigation may be by standard mouse with a few keyboard commands.

Other input devices such as joysticks can add better control to the model, although increasing the need for training and familiarisation. Distance perception is greatly enhanced by stereo display, for which there are many options depending on budget and the size of the intended audience at any one time.

VR animation is created in real time and is convincing if the frame rate is kept above about 15 frames per second. Below this value the user is increasingly irritated by jerkiness in the display. For larger models with more detailed content the most critical component is the graphics system. As an example of the speed of change, over the last 6 years the video memory of top-of-the-range graphic cards has increased 16 fold (in line with Moore's Law), with similar leaps in processor speed. But the cost of today's hardware is less than half of that of 6 years ago.

Some scientists and technologists are already starting to use games consoles for serious work. These are computers that are highly tuned for graphics performance. Their sensors and handsets are among the most sophisticated input devices available within a reasonable budget. It is very likely that greater acceptance of VR as an engineering tool will also bring this type of equipment into use in the professional engineering environment.

Software

Atkins made a strategic decision to adopt non-proprietary VR formats to avoid being tied to a product that might be withdrawn from the market. VRML97 is an ISO-Standard language for this task. It was designed as a web application with small files for fast download. X3D is the more modern XML version of the same concepts; XML is increasingly adopted for all types of data handling. Both VRML and X3D are plain text formats, so there is no risk of data loss through data files ceasing to be readable. Both are also extensible, meaning that new types of VR objects can be defined with potentially complex event-driven properties and interactions. Embedded software programs can allow each instance of an object to respond in an individual way to its inputs.

Collada, a yet more recent XML format for VR "assets", was defined through collaboration of several hardware and software vendors. It is "owned" by an independent foundation and its use is growing.

There are several VRML / X3D viewing programs on the market. The highest performance viewer Atkins has found for large models is BS-Contact from Bitmanagement GmbH, which also supports Collada.

Atkins uses in-house software tools to generate the VR content directly from a variety of data sources, including commercial road and rail design packages and CAD.

Printed on 9Lives Offset, part of the family of 'born again' papers, manufactured with National Association of Paper Merchants Certification (NAPM) using approved 100% recycled fibre pulp, 9lives Offset offers the ultimate in environmental management. The manufacturing mill and printer have also been accredited with the internationally recognised standard ISO 14001.