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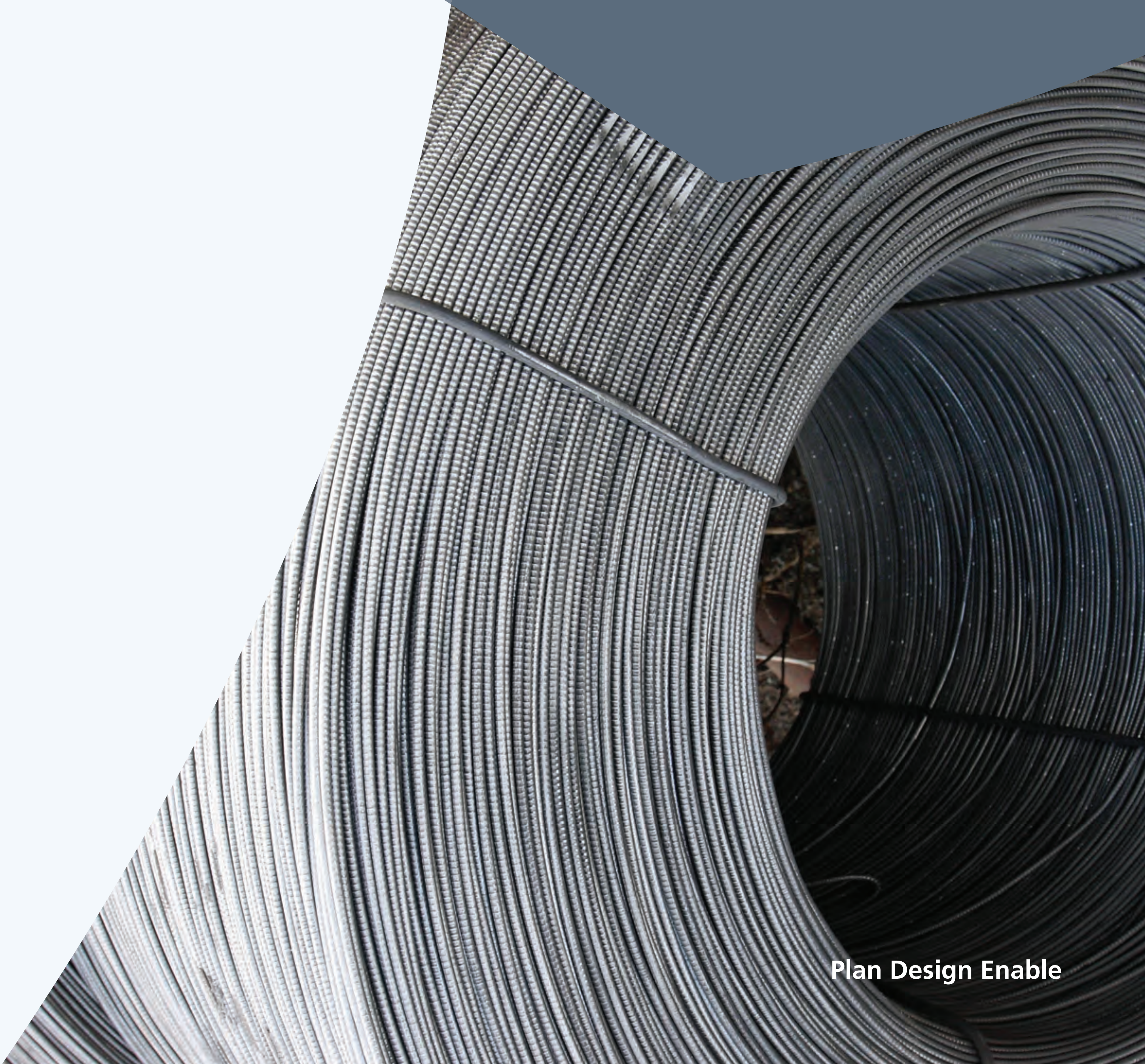
ATKINS

Technical Journal

Papers 120 - 134



Plan Design Enable

Welcome to the eighth edition of the Atkins Technical Journal which features papers covering design, strategic advice, research and thought leadership across a broad range of disciplines. All the papers showcase the innovation we bring, whether through improving composite aerofoil design, producing a concept for a 3D interchange viewer and walk planner, or developing a mobile system to manage highway defects which has enabled a 52% cost and efficiency saving. Coupled with this, however, remains a strong focus on technical governance and data security; systems engineering approaches feature strongly throughout.

The success behind much of the work described can be attributed foremost to the high calibre of our teams, but also to their adherence to sound principles for technical delivery. It is evident that the teams featured had an excellent grasp and understanding of our client's requirements at the start of the project and then delivered on time and to budget through effective project controls, checking and review. Many of the key lessons learnt have been captured for re-use through the papers in this Journal. It is no coincidence that precisely these behaviours are being reinforced through the recent introduction of Atkins' global Design Principles.

I hope you enjoy the selection of technical papers included in this edition. This eighth Journal, and all previous editions, are available on our external website. We are introducing an email subscription alert service for future editions. To find out more, please visit:

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Atkins

Foreword

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Cyber security and critical national infrastructure

Abstract

Cyber security is an all-embracing term, meaning different things to different people, but fundamentally it is the defences which shield computer systems from electronic attack. These may range from relatively small-scale email scams through to state-sponsored disruption of computer-based systems that run critical national infrastructure, such as the electricity grid, water or transport networks.

Governments and organisations that manage national infrastructure, therefore, face a challenge to ensure their systems are adequately protected. Moreover, the steps they put in place must protect from a range of cyber threats that are constantly evolving.

This paper explores the challenges faced by organisations and the standards and approaches which can be applied to help protect their systems, in particular Industrial control systems (ICS) and Superviisory control and data acquisition systems (SCADA) systems.

Why cyber security is important for national infrastructure

ICS and SCADA are utilised throughout national infrastructure in water, electricity, gas, petroleum, pipelines and transport. They are ubiquitous in manufacturing and even drive diverse things such as theme park rides, baggage systems and ski lifts.

As it has developed, cyberspace has enabled the automation and optimisation of the infrastructure that supports many businesses; for example, the SCADA systems that automatically control and regulate industrial processes such as manufacturing, water distribution, refining and power generation. – UK Cyber Security Strategy, Nov 2011

ICS and SCADA are the building blocks of automated systems where control or monitoring of a process is required; many also have varying degrees of safety-related functionality, from protecting operators, users or customers to members of the public. The potential disruption from a cyber event could be significant and not just in terms of loss of revenue, reputation and damage to affected brands. For example, 80% of the UK population rely on five supermarket retailers who hold only four days' worth of stock in their supply chain, so a cyber event could have a far reaching impact. (Defra 2006)[1].

The scale of the challenge

Two recent publications in the UK have underlined the importance of protecting critical national infrastructure and the scale of the challenge.

The UK Parliamentary Office for Science and Technology (POST), a body that provides independent analysis of policy issues with a science and technology basis, published a briefing entitled 'Cyber Security in the UK'[2]. The briefing highlighted recent events in cyber security and discussed the potential for large-scale attacks on national infrastructure, related emerging issues and the implementation of cyber security. Topics covered included responsibility for UK cyber security, types of attacks and an emphasis upon industrial control systems and the need to improve resilience, security and knowledge in both industry and Government. The POSTnote highlighted that 50% of respondents in a recent survey of security specialists across the industry stated there was a case for improving their cyber defences.

This was followed by the UK Government's Cyber Security Strategy[3], which outlines a programme of Government activity to work closely with companies responsible for critical national infrastructure systems. Moreover, it announces the Government's intention to work with a wider range of companies than those currently associated with national infrastructure; anywhere where the threat to revenues and intellectual property is capable of causing significant economic damage is now firmly on the Government's radar.

Nearly two-thirds of critical infrastructure companies report regularly finding malware designed to sabotage their systems. McAfee, Critical infrastructure protection report, March 2011.

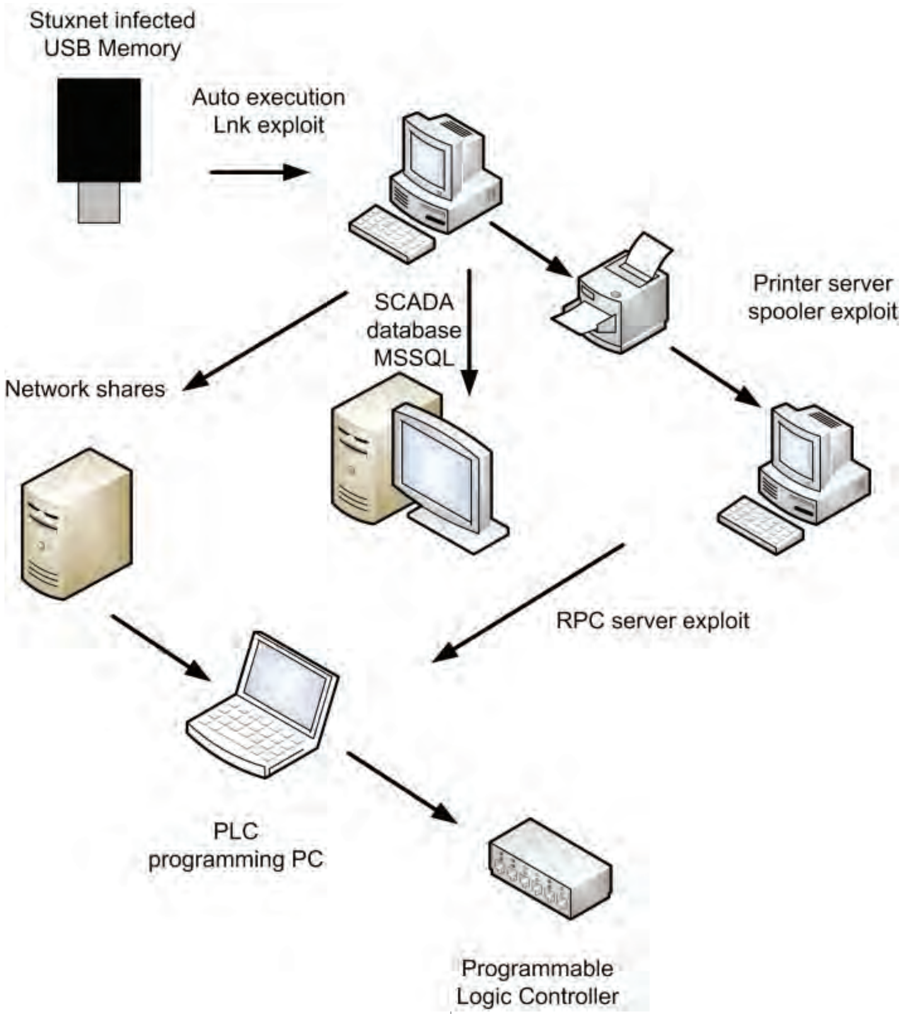

Figure 1. Stuxnet infection routes

Malware that targets control systems

In the same period as these publications, the antivirus firm Symantec released details of a new Trojan called 'Duqu', originally thought to be aimed at industrial control equipment vendors. Duqu gives an indication of the type and complexity of cyber threats that face organisations which manage critical national infrastructure.

Symantec has hailed Duqu as the precursor to the next 'Stuxnet', a previous worm that targeted physical infrastructure (and is believed to have been designed to target Iranian nuclear systems). Duqu functionality shows it is the foremost intelligence mechanism for gathering information on industrial control systems and discovering vulnerabilities for subsequent analysis and exploit development. It is not destructive, unlike Stuxnet, instead it steals information, including intercepting key strokes and taking screen shots and hiding the data in jpeg type images (**Figure 2**). The stolen data is then encrypted and transmitted via several compromised servers to make tracking more difficult. After 36 days the Remote Access Trojan removes itself, providing a high degree of stealth and making identification of affected systems difficult. In at least one instance Duqu installed, via a previously unknown kernel vulnerability exploit targeted via Microsoft Word documents.

Duqu exploits a flaw in the way that Microsoft Windows parses True Type fonts and when successful, it provides a tunnel enabling nefarious running of arbitrary code with full system privileges. This might include installing application, viewing, changing, deleting data and creating new administrator accounts. Duqu operates on virtually all supported versions of the Windows operating system.

Microsoft subsequently offered a temporary workaround and later released an official operating system patch. Research by Microsoft, in preparation of the patch, also highlighted the potential underlying vulnerability to be exploited by browser based attacks, referring to Internet Explorer, but this could also extend to other browsers.

Figure 2. Data is encrypted in a JPEG file of a Hubble image of NGC6745

Stuxnet: An unusually complex threat

Stuxnet has been described by Symantec as one of the most complex threats it has analysed, including:

- four zero day exploits (those exploits that are unknown, undisclosed to the software vendor, or for which no security fix is available – a rarity for any virus that would be considered wasteful by most hackers);
- Windows rootkit – software that enables privileged access to a computer whilst hiding its presence;
- first ever 'PLC rootkit' – infecting PLC programs and remaining undetectable;
- antivirus evasion;
- two stolen Taiwanese digital signatures;
- complex process injection and hooking code (to prevent programmers seeing the infected code);
- network infection routines;
- privilege escalation;
- peer-to-peer updates;
- remote command and control.

Box 1. An outline of Stuxnet characteristics

Duqu was almost certainly written in Object Oriented C and compiled in Microsoft Visual Studio 2008, according to research undertaken by Kaspersky Lab. Why would this be significant? It indicates professional programming techniques used by 'older', more experienced programmers employed on software projects. Whoever they are, they remain active, in March 2012 a new element of Duqu was discovered. The new Duqu driver was designed to avoid detection by antivirus tools developed by the very same group of researchers in Hungary, who originally discovered the Duqu Trojan.

Early versions of Stuxnet had similar functionality to Duqu (**Figure 1**); fingerprinting configurations of industrial control systems which were then attacked in a very precise manner, whilst infected non-matching configurations remained physically unaffected. The similarity also extends to the use of identical source code, indicating that the perpetrators are likely to be the same skilled and highly resourced group responsible for Stuxnet. Both malwares provide blueprints for future attacks, although the high level of sophistication limits the number of entities with the resources available to launch such attacks.

Ever wondered how malware is named? Some files created by the Duqu Trojan begin with the letters DQ. This is a variant of the Stuxnet worm which is believed to have been targeted at Iran's nuclear programme with the aim of disrupting processing by making changes to industrial control systems. This was a defining moment; Stuxnet was the first virus to target physical infrastructure, as opposed to abstract IT systems. Duqu is both very similar and yet different, instead of the payload that targets control systems, Duqu's payload contains the means to reconnoitre networks and steal information.

Protecting against cyber security threats

Against this backdrop, how do organisations protect critical national infrastructure from cyber threat?

Organisations generally manage information risk using Information Assurance (IA) processes based on ISO/IEC 27001 and ISO/IEC 27002 standards that were originally developed in the UK.

These series standards are a risk-based management system that specifies the overarching structural requirements for information management frameworks. As such, they are flexible depending on the requirements of the specific organisation in question and do not require specific security measures to be implemented.

Rather than focusing on specific attack examples when designing security measures, it is considered best practice to use an holistic approach employing a combination of solutions to address a wide range of possible vulnerabilities. – POSTnote, September 2011

For more specific technical guidance on ICS and SCADA security, organisations can consider a number of sources. In the UK, the Centre for the Protection of National Infrastructure (CPNI) is the Government authority that provides security advice to the national infrastructure. Specific SCADA advice is offered by the CPNI in a series of process control and SCADA security good practice guidelines, which have three guiding principles at their heart:

1. Protect, detect and respond – It is important to be able to detect possible attacks and respond in an appropriate manner in order to minimise the impacts.
2. Defence in depth – No single security measure itself is foolproof as vulnerabilities and weaknesses could be identified at any point in time. In order to reduce these risks, implementing multiple protection measures in series avoids single points of failure.
3. Technical, procedural and managerial protection measures – Technology is insufficient on its own to provide robust protection.

Additional specific technical standards are in preparation and will form the basis for European and international approaches to industrial cyber security. Work currently taking place in IEC (preparing the IEC 62443 standard) is incorporating a management framework that embodies the approach of the ISO 27000 series familiar to the IT industry, addressing the gap in the widely adopted IT management standard.

Further technical and management standards that will form a framework for UK implementation of industrial cyber security, include the work done by the US International Society of Automation (ISA). The ISA has published the ISA-99 series of standards that deals with Industrial Automation and Control Systems Security and collaboration between ISA and IEC is developing a similar series of technical standards under IEC 62443 which will incorporate a management framework that embodies the approach of the ISO/IEC 27000 series.

Protecting against cyber attacks requires action at many levels. Implementing technological solutions is vital but the skills, behaviour and attitudes of personnel are equally crucial – POSTnote, September 2011

Defence in depth coupled with agility

The pace of technological change is relentless. Keeping pace will require people who have a deep understanding of cyberspace and how it is developing. – UK Cyber Security Strategy, Nov 2011

In the majority of cases compliance with ISO standards for information assurance is voluntary and this is widely seen as beneficial. The absence of regulation in industrial cyber security ensures that developments in this fast-moving area are not stifled. However this also means that cyber security for critical national infrastructure is far from a simple checklist exercise.

Organisations wishing to protect their ICS and SCADA must keep abreast of technical developments and put in place systems that balance adequate protection with flexibility. The key is to adopt an holistic approach to implementing a range of measures that provide defence in depth, whilst recognising that cyber security is a continuous process and that contingency planning for inevitable cyber events is crucial.

How is Atkins advancing cyber security?

Atkins are leading innovations in cyber security, an example is the development of new sector specific standards for securing industrial control systems and SCADA. The goal is to improve systems resilience by education and raising awareness of the need to protect systems where it is often difficult to articulate the security requirement and therefore challenging to obtain investment against other more tangible business priorities. Since the approaches to ICS security differ from information assurance, with an emphasis upon different business goals, the new standards provide a basis for applying proven methodologies and techniques to this specific area. Ongoing work is addressing the gaps the Information Assurance Framework (ISO 27000 series) and will provide a scheme for common programme management that incorporate appropriate measures for SCADA and industrial control systems.

Potential industrial control systems incidents [4]

- blocked or delayed flow of information through ICS networks, which could disrupt ICS operation;
- unauthorised changes to instructions, commands or alarm thresholds, which could damage, disable or shut down equipment, create environmental impacts, and/or endanger human life;
- inaccurate information sent to system operators, either to disguise unauthorised changes or to cause the operators to initiate inappropriate actions, which could have various negative effects;
- ICS software or configuration settings modified or ICS software infected with malware, which could have various negative effects:
- interference with the operation of safety systems, which could endanger life.

Box 2. Potential impacts of ICS incidents

References

1. Defra Groceries Report 2006 http://archive.defra.gov.uk/evidence/economics/foodfarm/reports/documents/Groceries%20paper%20May%202006.pdf
2. POSTnote 389, September 2011.
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Modern technique for leakage detection at dams

Abstract

This paper describes a technique called Controlled Source Audio Frequency Domain Magnetics to track, map and monitor leakage through dams.

The problem most dam engineers face is that leakage can often be traced at the toe of a dam or downstream of it, in some cases upstream within the reservoir basin, but it is usually very difficult to track the leakage through or under the dam.

The 'Willowstick' technique, described in this paper, allows individual leakage patterns to be mapped through or under dams.

Introduction

Many dams leak and unless it increases in quantity or starts to take materials with the flow, it is not necessarily a problem. However, if some sort of erosion process takes place and the flow increases, then failure could occur. Remedial works would normally involve comparatively expensive techniques of grouting or slurry trench valley etc., but if the extent and location of leakages can be accurately predicted the amount of remedial works and so costs can be limited.

This paper describes the technique and how it was used to locate leakage and the proposed remedial works at a 200m high rockfill dam in Sri Lanka.

The Willowstick Technique

Controlled Source – Audio Frequency Domain Magnetics (CS-AFDM), nicknamed Willowstick, is a geophysical technique that uses a low-voltage, low-current audio frequency electrical signal to energise groundwater or seepage flows in the areas of interest. The Willowstick method works by measuring the signature magnetic field response of a controlled, alternating electric current (AC) flowing through a specifically targeted subsurface study area – ie the dam and its foundation.

The Willowstick magnetic field is created by a large electric circuit consisting of three parts: (1) the antenna wire connecting two or more electrodes; (2) the electrodes or points of coupling with the earth and (3) the targeted subsurface study area itself, which is located between and/or around the strategically placed electrodes. The diversity of site conditions in dams often necessitates wide variations of electrode and antenna configurations and interpretive parameters.

For a leaking dam, electrodes are placed upstream and downstream of the embankment or structure. The upstream electrode is placed in the reservoir water at sufficient distance from the dam to allow electric current to spread out in the reservoir before reaching the face of the structure. The downstream electrode is placed in strategic locations (seepages, observation wells or other downstream locations) to facilitate contact with seepage flowing through the dam. The electrical current follows preferential pathways by concentrating in zones within the saturated subsurface that offer the least resistance through, beneath, and/or around the dam's structure. As the electrical current takes various preferential flow paths through, beneath, and/or around the dam, it generates a magnetic field characteristic of the electrical current. This unique magnetic field is identified and surveyed at the ground's surface in a grid pattern using sensitive magnetic sensors.

The horizontal and vertical magnetic field magnitudes are measured at each grid measurement station on the surface of the ground to define the electrical current's subsurface distribution and flow patterns. In nearly all cases, the paths of least resistance for electrical current to follow infer zones of higher porosity within the saturated subsurface. The locations (coordinates) of measurement stations are obtained using a Global Positioning System (GPS) unit and are recorded in a data logger along with the magnetic field data. The measured magnetic data are then processed, contoured, modelled and interpreted in conjunction with existing hydrogeologic information to enhance the characterisation of groundwater beneath the area of investigation.

The overall approach to the fieldwork involves energising the groundwater of interest with an AC electrical current with a specific signature frequency (380 or 400 hertz) between the paired electrodes. As the electrical current follows preferential flow paths in the study area between the electrodes, it generates a recognisable magnetic field that is measured by sensitive coils. Magnetic field measurements are generally taken along lines ranging from 5 to 15m apart with stations on each line spaced at 5 to 15m intervals. These distances vary from one project to another depending upon resolution requirements and other site conditions. The grid pattern proposed for any particular investigation is designed to provide sufficient detail and resolution to adequately delineate the groundwater of interest, while at the same time optimising funds available for the investigation.

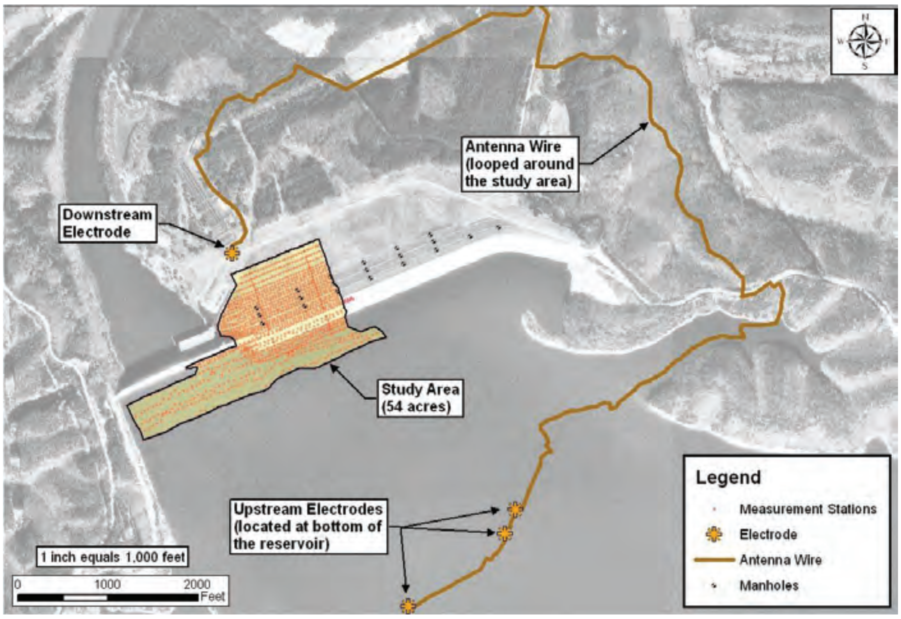

Figure 1. Typical Horizontal Dipole (Plan View)

Each measurement station's X, Y, and Z coordinates are recorded as part of the field work. The stations are designated by small red crosses or "+" signs shown in the figures. These spatial locations are critical to data processing, comparison of surveys, modelling and interpretation.

Equipment

The equipment used to measure the magnetic field induced by electrical current flowing in the groundwater of interest includes: three magnetic sensors oriented in orthogonal directions (X, Y, and Z axis); a data logger used to collect, filter and process the sensor data; a GPS used to spatially define the field locations; and a Windows-based handheld computer used to couple the GPS data with the magnetic field data and store it for subsequent reduction and interpretation. All of this equipment is attached to a surveyor's pole and hand carried to each field station (see **Figure 2**).

The Willowstick, filters and monitors various frequencies, amplifies the signals through noise-reduction algorithms, measures the strength of the signature magnetic field and converts all the information into a recordable data set that is ready for subsequent processing and corrections (collectively called data reduction).

Physical Principles involved in Willowstick

The following principles used by Willowstick show how the technique can be applied to identify, map and model electric current flow paths that infer groundwater distribution patterns through the subsurface:

- Electric current follows the path of least resistance. Groundwater is generally the best subsurface electrical conductor.
- Electrical current flowing in a conductor generates a magnetic field with characteristics that reflect the location of its source.
- Based on Maxwell's equations, an alternating electrical current in a conductor will generate an alternating magnetic field around the conductor. The converse is also true. An alternating magnetic field will generate an alternating electrical current in

Figure 2. Willowstick Instrument

a conductor that is under the influence of the alternating magnetic field.

- Two coils in close proximity to each other can be coupled magnetically. A transformer is a special case of two magnetically coupled coils. The electric current in the primary coil creates a magnetic field which then induces an electric current in the secondary coil, completing the magnetic coupling. The primary coil in the Willowstick technology is created by a large primary loop that consists of the antenna wire, electrodes and the preferential conductive pathways (groundwater) in the subsurface between the electrodes. The Willowstick technology's secondary coils are in the magnetic receiver. When the magnetic receiver is under the influence of the magnetic field emanating from the primary coil (conductive subsurface flow path), the three receiver coils sense and measure the strength of the magnetic field emanating from the primary coil.
- Conductive features will gather electrical current flowing in the ground. This is referred to as current gathering. Electric current flowing in the ground will follow long conductors or conductive zones that facilitate conduction between point A and point B (i.e. the two electrodes). The long subsurface conductors or semi-conductors can be formed into four general categories:

1. The first class of conductors is subsurface groundwater flow paths and channels. When electric current is biased to flow through a subsurface study area, the electrical current will tend to concentrate and flow in the most conductive medium, which is the groundwater/ seepage/leakage.
2. The second class of conductors is 'culture', or any long continuous conductor that is man-made. These include: communication cables, overhead and underground power lines, underground metallic pipelines, metal fences (chain link, barb wire, etc.), railway tracks, steel guardrails and other elongated continuous conductors. The locations of such are usually known, thus they can be accounted for when interpreting the data.
3. The third class of conductor is wet clays, which often pose a problem in DC resistivity and other electrical or electronmagnetic (EM) methods. Near surface clays can act as a "shield" and cause much difficulty measuring the electrical properties of materials beneath. The Willowstick method has two advantages: Firstly, it measures the magnetic field response which is not directly affected by wet clays as are the electric field or EM field. Secondly, by directly energising the medium of interest, greater control can be maintained in most cases to minimise the amount of electric current straying and flowing in the wet clays. In most cases, natural subsurface waters moving through a channel will have a lower resistivity than even wet clays, so the current will tend to focus in these if the proper energising perspective is employed.
4. The fourth class of conductor to consider is any and all other geologic semi-conductors such as graphite-bearing shale, sulphide ores and iron formations. Because of their nature and distribution, these geologic materials are rarely present in significant quantities to cause a problem around Willowstick surveys.

Interpretation and Modelling

After data reduction is complete, one or more footprint maps are created to reveal the patterns of electrical current flow in the subsurface by showing contrast between areas of high and low electrical conductance.

When identifying anomalous patterns in a typical footprint map, it is important to be able to distinguish the three main influences on electric current flow. Besides the groundwater influence, the residual effects of culture and electric current bias still exist in most cases - even though the data reduction processes may have reduced these effects to some degree. These three main influences are discussed below:

1. The groundwater influence in a typical footprint map is obviously of greatest importance. As discussed, Willowstick is based on the principle that its signature electric current is strongly influenced by the presence of groundwater, or areas of higher porosity where groundwater is accumulating and/or flowing. The electrical current will naturally gather and concentrate in these areas or pathways of higher conductivity, which are revealed in the footprint map.
2. The magnetic field may be influenced by culture, which is any conductive man-made feature such as pipelines, power lines or other long continuous conductors. Culture is not always present, but it is often a factor and sometimes very problematic because it tends to be near-surface and can cause large anomalies that mask some of the signal coming from the subsurface. The best approach is to identify all culture before a survey is initiated and either avoid as much of it as possible by strategic survey design or remove its effects when interpreting the data.
3. The magnetic field in any given survey is always subject to electrical current bias because electric current must travel from one electrode to the other in order to complete a circuit. The variable part of the circuit - and the interesting part - is what happens to the electric current when it is allowed to choose its own paths to flow between electrodes. It is always true, however, that 100% of the electric current must concentrate in and out of the points of coupling (the electrodes), and hence the magnetic field tends to grow much stronger as it nears these points.

Interpreting magnetic field contour maps could be compared to reading a topographic quadrangle map. On a topographic map, the ridge lines connecting the peaks could be thought of as the pathways offering the least resistance to traverse. In the same way, these lines in the magnetic field maps represent paths of least resistance for electrical current to follow, although it undergoes some measure of dispersal and regrouping in more complex ways than can be fully described. By identifying these high points and ridges and connecting them together through the study area, the centre position of strong preferential electric current flow can be identified (see dark blue lines in **Figure 3**). Note that the flow paths attributed to culture are highlighted to keep them separate from those attributed to groundwater.

In some cases electric current flow paths produce very tight and revealing anomalies that can be modelled with a fairly high degree of accuracy (depths to within 10% error). Other times electric current flow patterns are not as distinctive and the depth and character can only be roughly estimated—in which case it is very important to have additional data to help characterise the groundwater zone of interest such as, well logs, piezometric data or other geophysical or hydrological data. In any case, the horizontal position of electric current flow paths is generally determined with a high degree of consistency and accuracy.

The results obtained from a Willowstick geophysical investigation are used to make informative decisions concerning how to further confirm, monitor and possibly remediate groundwater problems through a given area of investigation.

Example of the Willowstick System in practice

The Dam is located in Sri Lanka just off the southern tip of India. The dam is located approximately 160km southeast of the capital city, Colombo (see **Figure 4**).

The Samanalawewa Dam impounds the Walawe River, the fifth largest river in Sri Lanka. The Walawe River in conjunction with another major tributary - the Belihul Oya River - flows from the mountains of central Sri Lanka. The two rivers flow in parallel valleys in a southeastern direction and eventually join together. The horizontal separation of the two rivers is roughly 6km, while the vertical separation between the Walawe River (after joining the Belihul Oya River) and Katupath Oya (a tributary of the Walawe) is over 300m. This difference is used as the head for Power Generation.

The construction of the Samanalawewa Dam was started in 1986 and completed in 1991. The dam and resultant reservoir are one of the largest storage facilities created in recent times in Sri Lanka. The dam is a zoned rockfill embankment with clay core, roughly 105m high and 530m long and retains a reservoir with a capacity of 254mm^3. The catchment area of the dam covers nearly 350km^2. Not only is the dam important for its renewable energy resource, but it also serves as a key element for water supply, flood control, fish and wildlife and many other

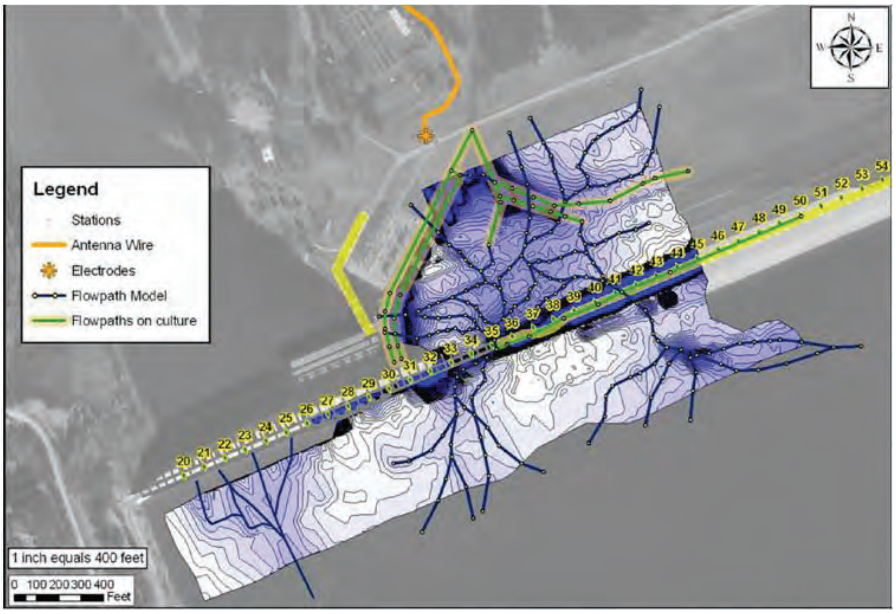

Figure 3. Electric Current Flow Path Model

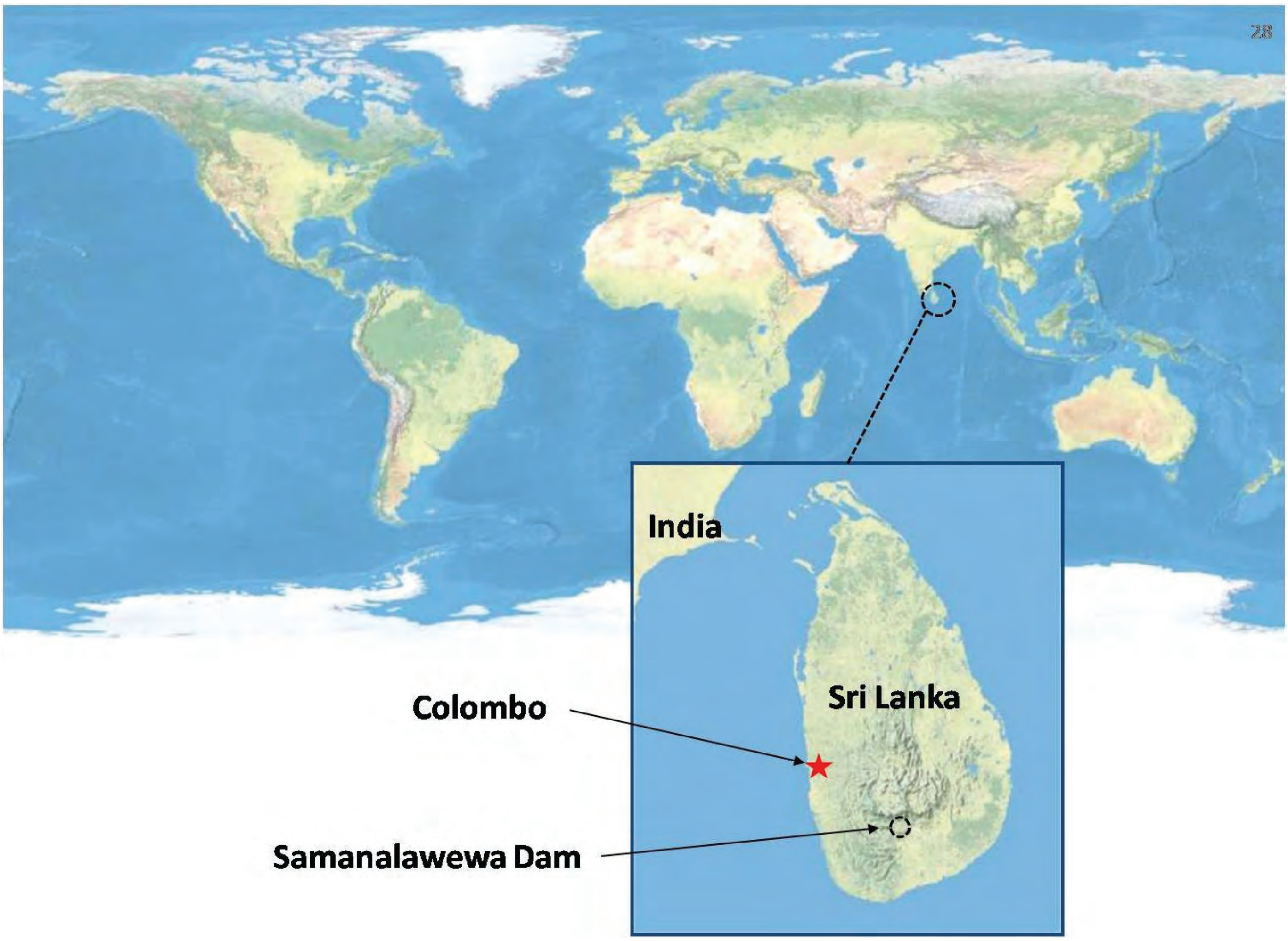

Figure 4. Samanalawewa Dam Location Map

Environment

immeasurable benefits to the country of Sri Lanka (see **Figure 5**).

Geological setting

The project is located within the highlands that lie in the Balangoda region of the central highlands of Sri Lanka.

The reservoir is situated in the 'Highland Complex' with the underlying rock types comprising metamorphic rocks, including granulate gneisses, charnockite, marble and dolomitic marble. These rocks are overlain by a thick weathered layer.

The dam's right abutment and right rim areas consist of karstic terrain. Karst conditions develop from the dissolution of the host rock along fractures, joints and/or bedding planes which become enlarged over time from the saturation and flow of groundwater along these features. In addition to karstic conditions, the right abutment and right rim areas have been subject to extensive folding, faulting and hydrothermal reactions, making the right abutment and right rim areas geologically complex.

During the investigation and construction phases of the reservoir it was recognised that karstic features were likely to be common in the right bank. The right abutment rises up to a peak of elevation of 545m AOD and then descends to a low ridge which extends southwards to form the right bank of the reservoir. The topography beyond the abutment is based on saddles; with four low saddles located on a ridge within a distance of 2.9km of the dam site. These saddles occur at levels varying from 20 to 60m above the top water level of 460m AOD.

A karstic feature, a cave, was discovered during the construction phase 300m upstream of the proposed axis of the dam on the right abutment. This cave appeared to form along a minor fault, one of

Figure 5. Photograph of Samanalawewa Dam

a number of minor, parallel faults which create the saddle features noted above. This and other signs of possible leakage through the right abutment area resulted in an extensive grouting program. Six large cavities, similar to that shown above, were found and sealed with concrete during construction.

Measures to limit water losses

During the construction of the dam four adits were driven along the axis of the dam.

Despite efforts to cut off seepage through the right abutment area, a small spring appeared downstream of the dam upon initial filling of the reservoir (June 1991). The seepage was large enough to suspend filling the reservoir. Additionally, a flat water table was observed responding to the reservoir levels up to a distance of 2.5km from the dam (along the reservoir's right rim). As a remedial measure, a 1,880m long tunnel was drilled beneath the right rim area. From inside the tunnel, a 100m deep by 1,600m-long grout curtain was constructed.

'Leakage incident' and subsequent remediation

On 22nd October 1992, water burst out of an area downstream of the right abutment of the embankment when the water had reached a level of 439.01m OD. The water level was immediately lowered to 430m AOD over a period of three weeks ending on 11th November 1992. However, groundwater levels in the right abutment area were kept high as a result of a blockage at the downstream end of the 'karst/pipe feature'. Once this blockage had been removed the groundwater level dropped by 2-3m in the abutment area. Nearly 25000m^3 of earth were washed away from the adjacent hillside.

The next remediation effort consisted of installing a dumping of clay from barges in an attempt to slow seepage flowing out of the reservoir into suspected ingress areas along the noted fault zones. However, after installing nearly 50,000m^3 of clay, the leak was not stopped. No reduction was noted after the first phase dumping but after the second phase, it was reported that the clay blanket helped reduce marginally the groundwater pressure in the right abutment.

Willowstick survey layout

The Willowstick investigation of the right abutment study area employed one horizontal dipole electrode configuration to energise the subsurface study area. The original scope of work was limited to only one demonstration survey.

Shortly after the fieldwork was initiated and based on preliminary results, Willowstick suggested a second survey. This second survey was targeted to investigate possible seepage through the right rim grout curtain area. This work was limited to a minimal number of measurement stations and was done to investigate whether or not seepage was a problem through the grout curtain. The intention of the additional work was not to detail or model seepage flow paths through the right rim grout curtain. Rather, it was to identify if major seepage path(s) exist through the grout curtain and if so, to determine what additional work should be performed to fully characterise seepage through or beneath the right rim's grout curtain study area.

In performing the two surveys, an injection electrode was placed in the reservoir (some distance from the upstream face of the dam and right rim area). A return electrode was strategically placed in contact with seepage flowing from the hillside down-gradient of the embankment.

Figure 6 presents an interpretation of Survey 1 results. This figure shows the positions of the ECF modelled flow paths (vertical and horizontal alignment).

It appears, based on these posted elevations, that seepage north of the tunnel is near the elevation of the reservoir level at the time of the survey (about 440m). The model also suggests that seepage north of the tunnel occurs above the tunnel and finds an opening in the adit's grout curtain very near the 440m elevation. Seepage south of the tunnel flows a few metres deeper at elevation 438m. At the location where the two flow paths converge, east of the adit, the elevation of the preferential flow path drops a little more rapidly as it flows to the discharge point out of the hillside. It is important that all elevations, as well as horizontal positions, are the result of a relatively inaccurate GPS

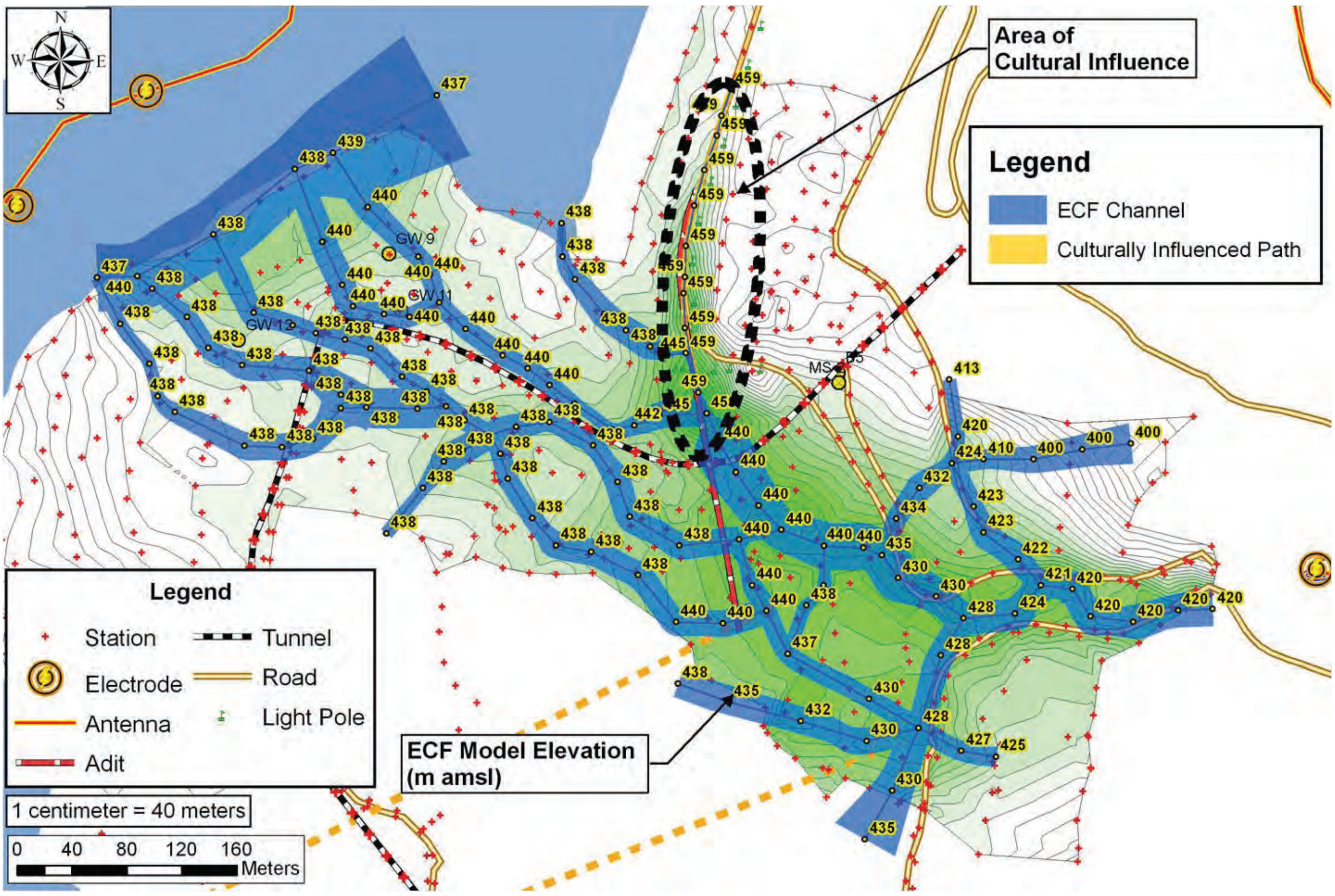

Figure 6. ECF Model with Posted Elevations

Environment

data set. Nevertheless, the data still provide a good indication of how seepage flows past the dam.

Seepage through the open gap in the tunnel's grout curtain appears to be slightly below reservoir water level as well. Had the grout curtain been placed below the tunnel in the gap, seepage would still have passed through this area because the seepage flow is above the tunnel. Where the grout curtain was placed above the tunnel, just west of the adit's grout curtain, seepage is split by the upward vertical grout curtain and some flows through the adit grout curtain and some flows south of the tunnel around the adit's grout curtain.

In conclusion, the results of the investigation suggest that there is a series of braided seepage flow paths north and south of the tunnel that run beneath the right abutment study area. Seepage appears to concentrate around the right side of the dam rather than underneath or through the dam's earthen embankment. There is some seepage occurring along the right rim grout curtain, but not to the extent that it is flowing through the right abutment study area. It has been recommended any seepage through karst topography needs to be carefully characterised, monitored and possibly remediated to ensure the integrity of the reservoir as well as the safety of those residing downstream of the dam.

Remedial works

The Willowstick survey has confirmed two main areas where the cut-off is compromised – one on the bend of the tunnel and the other where the original cut-off crosses the tunnel. Having identified the location of what are believed to be the two largest sources of leakage through the right abutment area, it is possible to undertake cost effective remedial works at the Dam to ensure the safety and integrity and to bring the reservoir back to its proposed top water level and so retain its ability to generate energy.

The work will involve measures:

1. to close two significant gaps in the existing supplementary grout curtain
2. to locate and fill major karstic features at approximate elevation of 438m.

Conclusion

The Willowstick technique allows individual flow paths to be mapped both in plan and elevation at depths in excess of 200m to an accuracy of 0.1m. This enables remedial works to be focused on the areas of defect and generate savings of millions of pounds on large schemes, where without this knowledge, extensive grouting or cut-off construction schemes would be necessary.

End note

The Willowstick technique can and has been used in a number of other fields where the passage of water needs to be traced. Examples would be the tracing of groundwater into excavations or tunnels, loss of water from canals, environmental pollution into bodies of water, and even tracing deposits of gold in waste tips where the water which is sluiced onto the tips adheres preferentially to the gold!

Acknowledgement

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Roman snail: An introduction to its ecology and legal protection

Abstract

In 2008, the Roman snail *Helix pomatia* was added to Schedule 5 of the Wildlife and Countryside Act 1981 (as amended), and it became an offence to intentionally kill, injure or take individuals of this species (as did possession and sale). Also known as the 'edible snail', the primary reason for its legal protection in England and Wales (and elsewhere in Europe) was an increasing trend in collection of large numbers by amateur cooks and for commercial use in restaurants. However, the legal protection this species is now afforded has implications for development projects. Distributed throughout south-east England (but especially the North Downs) and through the Chilterns and Cotswolds, and occupying a broad range of habitats (where suitable soils are present), this species could occur on a wide variety of sites. This article provides an introduction to Roman snail ecology and licensing requirements, and illustrates these using a case study in Surrey – the M25 Controlled Motorways Scheme.

Background

Atkins ecologists first came across Roman snails in early 2009, when working on behalf of the Highways Agency, undertaking an Environmental Assessment as part of proposals for the installation of new gantries along a stretch of the M25 motorway in Surrey (the M25 Controlled Motorways scheme). An empty Roman snail shell was found during an extended Phase 1 habitat survey, at the base of a steep chalk section of the motorway verge between junctions 7 and 8 of the M25. On a subsequent nocturnal survey a live individual was found, in an area of long, semi-improved grassland with dense patches of bramble, close to junction 8. Atkins ecologists have also found Roman snails on another section of the M25 motorway (close to junction 6), when working on a separate project for the Highways Agency. Shells were found within plantation woodland on the verge and live individuals have been spotted numerous times in the tussocky grassland situated directly behind the woodland.

Figure 1. Roman snail habitat on M25 verge

Figure 2. Roman snail habitat on M25 verge

As a result of these findings, and a need to resolve the issue of the presence of this legally protected species within proposed construction areas for the above scheme, further surveys have been carried out and appropriate licences sought.

Habitat requirements and distribution

The Roman snail is known to inhabit open woodland, rough and tussocky grassland, hedge banks, chalk quarries and areas of scattered scrub. **Figures 1** and **2** show the areas of the M25 motorway verge where Roman snails have been found.

This species requires loose, friable soil for burying into for hibernation and also for depositing eggs. Lime-rich, free draining soil is a habitat requirement in the UK and studies have found a preference for south-facing slopes[1]. Roman snails will not occur in sandy soil. They will also avoid grazed grassland and very open, exposed habitats.

Environment

Figure 3 shows a UK distribution map for Roman snail[2]. The species is not native to the UK and is thought to have been introduced by the Romans. Much of its distribution in the UK is considered likely to be due to local introductions by humans. There are documented introductions elsewhere in England and also in Scotland and Ireland, and these are still shown on some distribution maps, but these introduced animals rarely survived for very long[2]. This was presumably because soil and/or weather conditions were not suitable. The main hotspots for populations of Roman snails in England are along the North Downs (from Surrey to Kent), the Chilterns (especially in Hertfordshire) and throughout the Cotswolds and Mendip Hills fringes. There are also documented populations in Cambridgeshire.

Life history

Many aspects of the Roman snail's life history and behaviour contribute to its vulnerability to over-exploitation. In particular, their tendency to aggregate in high numbers and disperse only short distances leaves them vulnerable to collection. Individual snails may spend their entire lives within an area of approximately 30m in diameter and take two to five years to reach maturity and reproductive success may be low, with many British populations found to have a low proportion of young snails[3].

In England, Roman snails are typically active from May to August. The earliest and latest dates for activity in an area of the Cotswolds were April 30th and September 1st [3], with peaks in activity most likely in May and June[4].

Roman snails hibernate in the ground by digging down into loose soils, pulling vegetation and soil over the top to close the top of the entrance to their chamber. They remain in hibernation until spring.

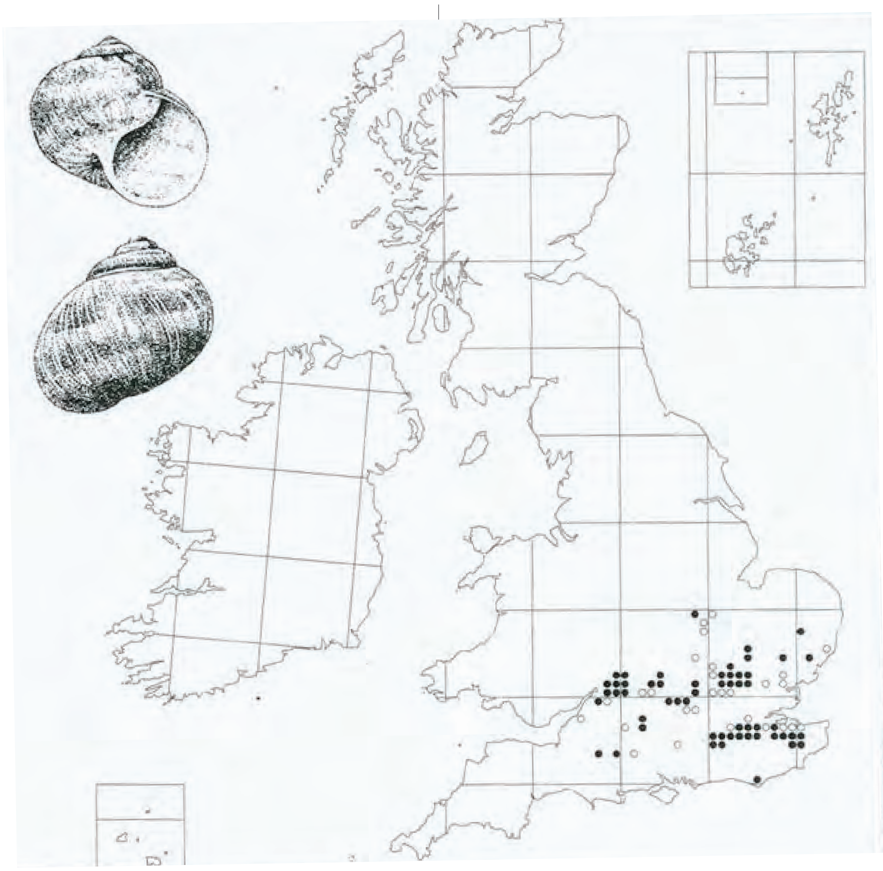

Figure 3. Distribution map for the Roman snail, from Kerney (1999)

Identification

Adult Roman snail shells are typically larger than those of other snail species in England, measuring up to 5 cm across and displaying a pattern of brown bands (see **Figure 4**). Crucially, the bands on their shell lack the zig-zag pattern found on the garden snail *Cornu aspersum* (= *Helix aspersa*- see **Figure 5**). The body of the Roman snail is pale grey and measures up to 10 cm long on adults.

Empty Roman snail shells often appear very pale and lack the brown colouration shown in **Figure 4**, as do juvenile Roman snail shells (shown in **Figure 6**). Empty shells become 'bleached' and in this state are usually more than one year old[4].

Figure 4. Adult Roman snail (Photograph: Dr Martin Willing)

Figure 5. Roman snail shell (left), garden snail shell (right)

Figure 6. Adult Roman snail shell (left), juvenile Roman snail (right)

Surveying for Roman snails

Whilst no standard published survey technique for Roman snails currently exists, it is considered that the combination of careful hand searches and one or two nocturnal torch surveys in suitable weather conditions, as described below, will allow an assessment of presence or absence of Roman snail at a site.

Daytime hand searches

Two survey techniques were used by Atkins for the M25 Controlled Motorways scheme, once the presence of the species had been confirmed, following the identification of an old shell during the initial extended Phase 1 habitat surveys in 2009. Hand searches of areas of habitat to be affected were carried out. This involved searching through areas of long grass and scrub by hand, looking for Roman snails and old shells. Particular attention was paid to searching underneath logs, brash and artificial refuges present on the verge of the motorway. Some gantry locations were ruled as not suitable for the species due to the presence of sandy soils. This hand searching technique was effective because each of the footprints for gantry construction were relatively small; the working area for each gantry footing (i.e. total vegetation clearance) was a maximum of ten metres by fifteen metres (150m²).

Figure 7. Hand searching for Roman snails

In larger areas of habitat, attention would best be focused on log piles and areas that could provide refuge (see **Figure 7**). This is best carried out during the snail's active period (May to August), after recent rainfall, especially in warm, humid conditions. Individuals will bury into the topsoil during prolonged hot/dry spells. At sites with well-established colonies, evidence of Roman snail presence can be found at anytime of the year, in the form of empty shells.

The tendency for Roman snails to aggregate in high numbers and the longevity of their shells means that hand searching over relatively small areas is an effective way to search for evidence of this species.

Torch surveys

In areas deemed potentially suitable for Roman snails, a nocturnal survey was also carried out in June, in order to look for active Roman snails. Ideal timing for torch surveys is late April to early June. This involved searching areas with a powerful torch at least one hour after sunset. This survey technique relies on appropriate weather conditions; it must be raining, have rained in the last 24 hours or be humid and it should also be warm.

A juvenile Roman snail was found during the torch survey for the M25 Controlled Motorways project.

Legislation and licensing

Roman Snail was added to Schedule 5 of the Wildlife and Countryside Act in April 2008. It is not a European Protected Species, although it does receive legal protection in other European countries. In the UK, it is protected in relation to Section 9(1), (2) and (5) of the Wildlife and Countryside Act only. This means that it is an offence to intentionally kill, injure or take this species. It is also an offence to possess a live or dead Roman snail (possession is only an offence if it has been illegally taken from the wild) and it is also protected against sale. It is not an offence to disturb Roman snail or to damage or destroy breeding places or resting places of this species. However, although disturbance is not an offence, a licence is needed to handle Roman snails, however briefly, because it is protected against ‘taking’. This has implications for consultants carrying out surveys for this species. It is necessary to obtain a licence from Natural England for the purposes of science and education to allow to you to pick up and examine Roman snails.

Furthermore, where Roman snails occur within areas that are to be affected by development proposals, such that there is a need to move them to avoid killing or injuring of individuals, any intentional movement of Roman snails must be licensed or should be covered by a relevant defence in the legislation, because moving Roman snails, even short distances, constitutes ‘taking’.

Licences can only be issued for specific purposes under the Wildlife and Countryside Act. There is no licensing purpose for development works. However, Natural England will consider issuing a licence for conservation purposes in certain circumstances. Any conservation licence application for Roman snails will need to demonstrate that the work proposed is essential and the impacts to the species cannot be avoided in any way. It would also need to demonstrate that the work will have some conservation benefit for the species. There is no standard methodology currently available for dealing with Roman snails and each licence application will be considered by Natural England on a case-by-case basis. The licence application for the M25 Controlled Motorways scheme is presented below as a case study example, to highlight the main issues for consideration.

Case study: licence application for the M25 Controlled Motorways scheme

A licence application for this scheme was made to Natural England in August 2010 and included information on four key areas, summarised below.

- Background to the project and details of why the work needed to go ahead.
- This included details about the scheme and how it would deliver safety improvements to the relevant section of the M25 motorway. Background to the Roman snail surveys and the habitats to be affected were provided. Across the 18 new gantry locations, vegetation clearance equalled 0.27ha with a permanent habitat loss of 0.11ha.
- Details of the population, i.e. locations and numbers involved and context in the wider area.
- The locations for each of the new gantries were provided, along with a brief description of habitat within each area. The results of the Roman snail surveys were set out. The location of this scheme, close to the North Downs (a hotspot for Roman snail in England) and within an area of well-connected habitat (the motorway verge) meant that populations were likely to be more robust than smaller populations elsewhere.
- Setting out the conservation aims and how these will be achieved.
- The conservation aim of the proposal in the licence application was to ensure the future longevity of the population of Roman snails in the area and help to maintain the conservation status of this species in the local area.
- Five new log piles would be created in areas outside of the gantry locations, in areas of habitat suitable for Roman snails to provide an enhancement to these species. Locations would be targeted at areas where woody cover is sparse. Log piles would be made from trees cut down as part of the gantry clearance and would be created under supervision by the ecologist.
- Areas of vegetation clearance would be hand searched for Roman snails and any individuals found would be moved to the surrounding suitable habitat (not more than 20 - 30m from where they were found). This would take place outside of the hibernation period.
- Fencing would be erected around each of the works areas at each new gantry location. This fencing would be designed to deter Roman snails from re-entering areas prior to works commencing. Fencing would be 13 mm diameter chicken wire netting with metal stakes used at the corners for support. This sized mesh is small enough to prevent Roman snails getting through, due to the size of their shells, whilst containing holes large enough to discourage movement of snails up the fence. The fence would be buried in the ground to a depth of approximately 30 cm to prevent snails burrowing beneath. The top of the wire netting would be folded outward to create a ‘lip’ on the outside to further deter snails from entering. The fences would be 1m high.
- One monitoring survey for Roman snails would take place in the year following completion of the works. This would take place within habitats around all of the new gantries and also immediately adjacent to the new gantries. The results of the survey would be assessed to ensure that the existing distribution of Roman snail within the local area has been maintained and would be used to inform further mitigation, if appropriate.
- Results of the monitoring survey would be passed to the Conchological Society national non-marine recording scheme and the local biodiversity records centre.

Delivering habitat enhancements for Roman snails will depend on the conditions at the site, but as well as creating log piles, could also be achieved by creating or introducing a base-rich, friable topsoil. In more open areas, creating more cover through planting of scattered scrub, or relaxation of management regimes could deliver enhancements. Woodland edge could be improved through the creation of ecotone habitat where this does not already exist.

The above application was granted by Natural England. However, subsequently a decision was taken by the Highways Agency not to build new gantries in this part of the M25 Controlled Motorways scheme and, therefore, this licence will not now be implemented.

Summary

The Roman snail is a relatively easy species to identify once familiar with its characteristics. Identifying the potential presence of the species can be achieved through understanding of its habitat requirements and will be aided by the fact that, broadly, its distribution is quite well understood and likely to be relatively unchanging in England due to its inability to colonise new areas quickly. However, increased surveying and reporting for the species, now it is legally protected, could lead to amendments to the distribution map and it would, no doubt, be beneficial to send records to local biological record centres and to the Conchological Society of Great Britain and Ireland.

Dealing with Roman snails on development sites is relatively new and mitigation and habitat enhancement measures are currently largely untested. Collation of information from future projects will enable ecologists and stakeholders to refine techniques and test new approaches. As with habitat enhancements for other species, measures to improve habitats for Roman snails are likely to lead to benefits for other species in the local area.

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Gainsborough flood alleviation scheme: Improving project delivery through an integrated team approach to reusing existing assets

Abstract

This paper shows how an integrated team enabled the Environment Agency to successfully deliver a cost effective and sustainable solution to provide improved flood protection to over 2600 properties in Gainsborough. The team worked together to develop and deliver innovative and sustainable solutions to achieve the scheme objectives. This approach provided programme and budget security that resulted in an outturn cost of some £3 million under the £19.9 million scheme budget.

The team identified opportunities to refine the scheme and its implementation to enable costs to be reduced and to manage and mitigate risks. The project team combined detailed condition assessment and numerical analysis of the existing flood defence structures to implement a strategy to re-use many of them, most of which were originally thought to be at the end of their design life. To achieve this, the selected solution to strengthen 700m of existing sheet piled flood defence walls was to install over 440 ground anchors, working over the tidal River Trent. Additionally, rather than reconstruct 1.75km of existing earth flood defences, plastic sheet piles were used to strengthen them. These were installed with minimum disturbance to the existing defences.

Keywords: Floods & floodworks, Retaining walls, Sustainability

Introduction

The Environment Agency's Gainsborough Flood Alleviation Scheme improved the condition of the town's defences and reduced annual flood risk to over 2,600 properties from the 1 in 70 annual chance event to a 1 in 200 annual chance event. Prior to detailed design, the project appraisal process developed a scheme that would deliver the objectives of reducing flood risk and ensuring the integrity of the defences, whilst giving a cost saving of around £7 million compared to the pre-feasibility estimates for the replacement of life expired assets. This scheme was then completed with a final overall cost of £16.8 million. This was a further saving of around £3 million against the approved construction phase budget of £19.9 million.

During design and construction the project team identified opportunities to refine the scheme and its implementation to enable costs to be reduced and to manage and mitigate risks. This paper summarises the key innovation and best practice items that the team developed and implemented to deliver a sustainable and cost effective solution to reducing flood risk to Gainsborough.

Gainsborough is on the eastern bank of the lower tidal reach of the River Trent. The town is protected from flooding by nearly 4km of defences in the form of sheet pile walls, mass gravity walls and earth embankments

(**Figure 1**). This pattern has arisen from the pressures on space along the river from commerce and development. Gainsborough is particularly vulnerable to flooding due to the meeting of tidal and fluvial floodwaters, and the low-lying nature of the town and buildings that are located close to the river.

Following a series of condition surveys and studies, it was established that generally, with the exception of one failed wall, the existing flood defences did not show signs of major distress. However, the analysis indicated that many defences would fail under worst credible loading conditions, i.e. during and after prolonged flooding. For each individual flood defence options were considered to replace, improve or continue to monitor the asset. Assessment was made of the residual risks and consequences associated with each option for consideration by the Environment Agency, so that a decision could be made on the most cost effective way forward.

The use of a detailed condition assessment coupled with numerical analysis, enabled the tabling of a strategy to re-use many of the existing flood defences including large areas of sheet piles, most of which were originally thought to be at the end of their design life. By balancing cost and risk, a sustainable and cost effective approach for ensuring the continuity of flood protection to Gainsborough was developed.

Construction works commenced in June 2006 and were completed in September 2010. The works were designed and delivered by an integrated project team, comprising the client, designer and contractor. The core team was drawn from Environment Agency framework partners. This facilitated the development of the integrated team approach as the team members were familiar with the client's objectives and procedures. The core team was supported by selected specialist contractors as the key design and construction elements were developed and delivered.

Figure 1. Typical river frontage in Gainsborough

The integrated team approach was the key to successful delivery of the project and crucial to this was the development of a highly performing team with the appropriate culture and attitude. The team developed with a common understanding of the aims, objectives, desired outcomes and concept of the scheme. The approach was to encourage integration of the design team and the contractor's temporary works team to develop practical, safe and efficient solutions and construction methods and included continual working with the client representative. Preferred solutions were selected using a risk-based approach to design development and decision making taking an active consideration of residual risks. This was particularly relevant as the preferred solutions were to strengthen existing assets rather than build new. During construction, a rigorous culture of challenge to any proposed changes and risk mitigation options was developed to ensure that value for money was maintained through to completion of the scheme.

History of flooding and flood alleviation schemes in Gainsborough

As a consequence of the major floods in 1947, a flood relief scheme for Gainsborough was implemented in the early 1950s to give added protection to the town. This scheme provided a rigid flood defence along the river frontage of the town, with many of the defences constructed on existing structures, some of which were already over 100 years old. A significant tidal surge in 1954 resulted in the defences being raised for a second time, usually in the form of additional concrete capping to existing river walls and frontages.

In 1990/91 the Environment Agency's predecessor, the National Rivers Authority, commissioned an asset survey. The survey concluded that, having been in place for over forty years and in most places being founded on original structures, the defences were unlikely to last more than five years and without intervention were likely to fail.

Works to replace 800m of the existing defences that were in the worst condition were completed in 2000 at a cost of approximately £20 million. These works included traditional new sheet piled walls together with a piled free-standing structure in the bed of the river. The remaining assets, which were not thought to be in urgent need of repair in the early 1990's, were the subject of the recent scheme.

Asset condition survey and appraisal

In 2004-05 an asset condition survey was undertaken on the remaining original defences. The survey established the residual life of the flood defences and devised a strategy for improvement works. The assessment concluded

Figure 2. Existing mass gravity flood wall with significant cracking

that, rather than replacement, the residual life of most of the flood defences could be extended by implementing strengthening works and improvement measures. The benefit was a reduced use of natural resources, lower construction cost and less impact to the riverside community.

The project team, incorporating the end user and specialist contractors alongside the designer, developed initial outline designs and budgets to identify options for improving the defences. Options for managing flood risk were identified and risks attached to each of the options were discussed. The consequences of any failure of the walls and defences were then reviewed to ensure that acceptable solutions were chosen.

The main outcome of this work was that for the existing mass gravity wall defences, it was deemed appropriate to implement a monitoring regime to identify further deterioration of their condition. If deterioration was identified then a programme of improvement or replacement works would be developed at that time. This approach deferred the need for a capital replacement scheme for these assets. For the existing anchored sheet pile walls and the earth embankments, the consequences of further deterioration and their possible failure were deemed unacceptable. Therefore a scheme of remedial works to extend the life of the assets was developed.

An economic appraisal of whole life cost and present value, comparing new walls against remedial works to extend life of assets and then replace in 20 or 30 years, confirmed that implementing remedial works now would be the best economic option.

The main improvement works required by the preferred option were:

- Replacement of 50m of failed mass gravity floodwall (**Figure 3**) with a new anchored steel sheet pile retaining wall and flood defence.
- Installation of a plastic sheet pile seepage cut-off to increase the stability of 1.7km of existing earth embankments (**Figures 3** and **4**).
- Raising 550m of earth embankment to increase the standard of flood protection from a 1 in 70, to a 1 in 200 annual chance event.
- Installation of 441 ground anchors and associated steelwork to strengthen 730m of hard defences to reduce the risk of failure (**Figures 5** and **6**).

Figure 3. Existing earth embankment flood defence in poor condition

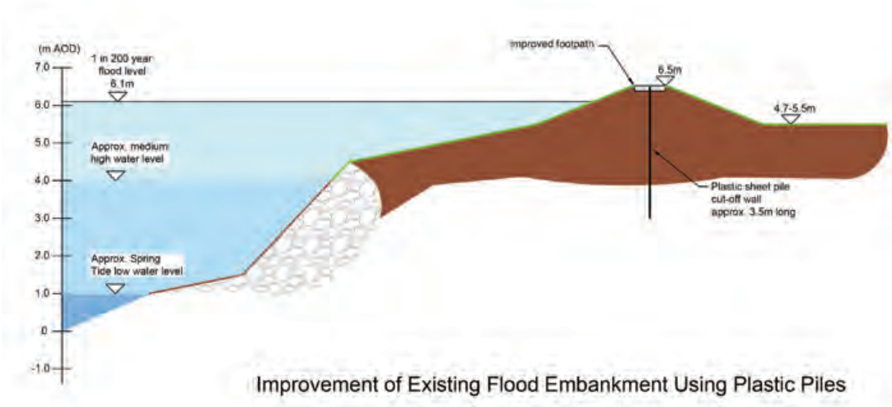

Figure 4. Existing earth defences were strengthened using a plastic pile cut-off

Risk based approach and team work

The nature of the works at Gainsborough meant that there are a significant number of risks to manage. Of primary concern was:

- Working with existing riverside retaining structures with only a limited knowledge of their construction and condition.
- The expectation that existing structures could only take limited additional loading during construction.
- Many of the working areas were constrained by existing properties and structures.
- The River Trent is tidal and subject to a large range of river levels and poses the risk of both fluvial and tidal floods.

Early on, the project team recognised that successful delivery of the scheme within budget required a continuing understanding, management and reduction of the ongoing risks. Consequently during the development of the scheme, the team ensured that they established a thorough understanding of risk through investigations, design and risk workshops.

Consultation regarding the project risks led the team to develop the solution to allow the continued use of the existing defences. This risk based approach, i.e. balancing the cost of improvements against residual risks and consequences, was embraced by the team and client throughout the life of the scheme.

Figure 5. Existing anchored sheet pile walls in visually poor condition

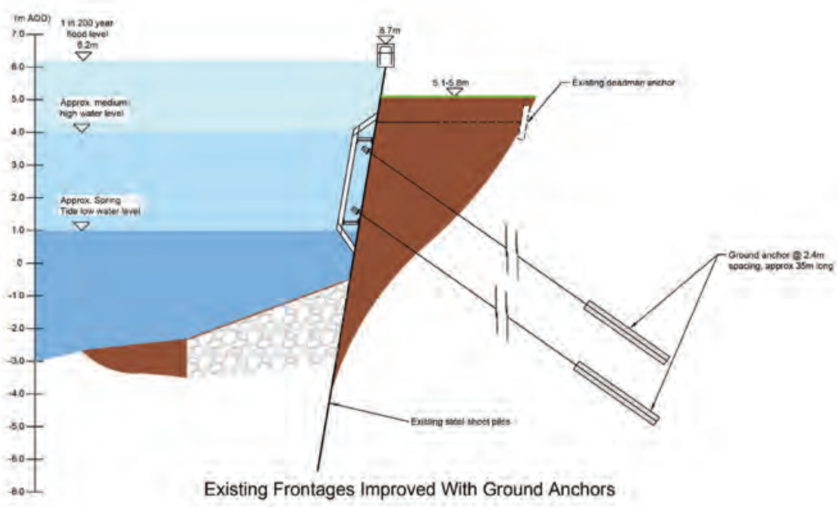

Figure 6. Existing frontages were strengthened using one or two rows of ground anchors with waling beams; fenders protect the finished works from river traffic

The team worked to mitigate risks in both the permanent works and in particular the temporary works used for construction. Without this commitment from all parties to a risk based approach, the project would not have delivered the innovative solutions to improving the defences.

An example of the risk based approach was a decision to reduce the frequency of anchor acceptance testing. The team had recognised that applying existing design codes for anchor testing to the remedial works was not directly appropriate. A more relevant testing regime was then developed in conjunction with the end user to balance construction risks and costs against the requirement to ensure satisfactory performance of the installed anchors.

This decision led to a significant reduction in the time required for anchor testing. This was particularly beneficial as the testing of individual anchors was constrained by the availability of suitable working windows between the tides. Typically this window could be as little as one to two hours, giving a testing rate of only two per day. Following testing of the first sets of anchors installed, we were able to reduce the frequency of testing to an average of 5% and 10% (bottom row and top row of anchors). This reduction led to a saving of five months on the overall construction programme, contributing to the overall cost saving on the scheme.

Sustainability and innovation

In delivering the scheme the focus was on maximising value whilst ensuring the upgraded assets were sustainable in both design and construction. The very nature of delivering the works whilst striving for value and sustainable solutions encouraged innovative ideas from the team for both permanent and temporary works. This approach to value engineering and openness to the use of innovation was the key to successful project delivery. Understanding product function and looking at how best to achieve this functionality always remained at the forefront of the team's thoughts. This section highlights some of the main sustainability and innovation achievements of the scheme.

Plastic piles

The use of plastic piles to strengthen the existing earth embankments provided an alternative to widening of the embankments and minimised the amount of imported material required (**Figure 7**). This use of plastic piles, containing recycled material, was one of the first for the Environment Agency and

Gainsborough has subsequently been used as an example site for other schemes. Additionally, during the construction stage, the design of earthworks for raising the defence levels was adjusted to enable the use of surplus excavated material that was obtained from the creation of a pond elsewhere on the scheme. This avoided disposal costs and avoided the need to import new material.

Strengthening sheet piled frontages

Using ground anchors to strengthen the existing steel sheet pile walls to extend the life of the assets, as opposed to constructing new walls, as well as being innovative proved to be sustainable. The reuse of the existing flood defences created clear carbon benefits, providing a saving of approximately 1,100t of carbon for hard defences alone, compared to the provision of a new steel sheet pile wall.

Innovation was required to deliver the installation of the ground anchors in the working environment of a major tidal river. This was achieved with drilling platforms hung from the existing walls. To enable anchor drilling to be undertaken during all tide levels, and thus prevent pollution and increase productivity, the temporary works included extension tubes from the anchor heads on the existing piles to reach above the predicted high tide levels.

The development of the temporary works was achieved through using the designer's ground and structural modelling to assess the existing flood defences strength, the sub-contractor's previous experience with ground anchors and the main contractor's temporary works expertise and their abilities to control safe systems of work. The team continued to work closely throughout construction solving technical challenges with bespoke temporary and permanent works solutions. The use of temporary working platforms avoided the need to build extensive temporary cofferdams in the river (**Figure 8**). To improve operational efficiency a small range of platforms were developed that could easily and rapidly be moved between locations to cater for the full range of wall types and alignments of the existing frontages.

Figure 7. Installing plastic sheet piles to prevent seepage in existing earth flood defence

Tidal monitoring

The main constraint to implementing the chosen option of strengthening the existing frontages was working over a fast flowing tidal river with a tidal range of 4.5m. During the early stages of construction, prior to commencing works in the river, the team implemented a method of detailed tide and river level monitoring. This information was used by the construction planner to identify efficient working windows for the in-river works. This enabled a realistic and cost effective construction programme to be implemented.

Figure 8. Temporary working platform to allow for installation of ground anchors over the tidal River Trent

Tidal monitoring continued throughout the construction phase and this supported an effective and efficient resource programme. The monitoring gave the team a better practical understanding of how the river responded to tides and fluvial flow conditions. This allowed productivity to be maximised enabling the main strengthening works to be completed ahead of programme and at a reduced cost.

Design change

At outline design stage the available information on river levels indicated that it would be possible to install low level anchors during normal tidal windows. However, the tidal monitoring information demonstrated that to implement the original design proposals would require significant temporary works in the form of cofferdams in the river. This would significantly increase the scheme costs by around £3 million and take it above the approved budget. More importantly, the use of cofferdams would generate increased health and safety risks. An alternative would have been to revert to a traditional new build that would have lost the sustainable benefits of reusing the existing assets.

To ensure the scheme remained viable and that risks were avoided, the team revisited the concept and design of the strengthening works in conjunction with the end user. By using the improved understanding of the river levels and available working windows, a modified scheme was established that still achieved the desired outcomes. This was achieved by redesigning the strengthening works with the bottom row of anchors installed at a higher level. This had the effect of reducing the estimated residual life of the strengthened frontages. However it was still assessed to be the most sustainable and preferable solution.

Design refinement

For waling beam installation (**Figure 9**) to ensure an even load distribution between the new beams and the existing sheet piles, the design required a packing material (**Figure 10**). The original design was based on the use of prefabricated steel packers. Due to the varying shapes, position and alignments of the existing piles, each packer was potentially unique requiring bespoke manufacture. During construction an investigation into possible alternative solutions was undertaken. This identified grout and epoxy resin bags as a viable cost effective alternative, a technology borrowed from bridge engineering. The solution provides savings and reductions in installation time, material cost and carbon cost as well as a reduction in health and safety risks compared to the original proposal.

Through the efficient use of materials additional savings were made. The original fender design required bespoke fenders at each location, 630 fenders in total. This was due to different lengths required to fit the varying shape of the existing frontages. The design was revisited, with additional survey information on the sheet pile geometry, to develop a simplified fender solution which could be used consistently throughout the scheme. This reduced the number of different fender sizes required and improved the efficiency of fabrication and installation. As part of this design development process the designer and contractor developed fixing methods that would allow installation in a quick, easy and safe manner (**Figure 11**).

A further review of the requirement for fenders to afford protection to the anchor heads from accidental river vessel impacts generated a significant change in the project team's understanding of the system. The result was to reduce the fender requirement at each anchor from two to one, decreasing the total number of fenders by 300.

Handover

Due to the fragmented locations of the various assets and the large number of individual operations, an advanced asset pre-handover and completion programme was created. This enabled earlier inspections of assets and individual work processes. The advantage of this was in reducing any potential delays and additional costs at the end of the scheme. Due to the constraints of tidal working and the need for specialist temporary works, it was preferable to maintain continuity of

Figure 9. Installing prefabricated waling beam sections – packers are required between the beams and the existing piles to ensure an even transfer of loads

Figure 10. Completed section showing waling beam, grout packers and fenders

working and to avoid the need for any separate remobilisation.

Summary

The key to successful delivery included the development of a highly performing team with the appropriate openness, willingness to be challenged, culture and attitude. The team developed a common understanding of the aims, objectives, desired outcomes and concept of the scheme. The approach was to encourage the integration of the design team and the contractor's temporary works team to develop practical, safe and efficient solutions and construction methods.

Maintaining close communication with the client has been fundamental to success. The development of trust and the transfer of knowledge and understanding has facilitated a risk-based approach to design development and decision making. Overcoming problems and risks jointly as a team meant that better solutions were quickly found and avoided unnecessary and abortive costs. A rigorous culture of challenge to any proposed changes and risk mitigation options was also developed to ensure value for money throughout the design and construction of the scheme.

The Gainsborough flood alleviation scheme was a large complex project that used an innovative approach to the continual management of flood risk by extending the life of existing assets. The nature of the works meant that there were a wide range of risks to manage. By developing a team with the appropriate skills and approach, the scheme has been able to successfully manage risks, identify opportunities to improve the value of the works, and work successfully within the community to deliver the works safely. An integrated team adopting a risk-based approach was the key to this success.

Figure 11. Installing fenders using the quick fixing methods developed by the team

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Lake water budget modeling white paper

Abstract

Each year, dozens of roads constructed of ice are built across the North Slope of Alaska in the winter to facilitate oil exploration. The construction material for these roads – water – comes from thousands of lakes that dot the landscape. The volume of water in the lakes fluctuates as weather moves through its seasonal cycle. The question of whether there will be sufficient water for ice road construction has recently come into focus, particularly in light of the potential shift in weather patterns brought about by climate change. This study focuses on forecasting the available water in a North Slope Lake under the potential effects of climate change. Through the development of a water budget model, which accounts for water entering and leaving the lake through precipitation, evaporation, transpiration, sublimation, groundwater exfiltration and infiltration, and of course extraction for ice road development, a clear method for estimating the remaining water in the lake over time is found.

The forecast is driven by outputs from General Circulation Models (GCM). These models, which are developed and run at several research centres around the world, are controlled by plausible greenhouse gas (GHG) control scenarios set forth by the United Nations (U.N.) Intergovernmental Panel on Climate change. The scenarios cover a spectrum from a world whose governments are regionalized and focused on economic growth to a world where governments are globally connected and focus on environmental protection. Each GHG scenario results in shifts in temperature and precipitation patterns that in turn affect the local weather around the globe.

The primary question for the research is, "If the current regulations continue to be used to permit water extraction for ice roads, what is the risk of a North Slope Lake running out of water, depleting water levels to environmentally dangerous levels, or adversely affecting populations of sensitive species of fish?" To answer this question, the water budget is configured to remove the maximum allowable water each year under the current regulatory guidelines. The model works with 12 separate possible GCM projections – as there are four GCMs and three GHG scenarios per GCM. Using this ensemble of projections, the model assesses remaining water in the lake across the ensemble, to ensure that all plausible scenarios are considered. The final product is a risk assessment chart that shows the percentage of time over the 21st century that the lake in question will be able to meet the regulated demand for water extraction. A worked example in Appendix A demonstrates the creation and running of a water budget model.

Under the North Slope Decision Support System, which is the umbrella project aimed at the larger question of optimally planning ice road alignment, the water budget modelling approach presented in this paper is employed at each lake that can be permitted for water extraction along a proposed ice road. The water budget results are used in an assessment of the feasibility and cost/benefit of the planned ice road.

Introduction

Alaska's North Slope hosts a wealth of natural, cultural, and economic resources. It represents a complex system, not only in terms of the biophysical system and its global importance but also from the standpoint of its social dynamic. Stakeholders of the North Slope include oil companies, regulatory agencies, local tribes, environmental advocacy groups, and researchers. Domestic energy development on the North Slope – particularly in light of changing climate – must be conducted with best management practices that will ensure benefits for all stakeholders. Establishing these practices and ensuring they are followed requires an all-inclusive stakeholder design process; one that results in cost-effective development strategies that fit within a broader context of long term cultural, economic, and environmental sustainability.

Ice roads provide a cost-effective means of oil and gas exploration with minimal impact to the sensitive underlying tundra. These ice structures have become a necessity for oil and gas exploration activities on the North Slope. Due to the large volumes of water required to construct and maintain ice roads and ice pads, their construction and maintenance have become a challenge to water resource managers. With energy consumption on the rise worldwide, water resources managers need a clear understanding of the viability and long-term impact of ice roads and ice pads.

The North Slope Decision Support System (NSDSS) is a technological solution that brings to bear the latest data storage and service, natural systems modeling, and decisions support and collaborative management theories in support of oil and gas exploration. Developed under a research grant from the U.S. Department of Energy's National Energy Technologies Laboratory, the NSDSS is used by all stakeholders as a common workbench for exploring options for ice road construction. The NSDSS explicitly considers optimal water use, direct and cumulative environmental impacts, and multiple objectives and values among stakeholders. Development of the DSS is a collaborative effort of academic and industry personnel with significant stakeholder involvement from multiple agencies of local, state, and federal government, private energy companies, and non-governmental organizations.

Development of the NSDSS has been partitioned among three groups. The University of Alaska at Fairbanks led the overall project administration and development of natural systems models; Texas A&M University led development of the optimization models for ice road alignment planning; the Atkins team, led by Stephen Bourne in Atlanta, Georgia in the United States, led the development of new technologies and the translation of conceptual models developed by the other teams into robust software tools.

This paper focuses on the NSDSS water budget modelling functionality. The water budget model is one of several natural systems models that the NSDSS is capable of producing. Using the NSDSS Environmental Analyst module, users can create and publish water budget models for any of the thousands of lakes on the North Slope of Alaska. The water budget model estimates the storage in the selected lake by solving a seasonal water balance equation as described in the sections below. Users have the option to both conduct an historical analysis or a future analysis. For historical analyses, data from the 30 year record of on-going permanent weather sites plus field surveys is used. For future analyses, the input data is a series of possible realizations of North Slope climate collected from a suite of general circulation models (GCMs). These GCMs have all been run according to varying global green house gas control scenarios laid out by the United Nation's Intergovernmental Panel on Climate Change[1].

The lake forecasting process uses a multiple GCM models, each with multiple GHG scenarios to create an ensemble of possible future conditions. The process uses the water budget model to create a forecast of lake storage based on each of the GCM model-scenario pairs. Through assessing the range of possible future lake storage levels projected by the GCMs, researchers can build understanding of if climate change will have a net drying or wetting effect.

In terms of ice road planning, a water budget model is created for each lake that stakeholders – oil firms, regulatory agencies, tribes, etc. – plan to use to build the ice road. Using the storage forecast results for these lakes, stakeholders can assess the risk of the lakes being unable to provide sufficient water for ice road construction, and use this risk in their decision making process.

Water budget formulation

Water budget models in the NSDSS are specific to the arctic environment of the North Slope. The following formulation begins with a general water budget modeling approach, and then customizes it to the seasonal arctic cycle of freeze and thaw.

A water budget model for a lake and its watershed can be defined as in **Equation 1**, where ΔS is the change in storage for a given time, Q is the net flow out of the system, P is the precipitation flux into the system, ET is the evapotranspiration out of the system, and Extraction is the water used for ice road construction.

Equation 1:

$\Delta S = P - Q - ET - Extraction$

To establish water dynamics, **Equation 1** can be converted to a state space form as follows, where ε is an error term associated with unaccounted fluxes, or systematic errors in the measurements of each flux term.

Equation 2:

$S(t+1) - S(t) = P(t) - Q(t) - ET(t) - Extraction(t) + \varepsilon$

Focusing on the lake itself, and the storage available for building ice roads, we can split **equation 2** into equations focused on the lake and the watershed, respectively:

Equation 3:

Lake: $S_{lake}(t+1) - S_{lake}(t) = P_{lake}(t) \cdot A_{lake} - Q_{lake}(t) - E_{lake}(t) \cdot A_{lake} - Extraction(t) + \varepsilon_{lake}$

Equation 4:

Watershed: $S_{wshed}(t+1) - S_{wshed}(t) = P_{wshed}(t) \cdot A_{wshed} - Q_{wshed}(t) - ET_{wshed}(t) \cdot A_{wshed} + \varepsilon_{wshed}$

Note that the evaporation term for the lake is simply the evaporation from the lake open water body. Note further that the precipitation and evapotranspiration/evaporation terms are typically measured as depths, and therefore must be multiplied by the area of lake or watershed to derive volume.

The following assumptions are made:

1) The flow into the lake, Q_{lake}, is equal to a fraction of the Net Basin Supply from the watershed. Assuming watershed storage stays constant, we can then say:

$Q_{lake} = \alpha Q_{wshed} = \alpha(P_{wshed} - ET_{wshed}) A_{wshed}$

2) The error terms are zero,

The Lake storage is therefore:

Equation 5:

$S_{lake}(t+1) = S_{lake}(t) + P_{lake}(t) \cdot A_{lake} - \alpha[P_{wshed}(t) - ET_{wshed}(t)]A_{wshed} - E_{lake}(t) \cdot A_{lake} - Extraction(t)$

Annual cycle – Alternating runoff coefficient

The annual North Slope hydrologic cycle is punctuated by two events, spring thaw, and fall freeze up (see **Figure 1**). The water budget in **equation 5** above must therefore be split into two seasons, one from spring thaw to freeze up, and the other from freeze up to spring thaw.

The runoff coefficient, α, differs for the two seasons. At spring thaw, the runoff coefficient is very high (e.g. 0.9) because the snow melt occurs over a few days. Over the summer season, however, a much lower coefficient is applied (e.g. 0.4) as movement of water occurs much more slowly and there is more opportunity for groundwater seepage and movement of water to neighboring watersheds. The runoff coefficient must therefore alternate from spring thaw (0.9) to summer (0.4) in the seasonal calculation of the storage time series.

Data sources

The NSDSS water budget model has the ability to do historic models and forecasts. For historical analysis, field data contained within the NSDSS databases is used. For forecasts, projections from General Circulation Models (GCMs) are used. The historical field data in the NSDSS stretches from approximately 1980 to present. The GCM data used in the NSDSS typically stretches from 2001 to 2100.

Using GCMs for climate change impact forecasting is becoming the current state of the art. GCMs are global numerical models of the coupled atmospheric-ocean-land system. They discretize the globe with grids that stretch from the North to the South Pole with grid-cell sizes of approximately 2 degrees latitude and longitude. They are typically run over time frames of many centuries at hourly time steps. Each GCM models hundreds of physical and chemical variables in the global system and produces statistics of those variables. The NSDSS uses the monthly total liquid precipitation

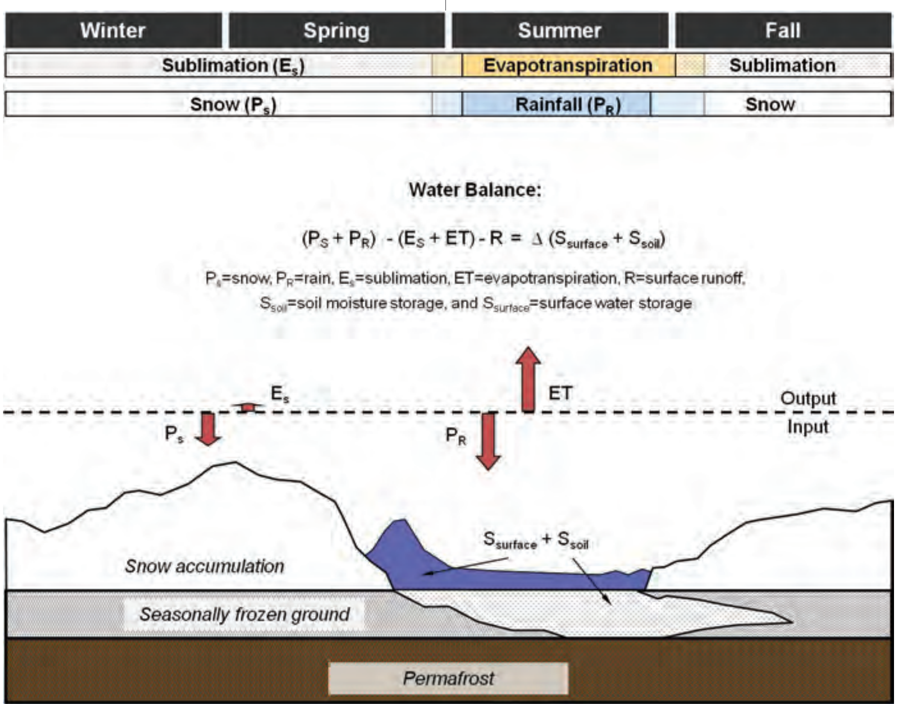

Figure 1. Annual North Slope Hydrologic Cycle (from Tidwell, 2009[2])

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and temperate in the GCM cells that overlie the North Slope as the basis for lake water budget forecasts.

The GCMs are intended to simulate the globe under several future green house gas emissions (GHG) regulation policy scenarios. The scenarios are set by the Intergovernmental Panel on Climate Change (IPCC), which was established by the United Nations Environment Programme and the World Meteorological Organization, and is considered the leading international body for the assessment of climate change.

The scenarios were set in the Special Report on Emissions Scenarios (SRES)[3]. They operate on two spectra – Environmental to Economic and Global to Regional, with the scenarios named according to where they fall on these spectra. For example, an "A" scenario reflects a world focused on economic growth, while a "B" scenario reflects a world focused on environmental preservation. A "1" scenario reflects a "flat" world, where globalization is the goal, while a "2" scenario reflects a regionalized world with comparatively little communication from region to region. The NSDSS uses the A2, B1, and A1b scenarios, which represent a regionalized economically driven world, a globalized environmentally focused world, and a globalized economically driven world, respectively. These scenarios represent a wide variety of possible green-house-gas-control-policies that may occur in the 21st century and therefore provide a comprehensive picture of how local climate may be impacted

Several research centers around the world develop and run their own GCMs. Each center makes its own assumptions about how to model the global system – there are hundreds of modeling choices and assumptions necessary. As such, it has become common practice to use projections from multiple GCMs from multiple centers to derive a comprehensive forecast of global climate. In this NSDSS water budget model, the following GCM model results are used. The data from these models was downloaded from the North Slope Decision Support System's GCM data web service[4], the data from which was originally downloaded from the IPCC data center.

- ECHAM5/MPI-OM (2004) from the Max Planck Institute, Hamburg, Germany
- MIROC3.2(2004) from the Center for Climate Research, Tokyo, Japan
- HadCM3(1998) from the Meteorology Office, Devon, UK
- CNRM-CM3(2004) from the Centre National de Recherches Meteorologiques, Toulouse, France

Time step considerations

For historic analysis within the NSDSS water budget model, field data is used, which typically is recorded at a daily time step, though sometimes sub-daily data is recorded as well as data at irregular time steps during snow surveys. For future analysis, monthly time step data is used from GCM model results. To estimate seasonal fluxes, this data is aggregated up to the seasonal scale. The start and end of seasons is defined using the following triggers.

Spring thaw trigger

Spring thaw occurs at different times each year depending on many factors. To estimate the flux of liquid water from snow at the beginning of spring, the depth of snow at the start of spring thaw must be known. The start of spring is defined within the NSDSS model as the period when the temperature rises above a user-specified level (5 deg. C by default).

For historical data, the liquid water over the lake and over the watershed is calculated by converting the snow depth at the time of spring thaw with a user-specified coefficient (default = 0.15 for both lake and watershed).

For forecast data, since the GCM precipitation flux term is an estimate of liquid water, no snow-to-liquid water conversion is necessary.

Fall freeze up trigger

Similar to the definition of start of spring thaw, the winter season is defined as starting when the temperature goes below a user-specified level (-5 deg. C by default).

Trigger search method

In each year, the spring thaw and fall freeze up triggers are unknown. Indeed, with the GCM-based forecasts, it is possible that a fall freeze up may not occur at all. To find the triggers, the NSDSS uses the following algorithm:

1) For each year in the time series
2) Starting in October search each month until the temperature falls below the freeze up trigger.
3) Define the month when the freeze up trigger occurs as the start of winter.
4) Starting from the start of winter, search each month until the temperature goes above the spring thaw trigger.

In the event that the fall freeze-up does not occur, no winter season is defined for the year in question. That is, the entire year is treated as a single non-freezing season.

Splitting the water budget into winter-time and summer-time

The water budget in **equations 4** and **5**, must be split into two equations each, one for winter-time and one for summer-time as follows:

Winter-time water budget

Equation 6a:

$$S_{lake}(t+1) = S_{lake}(t) + P_{lake}(t)\cdot A_{lake} - \alpha_{winter}[P_{wshed}(t)\text{-}ES_{wshed}(t)]A_{wshed} - ES_{lake}(t)\cdot A_{lake}\text{-}Extraction(t)$$

Equation 6b:

$$S_{wshed}(t+1) - S_{wshed}(t) = P_{wshed}(t)\cdot A_{wshed} - Q_{wshed}(t) - ES_{wshed}(t)\cdot A_{wshed}$$

Summer-time water budget

Equation 7a:

$$S_{lake}(t+1) = S_{lake}(t) + P_{lake}(t)\cdot A_{lake} - \alpha_{summer}[P_{wshed}(t)\text{-}ET_{wshed}(t)]A_{wshed} - E_{lake}(t)\cdot A_{lake} - Extraction(t)$$

Equation 7b:

$$S_{wshed}(t+1) - S_{wshed}(t) = P_{wshed}(t)\cdot A_{wshed} - Q_{wshed}(t) - ET_{wshed}(t)\cdot A_{wshed}$$

Winter-time calculations

The winter time water budget is calculated each year at spring thaw, and estimates the water fluxes over the winter season. The specific fluxes are described below.

Winter precipitation

If historical data is being used as input in the water budget, then the winter time precipitation is calculated as the snow water equivalent (SWE) at spring thaw. SWE is calculated as a fraction of snow depth (default = 0.15).

The best available data for snow depth estimates is sparse at best. Given the high degree of variability in snow depth due to underlying topography and wind, we expect the error associated with this term to be significant. Additional work should be done to increase the accuracy of snow depth estimates in the future.

If GCM data is being used as input, then the winter time precipitation is simply the precipitation value from the GCM model, as the GCM provides liquid water estimates regardless of season.

Sublimation

In **equation 6**, the ET and E terms have been replaced with sublimation (ES) estimates over the lake and watershed, as the landscape is snow-covered over the winter season.

In the current NSDSS version, sublimation is estimated as 20% of the wintertime precipitation.

In future versions, the sublimation terms will be estimated more accurately using Gray and Prowse's method, as described below. An important step to implementing this approach will be the inclusion of estimates of the moisture holding capacity of the overlying air (vapor pressure, etc.) in the NSDSS databases. Currently, no data are available in the forecasting or historical data, though these data are available and can be added to the NSDSS databases in the future.

Under Gray and Prowse's approach, latent energy flux (i.e., sublimation and evaporation) from snowpack can be modeled using a mass transfer equation[5]. Expressed as depth of liquid water:

$$SE = D_e \cdot u_z \cdot (e_s - e_a) \cdot \frac{1}{\lambda_v \cdot \rho_w} \cdot \left(86400\ {}^{sec}\!/_{day}\right)$$

Where:

SE = sublimation and evaporation (m/day);

D_e = bulk transfer coefficient for latent heat energy transfer (MJ/m^3/kPa);

u_z = wind speed measured height z (m/s);

e_s = saturation vapor pressure (kPa);

e_a = atmospheric vapor pressure (kPa);

λ_v = latent heat of vaporization of water (MJ/kg), calculated as a function of temperature (see above);

ρ_w = density of liquid water (constant at 1000 kg/m^3).

The value of D_e has been estimated empirically by several investigators as reported in Gray and Prowse (1993). Values are given in the table below and vary widely. The average of values where wind speed was measured at 1m height is also given as well as the average of all reported values.

D_e x 103 (MJ/m3/kPa)	Wind speed measurement height z (m)	Reference
0.0515	3.2	Hicks and Martin 1972
0.25	2	Male and Granger 1981
0.0662	0.7	Yoshida 1962
0.080	1	USACE 1956
0.100	1	Sverdrup 1936
0.114	1	USACE 1956
0.0217	1	Granger 1977
0.0789	Average of values where z = 1 m	
0.0854	Average of all values	

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Winter runoff coefficient

The coefficient α_{winter} describes the portion of precipitation minus sublimation that makes it from the watershed to the lake at spring thaw. This value is set to 0.9 as a default after Borisova et. al.[6]

Summer-time calculations

The summer-time water budget is calculated each year at freeze-up, and estimates the water fluxes over the summer season. The specific fluxes are described below.

Summer precipitation

The summer precipitation is assumed to be the total precipitation measured at a field site, if field data is used, or the GCM-based estimate of precipitation if forecasting is required.

Evapotranspiration

Evapotranspiration is calculated using the Priestly-Taylor equation[7]. Priestly-Taylor is a potential evaporation model; thus, it is suitable for circumstances where water supply for evaporation is not limited (e.g. lakes, wetlands, etc.). It is a simplification of the Penman equation, where mass transfer factors (atmospheric dryness and wind speed) are assumed to be minor compared to net radiation and can be approximated as a constant function of net radiation.

$$E = \alpha \frac{\Delta}{\Delta + \gamma} (R_n - G) \cdot \frac{1}{\lambda_v \rho_w}$$

Where:

E = evaporation rate (m/day);

α = Priestly-Taylor coefficient (dimensionless);

Δ = slope of the saturated vapor pressure function with respect to temperature (kPa/°C), which can be calculated as a function of temperature (see below);

λ = psychrometric constant (kPa/°C), which can be calculated as a function of temperature (see below);

R_n = net incoming radiation (shortwave and longwave minus reflections) (MJ/m²/day);

G = energy absorbed by ground (MJ/m²/day);

λ_v = latent heat of vaporization of water (MJ/kg), calculated as a function of temperature (see below);

ρ_w = density of liquid water (constant at 1000 kg/m³).

For temperature T (°C), the terms above and one intermediate term can be calculated as follows. Saturation vapor pressure e_s (kPa; an intermediate term in the expressions below):

$$e_s = 0.6108 \cdot \exp\left(\frac{17.27 \cdot T}{237.3 + T}\right)$$

Slope of saturation vapor pressure with respect to temperature Δ:

$$\Delta = \frac{4098 \cdot e_s}{(237.3 + T)^2}$$

Latent heat of vaporization λ_v:

$$\lambda_v = 2.501 - 0.002361 \cdot T$$

Psychrometric constant γ:

$$\lambda = 0.00163 \frac{p}{\lambda_v}$$, where atmospheric pressure p can be assumed to be 101 kPa (sea level).

Over short time scales (less than 1 month), ground heat exchange G is often negligible. For monthly time steps, it can be estimated as (with a unit conversion appended for consistency with other terms):

$$G = \frac{0.14(T_{month2} - T_{month1})}{\left(30\ {}^{days}\!/_{month}\right)}$$

{in units of MJ/m²

Runoff coefficient

The coefficient α_{summer} describes the portion of precipitation minus evapotranspiration that makes it from the watershed to the lake over the summer. This value is set to 0.4 as a default.

Ensemble forecasting using multiple GCM/ green house gas control scenarios

There are 4 GCM models used (ECHAM, CNRM, MIROC, and Hadley), each of which has three green house gas emission control scenarios (SRESa1b, SRESa2, and SRESb1) set by the UN IPCC. This totals 12 different possible realizations of the global earth system. Taken together as an ensemble, these 12 realizations reflect uncertainty in future policy, input data accuracy, methodologies being used in the calculations and, indeed, in the underlying science.

To produce a forecast, the complete ensemble is used to propagate the uncertainty through to the estimates of lake storage. That is, a water budget is calculated using data from each of the 12 GCM/Control Scenario pairs. This results in an ensemble of lake storage forecasts, which can then be used in planning the development of ice roads, and other applications that require estimates of water availability on the North Slope.

Localizing the GCM projections

The GCM data must be localized in order to better reflect the local climate of the region of interest[8-12]. To localize the data, a field site is selected close to the lake of interest. The field site contains a long record (20-30 years) of historical precipitation, temperature, and net radiation data. The NSDSS uses the following algorithm to localize the GCM data:

1) For each variable, precipitation,temperature, and net radiation,
2) Create a monthly climatology of historical data (i.e. monthly averages over the historical time horizon).

3) Create a time series that repeats the climatology year after year over the forecast time horizon.
4) Each GCM has a control run time series that represents a pre-industrial revolution world Subtract the GCM future time series from the GCM control run time series to produce a "delta" time series representing monthly departures from pre industrial revolution climate
5) Add the "delta" time series to the historical climate time series to produce a new localized forecast.

Calculating permitted extraction

The Extraction term in the water budget model is assumed to be the maximum water extraction permitted under current regulations. This is determined by the following decision tree:

Are there fish in the lake?

Yes:

Are any of the fish a sensitive species?

Yes:

Percent of Under Ice Water Permitted = 15%

Maximum Ice Thickness assumed to be 7 ft.

No:

Percent of Under Ice Water Permitted = 30%

Maximum Ice Thickness assumed to be 5 ft.

No:

Percent of Under Ice Water Permitted = 20%

Maximum Ice Thickness assumed to be 5 ft.

Based on the tree above, it is necessary to know some information about lake bathymetry. Currently, the NSDSS requires the user to enter a depth for the lake, and uses a "cone" method, where the lake is assumed to be an inverted cone in shape, to calculate lake volumes. The regulatory agencies use the cone method currently for estimating lake volumes. The user must also specify if the lake contains fish and if the fish are a sensitive species.

Final product

The end result of this assessment is an ensemble forecast of permitted extraction for the lake in question. That is, 12 time series of permitted extraction describe the range of possible permitted extraction levels for each year. This range, of course, represents the collective variability in GHG scenarios prescribed by the U.N. IPCC, and the range in science and technology implemented in the 4 GCMs.

Risk assessment

The 12-time series ensemble of permitted extraction by itself is not especially useful to ice road planning stakeholders, as it does not communicate risk of the lake being unable to provide sufficient water. The NSDSS tool applies an assessment to the ensemble to quantify this risk.

Risk assessment is conducted by using the ice road planning module in the NSDSS. To plan an ice road, the NSDSS user specifies the start point, end point, and the lakes to be used. The NSDSS then uses an optimization algorithm based on the behavior of ants in finding food[13] to find routes that minimize road length and cost, while ensuring they avoid environmentally sensitive areas and keep within regulation. As part of the ice road search process, the NSDSS assesses the risk of insufficient water by using the ensemble of permitted extraction produced by the water budget model. The ensemble is converted to three time series of 5%, median, and 95% permitted extraction levels in each year of the forecast. These time series are in turn converted to probability of exceedance curves, which are then compared to the volume of water required to build the road in question. Using this approach the risk of insufficient water under GCM-projected dry (5%), normal (median), and wet (95%) conditions can be assessed. The worked example in **Appendix A** shows this result in detail.

Conclusion

The water budget modeling approach in the NSDSS reflects the unusual hydrology present in the arctic. Two separate, variable-length, seasons must be defined with temperature-based triggers to reflect a winter season that is shortening due to climate change. Liquid and frozen precipitation, evapotranspiration, sublimation, and run-off each play a large role in the water budget. The water budget model tool in the NSDSS is designed to be flexible in terms of analyzing both historical water budgets and providing GCM-based forecasts. The tool is applicable to any lake on the North Slope, and can be used to publish water budget models, so that others can view them and even use them in subsequent ice road planning exercises. This ability to share models makes it possible to bring hydrologic expertise to the ice road planning process, as hydrologists can create the water budget models which are then used by professional planners in the ice road planning process.

Currently, work is under way with the NSDSS tool to conduct ice road planning case studies with the North Slope stakeholders. In the summer of 2012, workshops will be held in Anchorage, Alaska and Fairbanks, Alaska, to present the case studies and further develop plans for using the NSDSS in the 2013 ice road planning cycle.

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Appendix A

Example of creating a water budget

The following example shows the creation and running of a water budget model.

Step 1. Open the tool by going to NSDSS.net in your internet browser.

Step 2. Click on the Environmental Analysis Tool (EA) in the main toolbar. This will open up the EA widget.

Step 3. Select the "Create a simple water budget model" option in the drop down and click "Go". This will create a 4-step workflow for creating your water budget model (see **Figure 2**).

Step 4. Zoom to the place on the map where your lake of interest is. To do this, simply use your mouse scroller to zoom in and out, and click and drag the map to pan.

Step 5. Expand the Step 1 element in the tree. Click the "Lake Area" node. Click the "Select Lake on Map" option. Click on the lake of interest. The tree should automatically populate with the Lake ID, area, and watershed area. Note that the watershed area is simply the lake area multiplied by a factor that you can specify in the model parameters section of the workflow. **Figure 3** shows that Lake 25935 was selected. It has an area of 2.1M sq. meters.

Step 6. Set the modeling intent for the analysis by clicking the Modeling Intent node in the tree and clicking the Future option. Modeling intent informs the NSDSS of the type of input data you will be searching for. As there is significant data to be used as input - particularly in forecasts where 12 GCM/Scenarios are used – the NSDSS is equipped with the ability to suggest the model input data. To make the suggestion, the modeling intent is required.

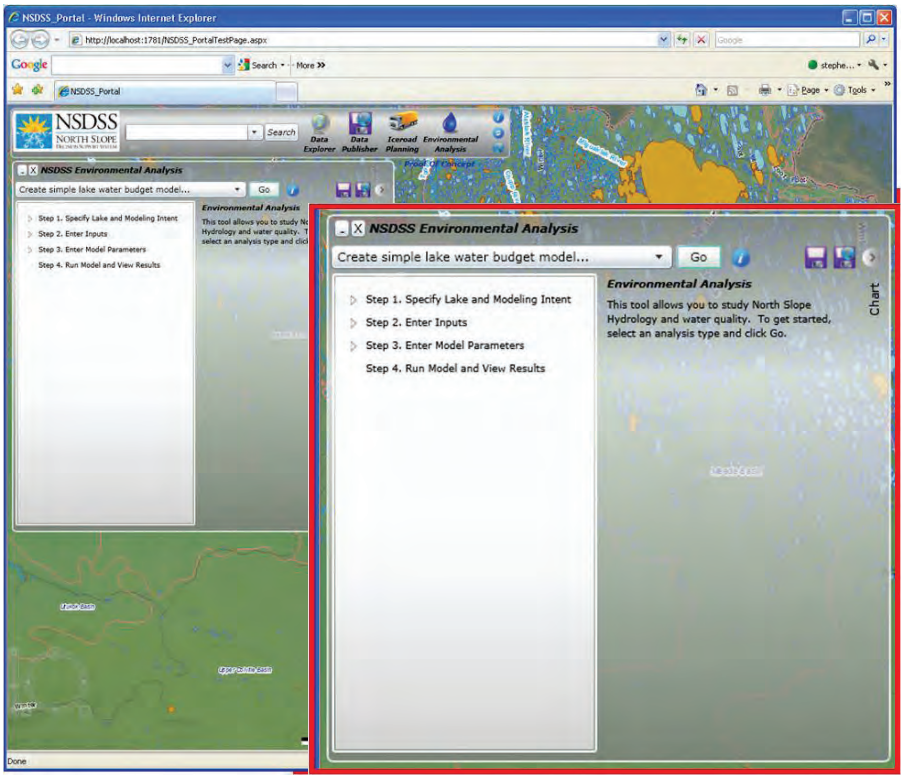

Figure 2. A newly created simple water budget model. To create the model, follow the 4-step workflow.

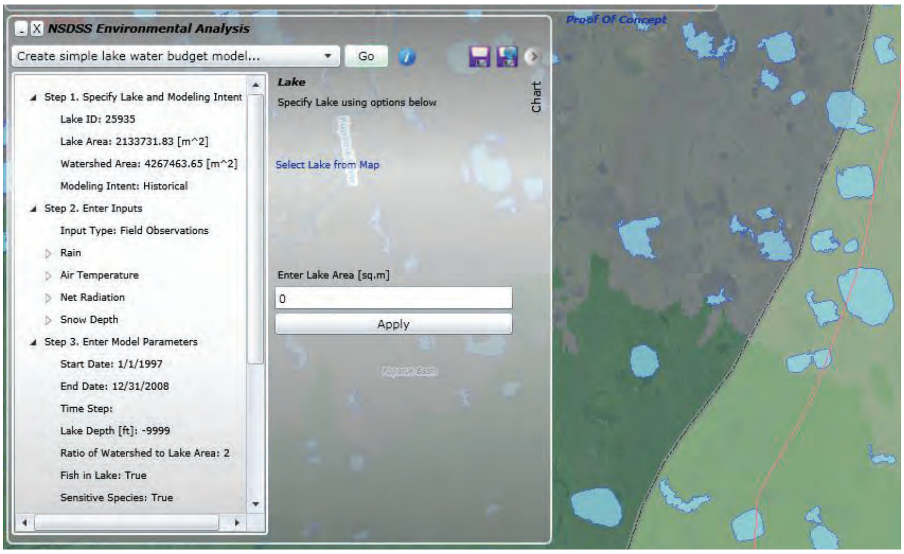

Figure 3. Lake 25935 has been selected. Its area and ID are populated in the tree automatically, and the watershed area is calculated based on the lake area. The lake to watershed area is specified in the model parameters section of the tree.

Step 7. Find the Inputs for the models. Inputs include the time series of precipitation, temperature, net radiation, and snow depth that are required for the water budget model. In the case of historical analysis, a field site is required.

In the case of forecasts, a field site is required for localizing and precipitation, temperature, and net radiation is required from each of the 12 GCM/Scenario runs.

To search for appropriate input data, click on the "Step 2. Enter Inputs" node and then click the "Suggest Model Inputs" option. This will start the data search process. A grid will appear that shows the suggested data including GCM data and the closest field data site with sufficiently long time series of precipitation, temperature, and net radiation.

The search process uses the location of the lake to find the GCM grid cells that overlay the lake. It also searches for the field site that is nearest and contains long time series of precipitation, temperature, net radiation, and snow depth, as these data are required in the localizing process. Each site within a large radius is ranked according to its distance from the lake and the length of the record of data it contains. The search chooses the site with the highest rank.

Figure 4 shows the grid when it is completed. Note that the grid contains 1) each of the GCM/ Scenario pairs, 2) the control run for each GCM, which are used to localize the data, and 3) the local field site. The grid also lists in its columns the variables that are to be collected, including precipitation (P), temperature (T), and net radiation (Ra). Latitudinal and Longitudinal wind speed (U,V) are also listed, but are not collected under the current formulation of the water budget. These variables will be required when more advanced estimates of sublimation are used in the future.

Step 8. The next step is to collect the data. Click the Collect Model Input Data option to collect all the data specified. The NSDSS will then contact each of the databases that contain the specified data and collect the time series one by one. This process may take a few minutes. As the process continues, the check boxes in the grid will become checked to indicate the data has been downloaded. When the process is done, the NSDSS will

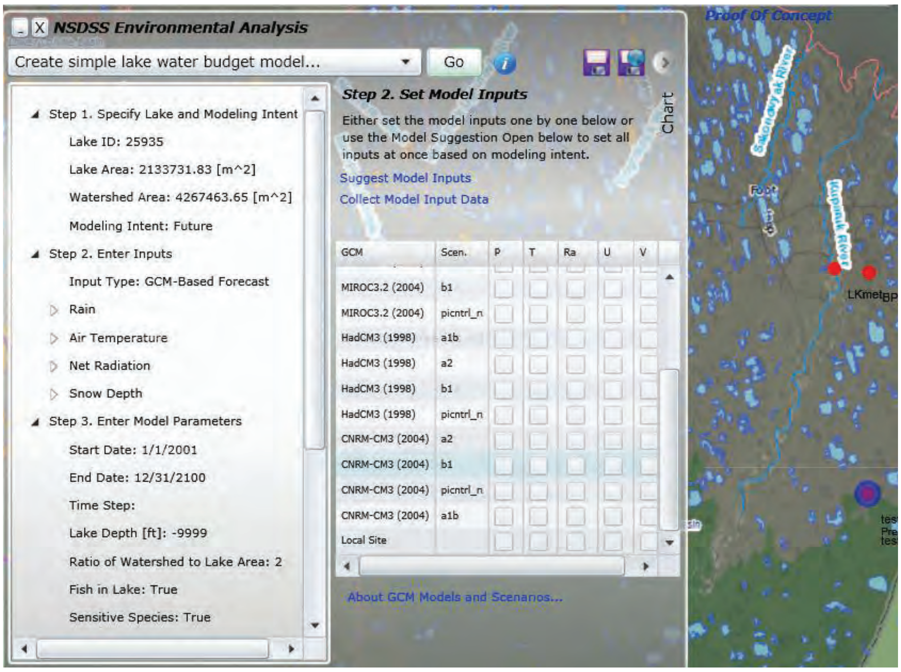

Figure 4. The suggest Model Inputs option suggests all the GCM/Scenarios pairs to collect precipitation (P), temperature (T), and net radiation (Ra) for. The suggest model option also finds the field data site closest to the lake that contains long records of these variables for use in the localization process. Note that there is a helpful link at the bottom of the tool – About GCM Models and Scenarios – that will provide information on how IPCC model scenarios are created and what they mean.

ask if you'd like to localize the time series. Select yes. When complete, a window will pop up that says Data Collected.

Step 9. At this point, it is a good idea to save the model to the local cache to ensure that the downloaded data is available on the next model run. Once the data has been downloaded to the local cache, it will be loaded quickly each time you open and run the model. This will save significant time. To save the model, click the left disk icon at the top of the tool. Give the model a name and click save. A window will pop up that says the data has been saved to the local cache. If you have not used NSDSS before, your internet browser may ask you to verify you want to save data to your local drive as a precautionary security measure. Select yes.

Step 10. Once the data has been collected, you can view it in the chart. Click on each variable within the tree (eg. Rain, Air Temperature, etc). Click on the "Chart all data" option. The chart will open to the right of the grid and will show the ensemble of GCM data. **Figure 5** shows the precipitation and temperature data for this example. Note that there is generally an upward trend in both across all the GCMs, but there is a wide range in estimates.

Step 11. The next step is to set the model parameters. You can change any parameter that is user specifiable by clicking on it in the tree and then entering the new value.

The model parameters include:

- Start and End Date for the analysis (default is 1/1/2001 – 12/31/2100 for forecast analysis)
- Lake Depth (default = -9999ft, so this must be set)
- Ratio of Watershed to Lake Area (default = 2)

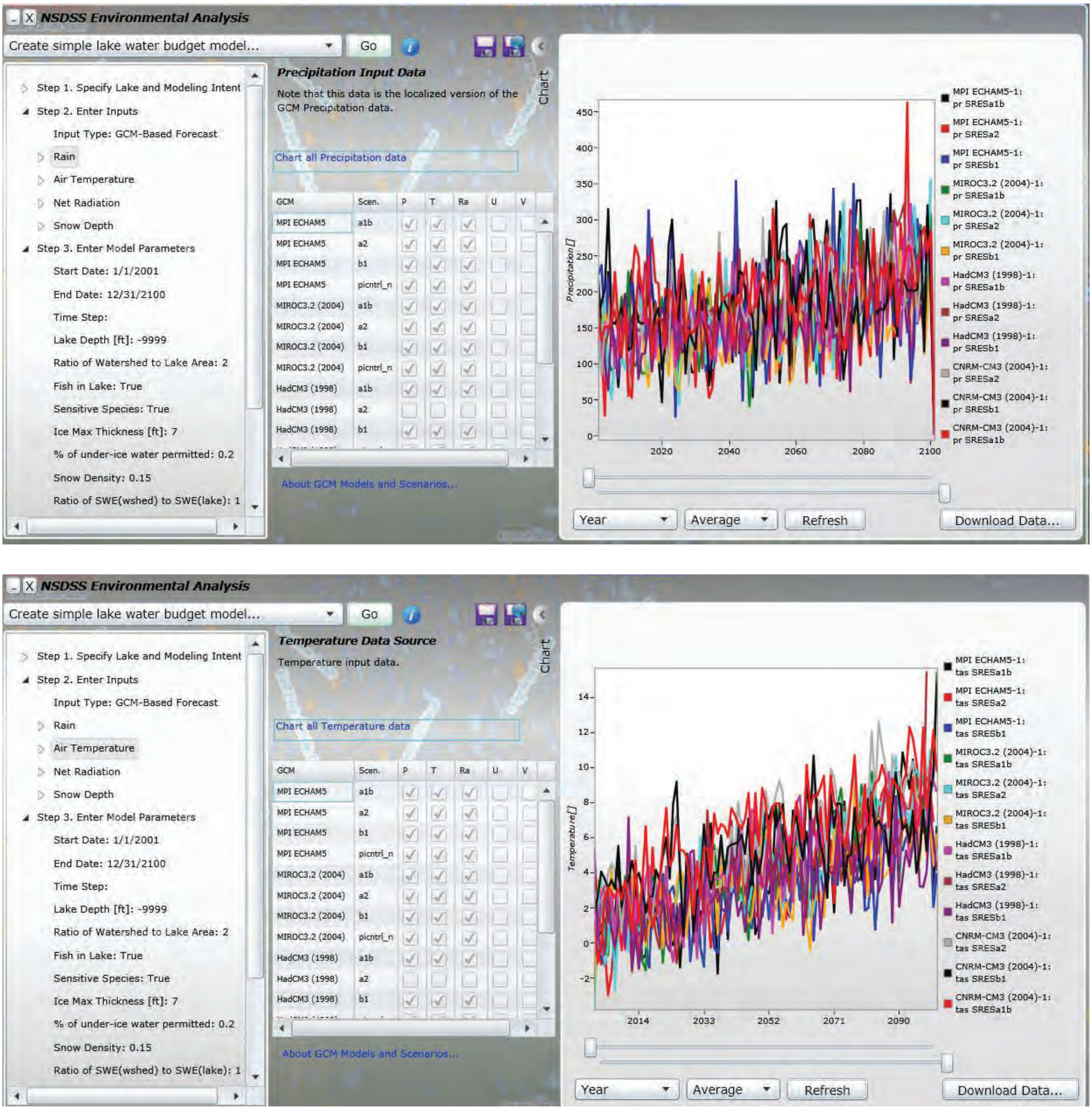

Figure 5. The upper panel shows the precipitation data (mm) for the grid cells overlying the lake for each of the GCM/Scenario pairs. The lower panel shows the temperature data (deg. C). There is generally an upward trend in both, though there is a wide range in estimates across the GCM/Scenarios.

- Fish in Lake (true or false)
- Sensitive Species (true or false)
- Ice Max Thickness (set through the decision tree describe above)
- % of under-ice water permitted (set through the decision tree described above)
- Snow Density (default is 0.15)
- Ratio of SWE(wshed) to SWE(lake) (default is 1, currently not taken into account in model)
- Spring Thaw Runoff Coefficient (default is 0.9)
- Summer Runoff Coefficient (default is 0.4)
- Priestly-Taylor Alpha (spring) (default is 1.26)
- Priestly-Taylor Alpha (summer) (default is 1.26)

Step 12. To run the model, click on the Step 4 node in the tree. Click the Run Model option and the model will take a few seconds to run. The ensemble of lake storage time series is presented as the results. **Figure 6** shows the result. Note that for the most part the lake storage is forecast as having an upward trend, similar to the precipitation and temperature series. The Hadley-based models show an unexpected "W" pattern over the time horizon, which requires further investigation.

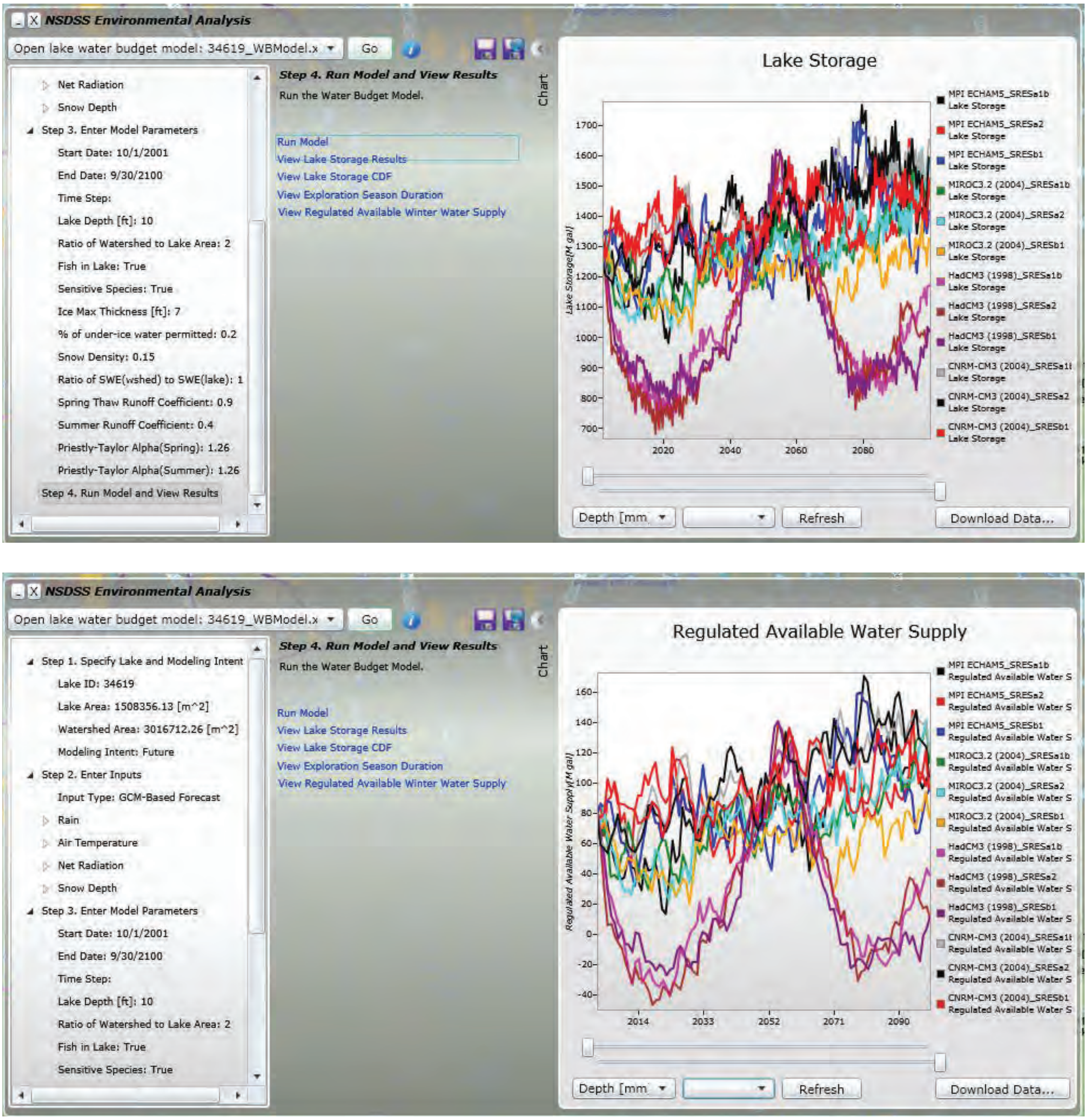

Figure 6. A water budget model result. The top panel shows lake storage in gallons over the 2001-2100 forecast time horizon. The lower panel shows the regulated available water supply for use in ice road construction. There is general upward trend in lake storage. The Hadley models produce a curious "W" shaped set of lake storage time series, which requires further investigation.

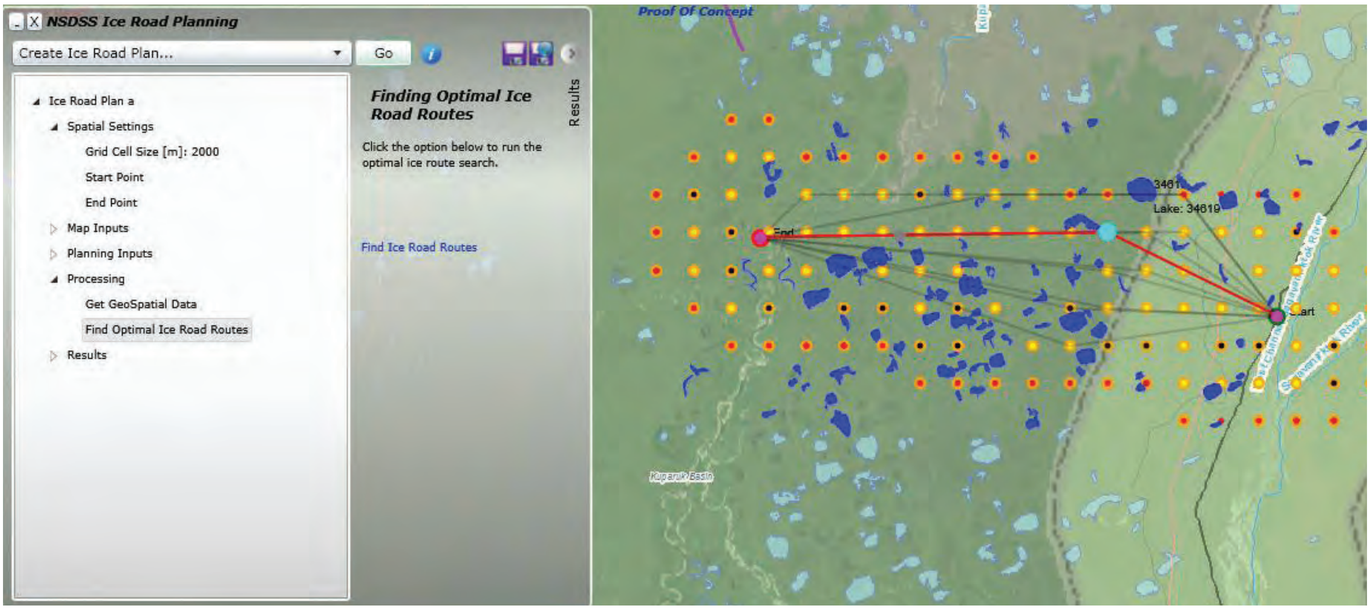

Figure 7. An ice road planning result. The dots represent the search domain that the NSDSS uses to seek optimal ice road alignments. The multiple gray lines represent the top ten best routes found in terms of stakeholder objectives. Lake 34619 has been selected as the lake to use as the water source. The red line is the shortest route that also minimizes haul distance – the distance between the route and lake water source.

You can also view several other model results using the options under the Run Model node. These include:

- View Lake Storage Results – this is the same as the result provided when you run the model.
- View Lake Storage CDF – this converts the lake storage time series to cumulative distribution functions.
- View Exploration Season Duration – this provides an estimate of the duration of the exploration season – the time from freeze up to spring thaw in months – for each of the GCM temperature time series. Due to the upward trend in temperature, the duration shortens over the forecast time horizon.
- View Regulated Available Winter Water Supply – this provides the Extraction term time series for all of the GCM/Scenario based models (see lower panel of **Figure 6**).

Risk Assessment

To understand the risk of insufficient water for ice road construction, the NSDSS user can create an ice road plan using the ice road planning module and then specify the lake as a water source for the ice road. The ice road planning tool will then use the water budget model created to assess the risk of running out of water. **Figure 7** shows an example of an ice road plan result. The dots represent the search domain that the NSDSS uses to seek optimal ice road alignments. The multiple gray lines show possible routes that the road can take from the start point to the end point. Lake 34619 has been selected as the lake to use as the water source. The red line is the shortest route that also minimizes haul distance – the distance between the route and the lake water source. An optimal route seeks to minimize both of these distances.

Figure 8 shows a risk assessment for the red line route in **Figure 7**. The black line represents the amount of water required to build the route. The regulated available water supply (see **Figure 5**, lower panel) is used to derive the curves. To create these curves, the ensemble is converted to time series of 1) 5% or minimum value of all time series in the ensemble in each year, 2) median value of all time series in the ensemble, and 3) 95% or maximum value of all time series in the ensemble. These three time series are then converted to probability of exceedance curves as shown in the figure. The end result is that we can see that if the 5% value occurs (red line) that we can expect we will have insufficient water approximately 70% of the time. That is, the red line is below the black line approximately 70% of the time. Of course, the red line represents a very dry condition as predicted by the GCMs, and is unlikely. Conversely, if the median GCM-projected condition occurs, we can expect that there will be insufficient water approximately 10% of the time. Finally, if the 95% condition occurs (a very wet condition), we can expect that in any given year there will be sufficient water to build the road.

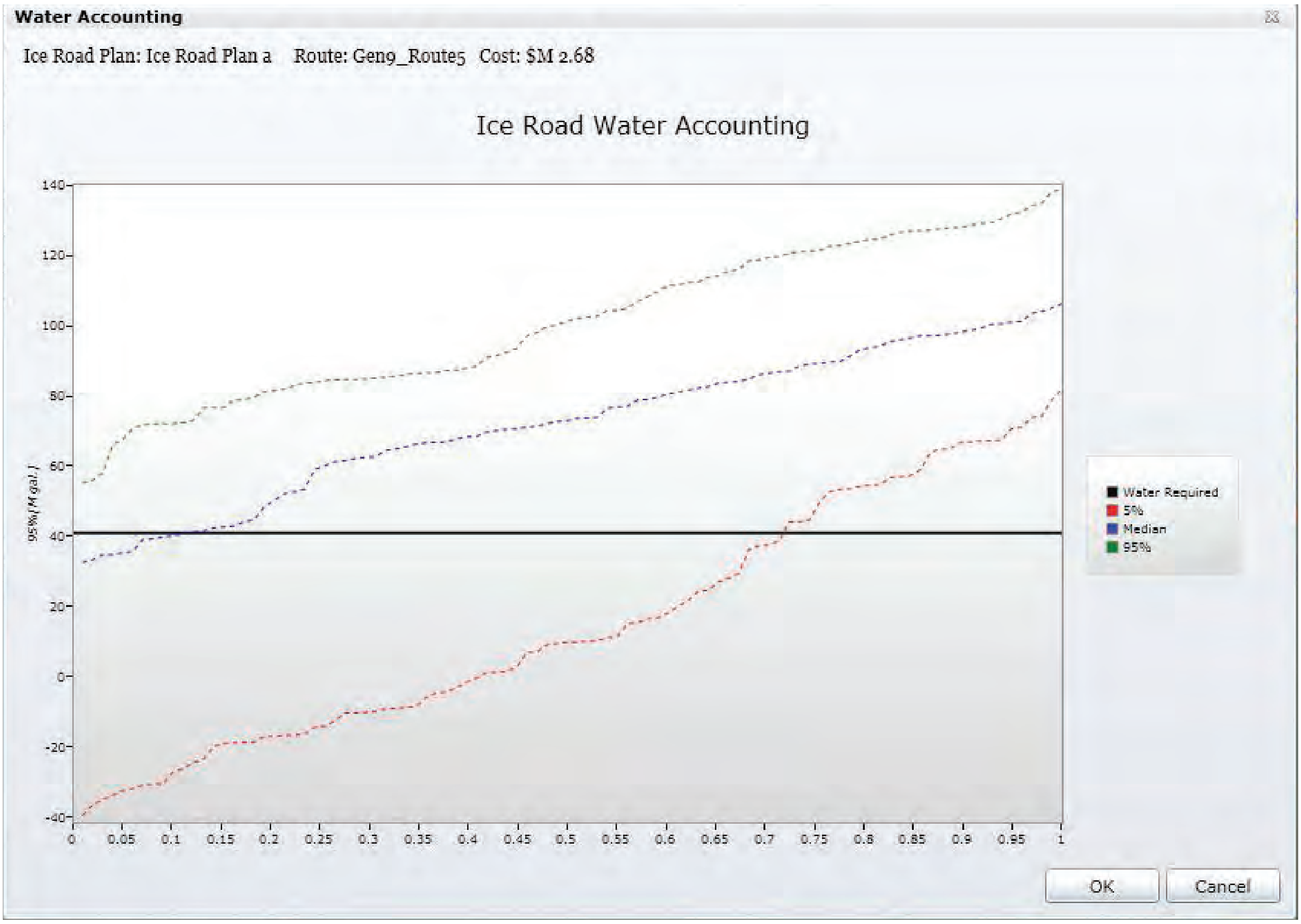

Figure 8. Risk Assessment Result. The black line represents the amount of water required to build the route selected in **Figure 7**. The regulated available water supply (see **Figure 5**, lower panel) is used to derive the curves shown above. To create these curves, the ensemble is converted to time series of 1) 5% or minimum value of all time series in the ensemble, 2) median value of all time series in the ensemble, and 3) 95% or maximum value of all time series in the ensemble. These three time series are then converted to probability of exceedance curves as shown in the figure above. The end result is that we can see that if the 5% value occurs (red line) that we can expect we will have insufficient water approximately 70% of the time. That is, the red line is below the black line approximately 70% of the time. Of course, the red line represents a very dry condition as predicted by the GCMs, and is unlikely. Conversely, if the median GCM-projected condition occurs, we can expect that there will be insufficient water approximately 10% of the time.

John Hilliard

Senior Design and Project Manager

Rail

Atkins

Orient Way carriage sidings: Design and construction

Abstract

In order to release land required at Thornton Fields for the 2012 Olympic and Paralympic Games, it was necessary to provide new railway carriage stabling sidings to accommodate the displaced trains which provide services to London Liverpool Street Station.

The Orient Way carriage sidings design brief was to construct a new carriage siding facility comprising twelve stabling sidings, each of which would accommodate a 12-car train. For this proposal the demolition of the existing maintenance shed, locomotive re-fuelling and storage equipment was required.

The site is bounded by Orient Way, the Up Lea Valley Line and the Eurostar Temple Mills Depot, and has rail connections to the Up Lea Valley Line via the Temple Mills Loop at both Ruckholt Road Junction and Temple Mills West Junction. All interfaces presented interesting engineering challenges which were met along the way.

There was an existing depot facility at the site originally built for rail freight operator EWS in 2001, which had been decommissioned and left to waste. The design layout was built using the large amounts of serviceable track already present at the Orient Way site.

Additionally the client requested that the project should be put forward for the Civil Engineering Environmental Quality and Assessment scheme (CEEQUAL) whole project award.

This paper explains some of the engineering challenges faced when tackling a tight programme, whilst at all times keeping the environment at the top of the agenda, and outlines methods utilised to overcome some challenging multi discipline conflicts along the way.

Existing conditions

Thornton Fields sidings were to be decommissioned to make way for the new aquatics centre which would play host to the water based events of the 2012 Olympic and Paralympic Games.

The site for the new sidings at Orient Way was chosen due to its proximity to Thornton Fields sidings, and because the site was already within railway land boundaries and allowed excellent re-use of available space. The site had previously been used by the former rail freight operator EWS (now DB Shenker) to service their fleet of locomotives, but as the current class 66 fleet requires less maintenance than its predecessors, the site was no longer required.

The site lay on an almost flat gradient and had two existing connections onto the running lines to the Up Lea Valley Line via the Temple Mills Loop at both Ruckholt Road Junction and Temple Mills West Junction. The intention was to create a new facility for twelve 12-carriage trains for stabling between peak traffic periods with only cleaning and water re-filling services required for the trains.

Design discipline brief

The disciplines covered by the commission were signalling, electrical and plant, civil engineering, telecommunications, permanent way, overhead line electrification, highways and architectural design.

One of the main constraints of the commission was to keep the design levels as close to the existing as possible to minimise ground re-profiling and cut and fill which would have required many road haulage journeys. The overhead line electrification serving the sidings was also required to tie in with the emergency exit on to Temple Mills Loop.

The work would be carried out over an 18 month period following amalgamation of the design and construction programme, and the proposed date for completion of construction was 30th June 2008, which provided a very challenging target.

The design speeds for the facility are 15mph and 5mph in reception road and berthing sidings respectively. Consequently the design of the plain line chosen was serviceable 113 lb flat bottomed rail (code113A) laid on recovered serviceable concrete sleepers and for turnouts serviceable vertical full section type "B" switches with 1 in 8 crossings laid on serviceable timber bearers.

Stakeholder management

The project required careful stakeholder management to balance various competing requirements: the Client naturally wanted to keep costs of design and construction to a minimum; the eventual recipient of the facility wanted a facility which made life easier and was installed to the latest specifications using the best available technologies; Eurostar, whose depot is adjacent to Orient Way, also required future provisional use of the facility and a revised entrance as part of the commission; and the rail operator also had a sign-off authority for the elements of the commission that affected the operational running lines. Network Rail also had to review and approve certain parts of the project, especially at the operational interface.

Design programming required a significant allowance for stakeholder reviews; a 20 day turnaround was planned to accommodate all parties. All comments and points which required modification could then be carried out in a single exercise and all comments documented and resolved before moving onto the next project stage.

Additionally the client requested that the project should be put forward for the Civil Engineering Environmental Quality and Assessment scheme (CEEQUAL). The award required evidence based submissions to a laid down criteria from the CEEQUAL board. The award recognises environmental work over and above that of a normal CDM compliant project, and the submission is subjected to an independent assessor whose role is to ensure evidence is in place to back up any submission. The project was awarded an 87.2% 'excellent' whole project award which relates to the client, designer and construction contractor as a team.

Design issues

Site preparation

At outline design stage, the new sidings footprint was overlaid onto mapping of the existing site, identifying a requirement to demolish the existing building and some of an existing highway already built on site. It was proposed that, instead of taking the spoil away from site, anywhere that did not carry an existing railway formation should be provided with the waste from the demolition following crushing to type 1a material, meaning that the site waste was re-used to create a compacted rail formation for the full width of the 12 sidings.

Overhead electrified lines

Overhead electrified lines were required for the traction power for the sidings. At an early stage of design an issue arose regarding an existing footbridge, shown in **Figure 1**, which was used to carry pedestrians from Orient Way to the adjacent golf course and pathways. The issue was one of structure gauge and the clearances required for the overhead lines. There were three main options for the bridge; build a new bridge to a compliant height: create small ground anchors to support the end of the overhead line sections; or provide a bespoke anchor structure which would act at overhead line height and would mean the lines could be curtailed just short of the existing structure which would save the structure and create more space to the north of the site. The decision was made to curtail the lines and design a bespoke anchor structure as shown in **Figure 2**.

Each section of the overhead line electrification (OHLE) was provided with an individual isolating switch, which meant that individual sections could be isolated for ease of maintenance and access without disrupting the entire sidings site. This means that the sidings can be maintained regularly but the overall facility would only ever lose less than 10% of its available stabling.

Permanent Way

Figure 3 shows an outline design which had been carried out by other consultants, which utilised all of the available space along the site's entire length but continued under the existing footbridge. As mentioned for the OHLE the bridge would have been required to be demolished as part of that scheme. However, as shown in **Figure 4**, the Atkins' designers were able to design an adequate length siding in front of the bridge and save the bridge from demolition. The designers had to provide sliding friction buffer stops, shown in **Figure 2**, to protect the area behind the bridge and also to protect the bridge pier.

The topographical survey was carried out for the entire site, including surface and buried services, Network Rail infrastructure, site boundaries, fencing, signalling, existing road alignments etc. It was soon established that with some minor earthworks and some re-use of crushed aggregate, there was an opportunity to design the vertical alignment at a perfect 0% gradient. There are many benefits to the track having a 0% grade: the track, walkways and all associated infrastructure are very easy to install - a laser level can be set up to a datum and everything can be built from that single datum. There is no risk of trains rolling back due to gradient effects. Shunters can view the train right down to the buffers despite the platforms. Walkways are safer as the track and walkways can remain at a single level throughout.

The design was altered from the original concept for horizontal alignment as well; the original design used curves to facilitate all of the rolling stock units and would have meant trains at different stagger, and would also have meant excessive track lowering under the footbridge to allow for OHLE or indeed a new bridge. The original design would have also resulted in issues with sighting signals and for staff to be seen by other trains, as the curved effects of the track when coupled with trains already stabled in adjacent sidings would have blocked the vision of the trains as they entered the sidings.

Materials were at the forefront of the design: the site chosen had previously been a maintenance facility for older class locomotives and even had a bogie drop facility, however as the

Figure 1. Existing footbridge

Figure 2. Existing footbridge, bespoke anchor structure and friction Buffer stops

former EWS had commissioned a new fleet of class 66 engineering locomotives, the need for servicing and maintenance was reduced by the longer servicing interval. The site had already supported a train facility so there was opportunity for the re-use of much of the existing material.

Initially a survey was carried out on site for the Permanent Way (PWay) materials: the original site had ordered some switch and crossing units made from serviceable rail material but utilising new timber bearers. The materials were all assessed as fit for purpose and saved tens of thousands of pounds in PWay component costs. The units were railway standard units which were designed using straight through route geometry, example shown in **Figure 5**. The straight geometry meant that the layouts could be created using a 'fan' system so each point led to the next point, thus giving further opportunity to optimise the available space and ensure that spatial provision could be given to all engineering disciplines at an early stage.

There were also four existing long stabling sidings on site; these were made up of predominantly serviceable rail on serviceable concrete sleepers. The sleepers and rail were 18.3m (60') lengths and were joined by fishplates. This meant that the track could be taken apart in sections at the construction phase and stacked for re-use at the re-laying stage.

As the track has been designed to save the footbridge another challenge arose. The sidings would terminate close to the bridge pier and the pier would require protection. Various options were considered including a trap at the end of the sidings close to the pier, a large bespoke impact wall to protect the pier, or sliding arresting buffer stops which would slow the trains over a distance minimising the chance of derailment. The decision was made to provide sliding arresting buffer stops for all of the sidings as this would mean the area behind the buffer stops could be used for almost any purpose and would not limit site expansion of the facility.

Telecoms

The telecoms provision was a simple system which linked the internal facility with the rail network and utilised a telephone concentrator and train describer to integrate into the wider rail network.

Figure 3. Original concept design

Figure 4. Final re-design position

Civil engineering

The civil engineering team designed bespoke support to the facility infrastructure (walkways, shunters cabin foundations etc.), given that the site had poor ground conditions, as it was found during geotechnical investigations that a soft alluvial layer was present on site. Therefore the infrastructure had to be supported by oversize foundations to avoid unacceptable amounts of initial

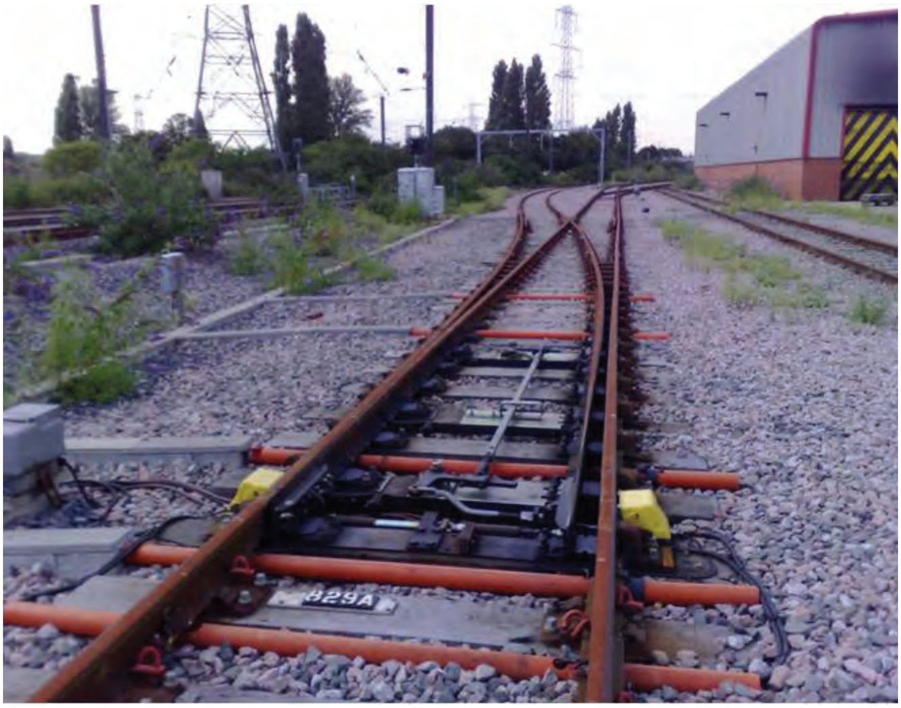

Figure 5. Existing connection to the operational railway

Figure 6. 9 No. duct large UTX

settlement. The civils team also integrated the power and cabling requirements into the foundations of the shunters cabin. The foundations were designed with the equivalent of troughing route runs through them so cable management was easily concealed but also easily accessible.

Figure 6 shows the custom under track crossing (UTX) pits for mechanical and electrical (M&E), signalling and telecoms cable management.

Mechanical and electrical

There was a requirement to provide a new mains supply to the site itself and local distribution to the facility, shown in **Figure 7**. This is normally a fairly straightforward operation, however, adjacent to the site was the newly constructed Eurostar depot. Eurostar has a bespoke electrification system which has a fault current of 12kA. Network Rail OHLE has a maximum fault current of 6kA therefore each facility had a different fault condition which could create a touch potential problem. If a live cable or live item from Eurostar were to leak onto the NR system, it would possibly melt the bonding and damage the infrastructure; similarly if someone were to come into contact with the 12ka system and the 6ka system the difference could potentially cause electrocution. The remedy was that anywhere there was a touch potential, i.e. anywhere that metallic objects at the boundary could be 2m or less from each other, the boundary would require a non conductive screening. This screening was provided to completely cover the fencing removing any touch potential in that area.

Figure 7. New power distribution case

Highways

As the project would require a car park and staff facility on site, there was a requirement for an entrance off the highway. The site already had an old sliding entrance gate, but the width of the new sidings had meant that part of the existing highway had been removed to make way for siding 12.

This meant a new access had to be created and the most logical option was to use the adjacent Eurostar entrance road and create a new gate into the carriage sidings. Allowances had to be made for car parking and refuse trucks collection and turning circle radius. The area had to be provided with toughened crash barriers at the turning circle as the OHLE bare feeder section was close to the turning circle and any overrun of vehicle manoeuvres may have led to electrical contact.

Signalling and shunting

A simple signalling system was designed for the sidings where two trains could be waiting to enter the sidings at peak periods on a reception line while the shunter could set the route; this was a requirement as there was a risk with a single section that if a second train wanted to access the sidings it would have either been standing across the junction, which would delay the trains on the network, or would be held at a red aspect, again delaying the operational services.

Once the trains came into the facility the route would be manually set by the shunter, however, to make passive provision for fully worked sidings from a control panel, all points were fitted with trailable motors (i.e. suitable to be run through from one siding when the switch blades were set for the alternative route). A bespoke switch normal/reverse button was designed so the shunter had no need to use strenuous point's handles to set the route. This meant that if at any time in the future the facility manager wanted to upgrade the facility, the infrastructure would be in place.

An upgradable panel was provided in the shunters cabin with the signal sections of the reception line shown, and each siding represented on a sliding board to keep track of the stock in the sidings. There had been an ergonomics assessment for the shunters cabin carried out involving human factors experts and the train operators, as a result the windows were designed to be in a specific location to avoid the shunter having to get up and down to watch the trains all of the way down to the buffer stops, whilst ensuring that solar gains were not pushed up above the values specified in the building regulations.

The connection into and out of the sidings was via two links to the main line. There were already connections for the track, but as the classification of the sidings was changing and as new signal sections had been introduced, there was a need to carry out additional works to ensure the safety of the main line signalling was not compromised. A signalling assessment was carried out between the designers, train operators and Network Rail to assess the proposed layout and to indicate if there was any increased risk brought about by the facility which would mean that design standard limits were exceeded. The layout was proved to be safe and, therefore, only a staging strategy had to be produced to ensure the signal changes were completed without any unnecessary disruption to the network.

Programme

Originally the programme was planned around an opening date of the 30th June 2008. However the construction company were challenged with early delivery date of 30th April 2008. By working with the designer and subcontractors on the procurements and approvals of the key milestones, the site achieved its optimistic target leaving the cosmetics and snagging activities to a series of possessions.

CEEQUAL

Carbon critical design demands that we aim to reduce carbon from the outset and at every stage of a project. This philosophy led to a prestigious award from the Civil Engineering Environmental Quality Assessment and Awards Scheme (CEEQUAL) for the Orient Way carriage siding project. The award was presented to the project team (client, designer and construction contractor) at the ICE London and National Dinner at the Park Plaza Riverbank Hotel in London on Thursday 2nd April 2009.

At first glance the new rail depot at Orient Way in East London does not look like an obvious candidate for sustainable design. CEEQUAL, which is a well-respected assessment and awards scheme for improving sustainability in civil engineering and public realm projects, takes into account a vast range of environmental performance criteria. These include meeting time and budget targets and sending out letters to local residents to advise of potential noise disruption. It is an evidence based scheme and takes various factors into account, such as designing the layout to maximise the re-use of existing track and ballast.

The Orient Way site had a recycling rate of 99 per cent of the demolition and site clearance waste. This included:

- 4,000 tonnes of crushed concrete, of which 1,000 tonnes was reused on site and 3,000 tonnes was reused off site;
- 620 tonnes of tarmac;
- 180 tonnes of steel;
- 20,000 tonnes of previous site ballast was screened and reused;
- 3,250 yards of track was reused.

Figure 8. Bio-diverse green roof after planting

New offices at the site for train drivers and other staff were designed with a green roof, shown in **Figure 8**. This helps the offices to fit in with the natural surroundings and support the natural environment. The wind turbine shown in **Figure 9** helps to support the electricity needs of the offices. The team put environmental considerations at the top of the agenda – resulting in the excellent 87.2 percent score from the CEEQUAL assessors. It also demonstrated that sustainable design doesn't have to relate to high profile buildings – it relates to absolutely everything.

Figure 9. New wind turbine and high mast lighting column

Conclusion

The new carriage stabling sidings at Orient Way were designed and constructed to a very demanding timescale to meet severe operational and environmental constraints and, as a result, made a valuable contribution to the London 2012 project.

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Structural analysis of composite aerofoils using aero-and inertial-elastic tailoring

Abstract

The reduction of structural weight in the design of composite structures is of prime importance because it has a dual benefit of improved overall performance and efficiency and reduced manufacturing cost. An Aerofoil Structural Model (ASM) has been developed to capture the elastic response of a generic composite aerofoil under aerodynamic and inertial forces. The intent is to generate a structural modelling and rapid sizing capability for aircraft wings, fan and propeller blades and wind turbine blades to achieve optimal utilisation of the composite material. For a given blade shape, the stiffness and inertial properties of the blade are tailored by altering the structure configuration, material properties and structural and non-structural mass distribution.

The ASM uses the ABAQUS finite element code to simulate the aerofoil. The calculated aerodynamic and inertial loads are applied to the model at span-wise locations using user-defined subroutines. Internal structural strains arising from these loads are extracted and, in conjunction with established strength and stiffness criteria contained within sizing routines, used to derive improvements to the structural configuration of the aerofoil, thereby assisting weight minimisation. The use of ABAQUS for the ASM allows the model to simulate quasi-static, non-linear and dynamic load histories.

The ASM seeks to analyse the impact of configurational changes on the structural performance of generic aerofoils. Such changes might include the distribution of non-structural and structural mass, the structural configuration, such as internal rib pitch, or material configuration, such as the deployment of un-balanced laminates to achieve aerofoil bend-twist coupling. Ultimately the ASM will be used to develop an initial low weight structural configuration for a generic blade. This development exploits the multi-disciplinary use of carbon composites, deploying them in a manner that more fully utilises their potential.

Nomenclature

a	Rib Pitch
Li	Length of superstringer at bay i
Ai,j,	i=1-3,j=1-6 In-plane stiffness matrix
n	Number of bays along a superstringer
ai,j,	i=1-3,j=1-6 In-plane compliance matrix
Nbb	Critical bending buckling load
Aw	Web area
Nx	Running load on superstringer (span-wise)
b	Stringer Pitch
N_i^{cr}	(i = x, y, xy, w) Critical buckling load
bi	(i=s,f) (s=skin, f=flange) Width
H(i)	(i=1,3) Laminate transverse shear flexibility stiffness
RF_{GS}	Global Shear Buckling RF
H_{55}	Transverse shear flexibility
Ny	Running load on super-stringer (chord-wise)
C_L	Coefficient of Lift
Nxy	Shear load on super-stringer
C_M	Coefficient of Moment
N_{skin}	Running Load in skin
$D_{i,}$	i=1-3,j=1-6 Bending stiffness matrix
P_{cr}	Critical buckling load of a column accounting for Transverse Shear loading
$d_{i,j}$	i=1-3,j=1-6 Bending compliance matrix
P_e	Euler buckling load
Dc	Bending stiffness of the super-stringer element per unit width
H_{55}	Transverse shear flexibility
t_i	(i=cov,web,cap) Plate Thickness
I	Second moment of Area
ti	(i=s,f,w) (s=skin, f=flange and w=web) Thickness
RF	Reserve Factor
E_i	(i=cov,web,cap,1) Membrane stiffness, Membrane stiffness for single ply (1=fibre direction)
RF_{ns}	RF normal-shear buckling
G_{12}	Shear modulus for single ply
RF_{sc}	RF shear and compression
Gw, Gi,j	(i=1,2 j=2,3) Shear modulus for web, shear modulus for single play (1=fibre direction, 2,3=transverse direction)
RF_{GLS}	Global Longitudinal-Shear Buckling RF
h	Height of the web
RF_{GL}	Global Longitudinal Buckling RF
Ks	Shear Factor
c	Rib pitch/spar height ratio
Ki	(i = x, xy) Non dimensional buckling coefficient
m	number of half wavelengths

Introduction

The reduction of weight in laminated composite structures is of prime importance due to the dual benefits of improved machine performance and reduced manufacturing costs. For example, composites are widely used in the blades of the increasingly ubiquitous wind turbines, and such blades are increasing in size to extract more power from the wind. Manufacture of the root section of these aerofoils dominates the overall fabrication costs, so reduction in weight in this region, perhaps achieved via aero-elastic tailoring through alleviation of the root bending moment is highly beneficial.

The optimisation of the structural configuration of a general composite aerofoil structure is a goal in many industries. However, as large aerofoil structures have traditionally utilised metals, experience in the configurations best suited to composite construction is very limited. The prospect of a protracted evolution of composite structures towards the best configuration is unappealing, so a means to accelerate this development is sought. The aim of the Aerofoil Structural Model (ASM), described in this paper, is to provide a tool that allows rapid structural assessment of generic composite aerofoil structures. This capability facilitates re-iterative assessment of a range of structural configurations, allowing investigation of a wide design space. The ability to address generic aerofoils is enabled by the extended scope of geometry and loading that the ASM permits, so the assessment of fixed aircraft wings, wind turbine blades, or gas turbine fans and propellers is feasible. The ultimate aim is to achieve configurational optimisation, so that technology such as aero-elastic tailoring can be considered at the design stage. This paper concentrates on the aero-elastic tailoring aspect of structure configuration optimisation.

For a commercial airliner, aero-elastic tailoring would be used to reduce air-induced structural loading – especially in weight-sensitive locations within the structure. For example, reduction of the bending moment at the root of the wing would allow reduction in structure weight. Aero-elastic tailoring is the subject of much research[1-4], including coupling with structural optimisation[5-7]. However, it is not currently fully exploited in commercial aircraft design, partly due to the computational expense of full aero-elastic analysis. Although the ASM is aimed at a more general optimisation of structure configuration, aero-elastic tailoring is an obvious subset of this general capability.

Aero-elastic tailoring can be achieved through the use of un-balanced lay-ups (an unequal number of +45° and -45° plies). By using un-balanced lay-ups in a composite wing panel, direct-shear coupling terms are introduced into the constitutive equations and these may be utilised to control the twist-bend coupling of the wing. For example, in the normal flight of a large civil transport, the upper surface of the wing is in compression and the lower surface in tension. If un-balanced composite panels are used in the wing skins, the direct-shear coupling terms can be used to cause the top skin to shear forward and the bottom skin to shear aft. This effect can be used to cause an aerofoil section to twist nose-down, reducing the local angle of attack and therefore reducing the lift generated at that section.

By tailoring the outboard section of the wing to respond in this way, the centre of lift of the wing can be moved inboard, reducing the root bending moment under gust or elevated 'g' conditions. There will be a reduction in strength, and so a small increase in weight, of un-balanced portions of the aerofoil. However, less material will be required in the heavy inboard section, yielding an overall weight saving.

The paper illustrates aero-elastic tailoring of a high aspect ratio aircraft wing with a simple example generated within the ASM environment.

Description of the ASM

The initial requirements for a sizing tool for a generic composite aerofoil demanded the following capabilities:

- Analyse a wide range of aerofoil structural configurations;
- Full, six degree of freedom (translational and rotational) loading;
- An ability to simulate static, steady state dynamic and transient dynamic load histories;
- Allow both linear and non-linear response;
- Perform rapid sizing iterations;
- Incorporate bespoke integrity assessment criteria;
- Allow simple modification to internal structural configuration (e.g. rib pitch).

To permit the full range of these requirements, the finite element code ABAQUS was selected as an environment in which to develop the ASM.

A shell finite element model is used to calculate the internal structure loads and strains arising from the impressed aerodynamic and inertial loads. A basic surface model is created with the surfaces simulating planes of composite structure. Initially these surfaces are populated with nominal thicknesses and material properties. Element mesh density is user-defined, as is non-structural mass distribution, boundary conditions and loading. Loading, however, is constrained only by the capabilities of ABAQUS and embraces aerodynamic and inertial components. As fuel (internal) and air (external) pressure can be significant for an aircraft wing, these components are also accommodated.

The ASM was conceived as having three basic construction types:

- A panel/stiffener configuration, like an aircraft wing;
- A similar arrangement but with sandwich construction covers;
- A blade with a solid core, similar to a propeller or gas turbine fan blade.

In practice, no development has yet been applied to the last two of these configurations.

The ASM consists of three modules: an input deck generation module; a loading module for impressed loading and a sizing module.

The input deck generation module uses aerofoil surface information, initial structural configuration, material data and loading conditions, to create an input deck for ABAQUS analysis. The module calculates the grid point co-ordinates and if appropriate, the idealisation of the cover material properties from a skin-stringer configuration to a representative shell element.

The loading module comprises two components: an inertial component and an aerodynamic component. The inertial component uses a mass matrix to represent the structural and non-structural mass distribution within the aerofoil. For an aircraft wing, the large inertias represented by the engines can both alleviate and exacerbate structural loads under gust loading. For a propeller or fan blade, the centrifugal forces generated by the structural mass often dominate the overall aerofoil loading. In both of these examples, the influence of mass distribution on the applied loads is apparent.

The aerodynamic loads are derived using thin aerofoil theory. A user defined subroutine updates the load calculation based on the nodal deflections throughout the loading history. For an aircraft wing, aerodynamic force through a gust will be time-variant. For a propeller blade, there will be far less variation of aerodynamic load with time. In both of these examples the influence of structure stiffness on the applied loads is apparent.

Finally a sizing module extracts the internal loads and strains to calculate reserve factors based on covers and spars strength and stability criteria. The ASM has been built using a modular approach, which enables additional sizing routines to be added or the loading modules to be adapted as required.

Input deck generation module

Generic meshing ability

To rapidly model configuration changes there is a requirement for the ASM to produce ABAQUS input decks for generic aerofoils, with different structural configurations and loading conditions. The surface of the aerofoil is divided into span-wise panels as shown in **Figure 1**. The surfaces are defined using Lagrange polynomial interpolation functions, a cubic function being used in the chord-wise direction, and a quadratic function in the span-wise direction. A typical mesh is shown in **Figure 1**. This approach is able to capture a large rate of twist, sweep and dihedral along the span of the aerofoil and many options for structural configuration, whilst maintaining a simple input and generic meshing capability.

The aerodynamic and inertial loads are applied as concentrated loads and moments to loading nodes located at span-wise intervals along the aerofoil. Distributed coupling elements are used to transfer the external loads to the wingbox in six degrees of freedom.

Material idealisation

The material properties are modelled using a general shell section within ABAQUS. The stiffness of the shell section is entered directly in the form

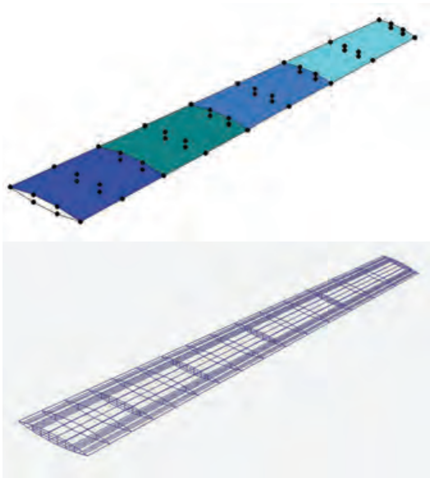

Figure 1. Surface Points and Typical Mesh

$\{N,M\}=[D]:\{\varepsilon,K\}$. This allows the in-plane running loads, N, and out-of-plane running moments, M, to be described in terms of the in-plane strains, ε and the cover curvature, K.

Aircraft wing upper and lower surfaces typically have a panel-stiffener (skin-stringer) construction, this combination being termed a "cover". To maintain a generic baseline model, the covers have been modelled using a surface mesh. If there are stringers in the cover construction, these have not been explicitly modelled. Instead, the stiffness and mass properties of the stiffeners are smeared over the surface elements, modifying the in-plane and out-of-plane span-wise stiffness of the panels. This approach allows a generic baseline model and a coarse grid mesh to be used, as the elements do not have to align with the stringer locations. The simplified mesh aids rapid sizing iterations.

For the cover material idealisation the components are assumed to be manufactured from symmetric composite lay-ups, so there is no membrane/bending coupling in the covers. The overall process for smearing the stiffness re-idealises the skin, stringer flanges and stringer webs as a single panel, the span-wise direct stiffness and panel shear stiffness being modified accordingly, and membrane properties calculated from the product of modulus and area/thickness.

For an aero-elastically tailored wing, there is coupling between the shear and direct membrane stiffness terms. Such coupling is assumed to exist only in the skin, the stringer attached flanges being uncoupled. Depending on the lay-up used in the attached flanges, the stiffeners could act together with the skin to enhance the direct-shear coupling. The effect is initially assumed to be negligible, and so the skin properties for the direct-shear coupling terms are used.

For cover bending properties a similar approach is taken, but using the product of stiffness and second moment of area. The main influence of the stringers is to increase the out-of-plane bending properties of the cover in the span-wise direction. Therefore only this bending term is smeared, the other terms of the material stiffness matrix being taken from the skin.

Loading module

The ABAQUS finite element code allows static or dynamic loads to be applied to the structural model. Within the ABAQUS solver the load history is divided into a series of time or load increments. Response of the model can be either linear or non-linear within each increment.

The nodal deflections are output and used to calculate the load applied at the next load step. By this means, the external load applied to the structure is dependent on the deflection of the aerofoil, and this capability is exploited in the inertial and aerodynamic loading components.

Load histories

A quasi-static analysis can be used to assess the response of the aerofoil to a steady state load, such as an aircraft in a steady 2.5'g' turn. A time-transient load history can be employed to investigate the response of an aerofoil to time-variant loading, such as the response of an aircraft to a gust. Such load cases are analysed in the "time domain". The ASM exploits direct time integration method within ABAQUS to obtain a time transient response for the aerofoil structure. An implicit method is employed to integrate the equations of motion. Whilst this is computationally more expensive than an explicit method, it provides an unconditionally stable solution.

A steady state dynamic analysis can be performed in the frequency domain to assess phenomena such as flutter, or the response of an aerofoil under cyclic loading. At present this capability has not been implemented within the ASM.

The user provides the initial and final condition of each input for the given load step (e.g. the initial and final vertical wind speed to represent a gust load). Options are available to describe the form of the parametric variation over time. The user-subroutines that define the aerodynamic and inertial loading calculate the current value of the input parameter at the current time increment before computing the external loads.

Aerodynamic method and assumptions

Consistent with the aim of rapid analysis times, the aerodynamic model was kept as simple as possible, whilst providing reasonably accurate results for sizing. Thin aerofoil theory has the advantage of being rapid to compute. However, as it is a 2-D inviscid solution, it does not consider many effects such as stall, tip loss, compressibility or turbulence. This will mean that for some applications, the aerodynamics will lose accuracy. It is assumed that, for future development, the theory may be supplemented by corrections to allow for these phenomena.

Aerofoil lift is calculated at each load or time step. The nodal positions are extracted and stream-wise planes are defined that contain the loading points, the airflow vector and the vertical axis, as shown in **Figure 2**.

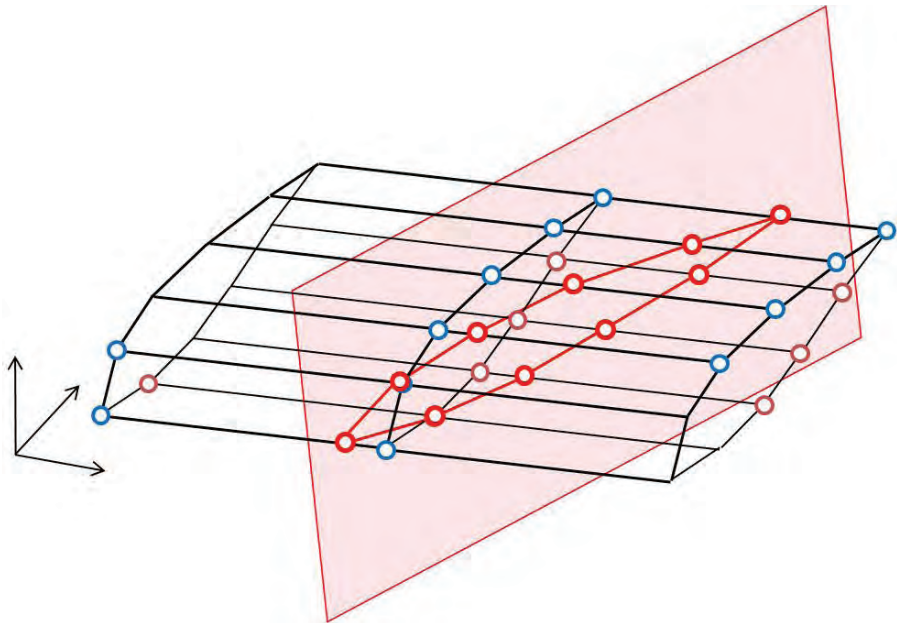

Figure 2. Section through Aerofoil to calculate Aerodynamic Loads

The mean camber line represented by the aerofoil section profile is calculated, and values for C_L and C_M at the quarter chord point are computed.

For analyses involving a rotating blade, the aerodynamic module is extended by applying Blade Element Momentum (BEM) theory, which is often used in wind turbine analysis. In order to apply BEM the axial force and torque are calculated from the aerodynamic forces for sections of the blade, and from conservation of axial and angular momentum for an annular stream tube. A set of equations are derived that can be solved iteratively for axial and angular induction factors. These induction factors are used to adjust the angle of incidence. This is an extension to the non-rotating case. [8]

Elliptical lift distribution

When designing aerofoils it is desirable to reduce the induced drag, experienced due to the shedding of tip vortices. The minimum amount of induced drag is obtained when an elliptical lift distribution is created along the span of the aerofoil. The ASM has been given the additional functionality of being able to enforce an elliptical lift distribution for a particular loading condition.

The aerodynamic subroutine optionally calculates the required amount of twist for various sections along the aerofoil that would achieve an elliptical lift distribution for a given set of user input parameters. The user supplies a total mass of the aircraft and a flow velocity for which an overall 1g load is obtained. The subroutine then calculates the required change in angle of attack and applies this to all subsequent load steps. The necessary angle adjustments at each section are provided to the user which can be later incorporated into the design through twist in the aerofoil.

Inertial module

The inertial relief is considered for structural mass and non-structural masses, which could include systems mass or mass of the fuel in an aircraft wing. System masses are represented as point masses, defined by a full mass matrix; this reduces the complexities of the model while maintaining an accurate representation of the global response. In order to represent the fuel, elements with appropriate mass but negligible stiffness are created from the bottom cover nodes. ABAQUS will internally describe the structural and fuel inertia with a mass matrix.

Steady state inertial loads are simulated using a *GRAV card applied to those mass matrices. If applicable, a *CENTRIF card will model the centrifugal loads from rotation. Time transient inertial loads are accounted for by the equation solver within ABAQUS.

Rapid sizing module

The final module of the ASM includes bespoke strength and sizing criteria for the aerofoil. The approach discussed focuses on an aerofoil of typical aircraft construction, comprised of stiffened panels, a front and a rear spar and ribs. In this configuration the covers and spars contain a high percentage of structural mass, so initial designs will focus on the strength and stability criteria for these components. Due to the modular approach of the ASM, the sizing modules used can be bespoke to the type of aerofoil analysed.

Covers sizing

The covers tool analyses the skin-stringer cover configuration as a stiffened panel. Global and local, longitudinal and shear buckling are considered for the skin and stringer web. In addition, the strength requirements are assessed using in-plane strains. The covers can be idealised as a series of super-stringers, these consist of the skin stringer flange and stringer web. **Figure 3** shows a wing panel with three super-stringers, with the panel loading paths.

The buckling solution used is dependent on the ratio of rib pitch to skin thickness for skin and skin-stringer interaction, and the ratio of web height to web thickness for the stringer web buckling calculation. If the thickness ratio is greater than 25, the transverse shear flexibility

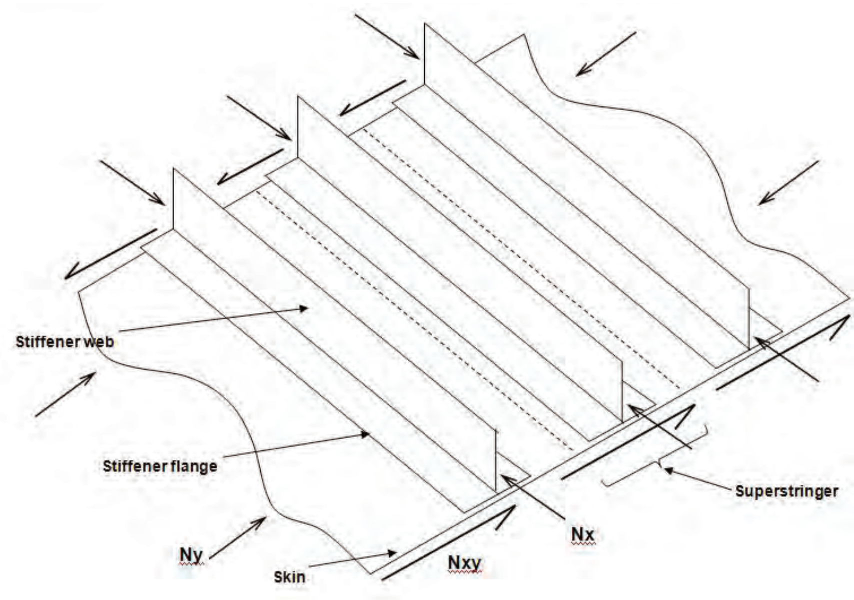

Figure 3. Skin-Stringer Super-stringer

becomes more significant and so is included in the buckling calculation.

In all buckling calculations, the critical buckling loads are compared against the running loads extracted from the ABAQUS analysis to ensure that the criterion has been met.

Skin buckling

The critical Buckling load is highly dependent on the D matrix of the skin. From [9] the critical buckling load of a long anisotropic plate, with simply supported boundary conditions along the edges, is presented in **Equation 1**, for skin longitudinal buckling, and **Equation 2**, for skin shear buckling.

Equation 1

$$N_x^{cr} = K_x \frac{\pi^2}{b^2} \sqrt{D_{11} D_{22}}$$

Equation 2

$$N_{xy}^{cr} = K_{xy} \frac{\pi^2}{b^2} \sqrt{D_{11} D_{22}^{3}}$$

For thick skins the Rayleigh Ritz method with transverse shear flexibility is used, where the compression and shear buckling load is calculated based on the principle of minimum potential energy. For the neutral equilibrium the total potential energy is balanced by a factor of the work done by the external loads.

The normal and shear buckling reserve factors are combined to calculate a normal-shear buckling interaction reserve factor. The lowest of these reserve factors is presented as the skin buckling RF.

Web buckling

The web buckling analysis is similar to the skin analysis. Local buckling of the skin and the web are considered separately assuming no interaction between them.

For thin webs the effect of the transverse shear flexibility is neglected, the web critical buckling load, Nwxcr, is calculated by **Equation 3**, [9].

When the ratio of stinger height to thickness is less than 25, a closed form solution is used, including the

Equation 3

$$N_{wx}^{cr} = \frac{12^2}{b^2} \left(D_{66} - \frac{D_{26}^2}{D_{22}}\right)$$

effect of transverse shear flexibility, given in **Equations 4** and **5**.

Equation 4

$$N_{wx}^{cr} = -\frac{1}{\dfrac{1}{4H_{55}} + \dfrac{h^2}{12D_{66}}}$$

Equation 5

$$H_{55} = \frac{5}{6}\, G_{12} t_w$$

Global buckling

Global longitudinal buckling, shear buckling and longitudinal-shear interaction are considered. The critical longitudinal buckling load accounts for the induced shearing force within the stringer web and is given by **Equations 6** and **7**.

Equation 6

$$P_{cr} = \frac{P_e}{1 + (P_e/(A_w G_w))}$$

Equation 7

$$P_e = \frac{\pi^2 EI}{a^2}$$

The critical shear buckling load, N_{sh}^{cr}, is calculated by **Equation 8**, where D_c is the bending stiffness of the super-stringer element per unit width.

Equation 8

$$N_{sh}^{cr} = K_{sh} \frac{\pi^2}{a^2} \sqrt{D_{22} D_c^3}$$

Using the longitudinal buckling and the shear buckling RF, the reserve factor for global longitudinal and shear buckling interaction is then calculated. The most critical global buckling RF is then presented as the global buckling RF.

Spar sizing

The ASM can currently analyse two types of spar to cover attachment, traditional C-Caps or Internal Spar Caps (ISC). ISC typically consist of a back to back curved section bolted together, co-cured or co-bonded to the covers. This is a more resistant joint to react the fuel pressure loads.

The spar tool involves five different analysis, cover and web stability analysis involving compression and bending buckling analysis, corner of radius analysis which calculates the maximum bending moment at the cap and the inter-laminates transverse shear stress, bolted joint analysis, strength analysis and dimensional constraints.

Corner of radius analysis

The corner of radius analysis calculates the inter-laminar stresses in the spar cap under maximum fuel pressure load and compared with the allowable stress to calculate the RF.

For traditional C-Cap, the maximum bending moment is constant and calculated using ESDU 93011. For ISC, this cannot be used as the bending moment along the ISC varies. Castigliano's 2nd theorem is used to calculate the bending moment at three positions, two points at the closing corner and the maximum bending moment at the opening corner. The maximum bending moment at these three points is used to calculate the inter-laminar stress.

Example

This example is presented to demonstrate the potential of using unbalanced to reduce internal loads under elevated 'g' conditions. Three configurations of aerofoil were considered, with varying use of unbalanced composite skin panels. Imbalance is defined as the percentage of angled 45° fibres in the -45° direction: Imbalance = 100 x N_{-45} / (N_{-45} + N_{+45}) %.

The first configuration, Case 1, has fully balanced composite skin panels, so there will be no direct-shear coupling terms in the skins. Case 2 uses balanced composite skin panels on the inboard section of the wing and 80% imbalance on the outboard panels. Case 3 uses composite panels with 80% imbalance for all cover panels. The conditions chosen are used for comparative studies, and so an optimum is not yet achieved.

The geometry of the unswept aerofoil was selected to locate the shear centre of a section midway between front and rear spars. The aerodynamic loads were applied at the shear centres, ensuring that the aerofoil does not twist under lift loads. Although this may not be representative of typical aircraft wing characteristics, it does allow an understanding of the effect of unbalanced composite panels on the twist of the aerofoil without secondary effects.

The wing analysed has a half span of 10m and a chord of 2m at the root, tapering to 1m at the tip. Constant thickness to chord ratio was maintained throughout. There are six span-wise skin panels with thickness varying from 20mm at the root to 5mm at the tip.

Lift-induced twist effects

The inbuilt twist of the aerofoil was adapted to give an elliptical lift distribution at the 1g condition. The inbuilt twist applied is within the linear lift region and is presented in **Table 1**, with Rib 1 as the furthest inboard rib and Rib 6 as the furthest outboard. This wing design was then tested with an elevated 'g' manoeuvre of 3.75g applied. The total mass, and therefore lift, remains constant across the different configuration analyses.

Table 1 also reports the lift-induced twist for the three cases. For Case 2 there is negligible induced twist in the balanced section, but an induced washout of approximately 1° is achieved at the wing tip, with the majority of this twist occurring in Bays 3 and 4. Case 3 shows the most significant wing twist with 2.67° washout occurring at the Rib 6. However, the highest rate of twist occurs inboard, with a reduction in this rate at each rib bay going outboard. As discussed later, this reduction is due to the lower direct span-wise strains in the outboard section and the consequent reduced shear strains induced by the coupling terms.

Influence on lift distribution

Figure 4 shows the impact that the differing angle of twist has on the lift profile. When compared to the balanced case, both wings that utilise unbalanced composite panels have a higher magnitude of lift inboard and a relatively lower magnitude of lift in the outboard sections. This lift profile reduces the bending moment across the span of the wing, **Figure 5**.

Strain distribution

Figure 6 shows the span-wise strain distribution for the top and bottom covers for each of the three cases. The inboard section of the wing experiences significantly higher levels of span-wise strain than does the outboard section. The thickness of the outboard sections is determined by minimum thickness (damage tolerance) requirements, which leads to lower maximum strains. As a consequence of the small span-wise strains the rate of twist in the outer wing is so low. Therefore Case 2, which utilises unbalanced panels only in the outboard bays, exhibits only limited twist.

Critical strength reserve factors (RFs)

From the balanced wing datum RFs, the critical sizing criteria for this structural configuration were skin buckling, global buckling and skin and stringer web strength. The initial RFs for the balanced case are shown in **Table 2** and the percentage improvements in RFs for the fully unbalanced and partially unbalanced cases are presented in **Table 3** and **Table 4**

Although the fully unbalanced Case 3 shows an improvement in RFs for the majority of criteria, it has not improved for all criteria. There is a large percentage reduction in top

Rib No.	Inbuilt Twist [deg]	Induced Twist [deg]		
		Case 1	Case 2	Case 3
1	8.95	-0.01	0.00	-0.66
2	9.53	0.00	0.01	-1.23
3	9.90	0.01	-0.02	-1.75
4	9.89	0.02	-0.43	-2.18
5	9.12	0.03	-0.83	-2.54
6	6.02	0.05	-0.98	-2.67

Table 1. Inbuilt twist for 1g load and induced wing twist for 3.75 g load

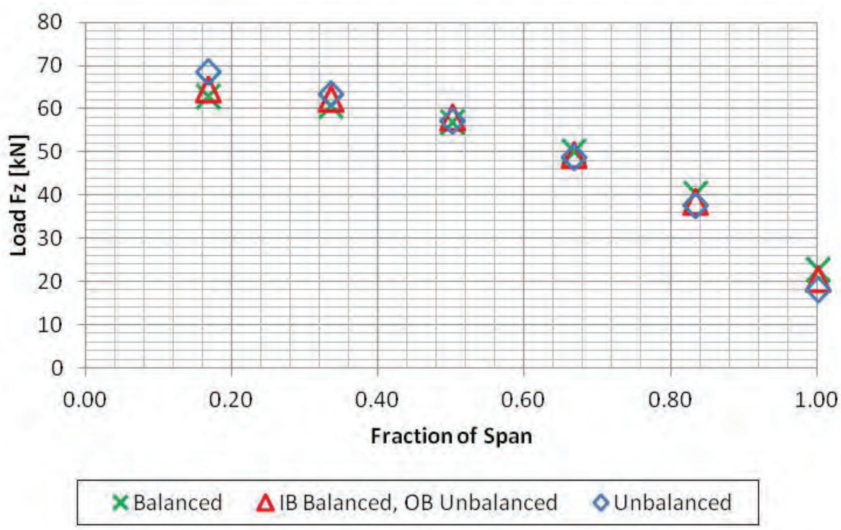

Figure 4. Vertical loading against Fraction of Span at lift nodes

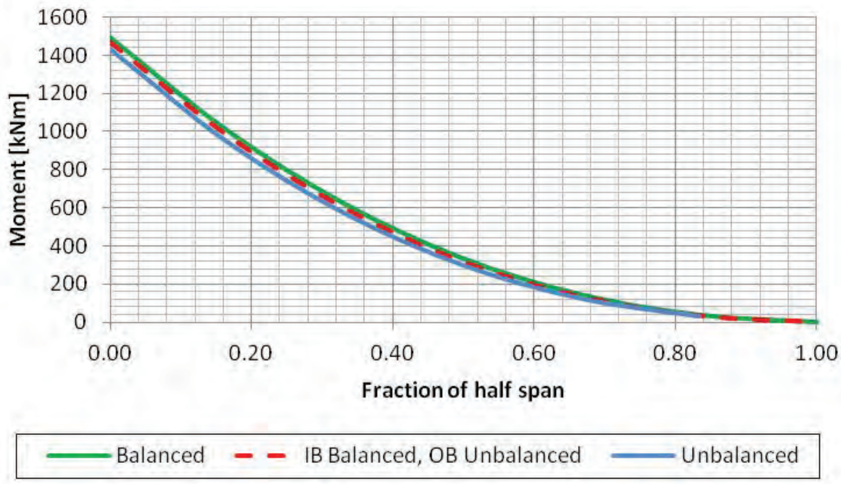

Figure 5. Span-wise Bending moment against Fraction of Span

cover skin buckling RF for the unbalanced case. This reduction is because the critical buckling load has reduced due to the coupling terms D16 and D26 in the unbalanced skin panel. Due to the high initial RF, the minimum RF is still acceptable.

There is a reduction in all RFs for Case 3 in Rib Bay 1. This reduction is due to higher peak span-wise compressive running load towards the trailing edge of the upper skin (and higher tensile running load in the lower skin) as illustrated in **Figure 7**. This is a result of the greater curvature at the trailing edge compared to the leading edge for Case 3.

The peak loading for the partially unbalanced wing, Case 2, is reduced by 2%. The inboard load profile for the partially unbalanced wing more closely follows that for the balanced wing, since the additional curvature is isolated to the outer wing bays.

For the partially unbalanced case, there was an improvement in most RFs, the exception being the top cover skin buckling in the outboard bays. Once again, this phenomenon is due to the reduction in buckling allowable for the unbalanced panels. All other critical RFs were improved with the addition of unbalanced composite panels. The average increase in RFs was 5.8% for Case 2 and 16.4% for Case 3.

The key points that can be determined from comparison of the three wing configurations are:

- The use of unbalanced panels in an aero-elastic tailored configuration can induce washout, reducing lift at the wing tips.
- Aero-elastic tailoring reduces the bending moment along the wing and, for this configuration, had the most pronounced effect with the entire wing unbalanced.
- The application of aero-elastic tailoring has the effect of improving most, but not all RFs,

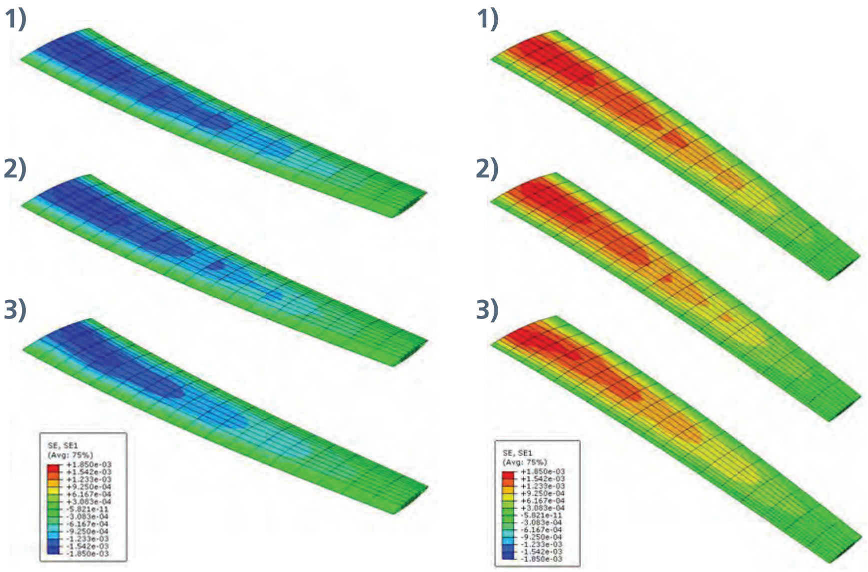

Figure 6. Top (left) and Bottom (Right) Cover Span-wise Strain ε_x for **1)** Balanced, **2)** Partially Imbalanced and **3)** Fully Imbalanced Covers

Criteria	Bay 1	Bay 2	Bay 3	Bay 4	Bay 5	Bay 6
Top Cover Skin Buckling	20.0	26.5	23.6	27.4	14.3	24.1
Top Cover Web Buckling	3.17	3.35	3.52	3.21	3.92	7.60
Top Cover Global Buckling	2.34	1.94	1.75	1.48	1.50	2.44
Top Cover Skin Strength	2.01	1.96	2.23	2.52	3.53	8.41
Top Cover Web Strength	2.01	1.96	2.23	2.53	3.53	8.41
Bottom Cover Skin Strength	2.00	2.02	2.25	2.50	3.39	7.29
Bottom Cover Web Strength	2.00	2.02	2.25	2.50	3.39	7.2

Table 2. Reserve Factors for the balanced covers

Criteria	Bay 1	Bay 2	Bay 3	Bay 4	Bay 5	Bay 6
Top Cover Skin Buckling	-56%	-50%	-40%	-31%	-21%	-15%
Top Cover Web Buckling	-4.6%	3.3%	9.6%	12%	15.2%	13.8%
Top Cover Global Buckling	-4.3%	3.6%	9.8%	12.3%	15.6%	14.1%
Top Cover Skin Strength	-6.5%	1.2%	7.4%	9.8%	12.9%	11.5%
Top Cover Web Strength	-4.6%	3.3%	9.6%	12.0%	15.2%	13.8%
Bottom Cover Skin Strength	-4.5%	3.2%	5.6%	9.2%	13.0%	18.9%
Bottom Cover Web Strength	-2.6%	5.3%	7.7%	11.4%	15.3%	21.4%

Table 3. Percentage improvement in RF from balanced covers to fully unbalanced covers

Criteria	Bay 1	Bay 2	Bay 3	Bay 4	Bay 5	Bay 6
Top Cover Skin Buckling	0.2%	2.5%	3.7%	-35%	-28%	-25%
Top Cover Web Buckling	0.2%	2.5%	3.7%	5.1%	5.4%	-0.5%
Top Cover Global Buckling	0.2%	2.5%	3.7%	5.4%	5.8%	-0.2%
Top Cover Skin Strength	0.2%	2.5%	3.7%	3.0%	3.3%	-2.5%
Top Cover Web Strength	0.2%	2.5%	3.7%	5.1%	5.4%	-0.5%
Bottom Cover Skin Strength	1.5%	2.4%	3.9%	2.6%	3.0%	3.9%
Bottom Cover Web Strength	1.5%	2.4%	3.9%	4.7%	5.1%	6.0%

Table 4. Percentage improvement in RF from balanced covers to unbalanced covers on OB section

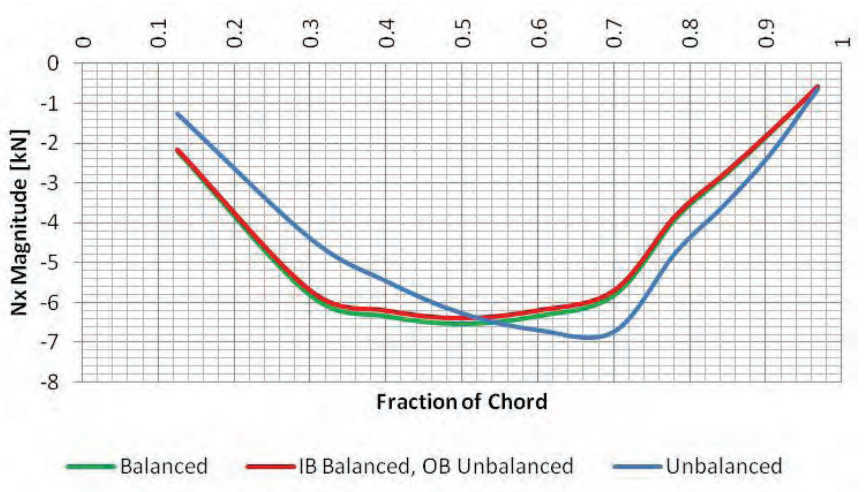

Figure 7. Span-wise section force at the wing root

dependent on the configuration of unbalanced panel selected.

- Aero-elastic tailoring can alter the distribution of strain in the top and bottom covers, particularly in the bay adjacent to the root. This can in some cases result in increased peak strains despite a lower overall strain.
- A wing design with unbalanced panels in the outboard sections and balanced panels in the inboard sections, shows promise as an effective means of reducing wing bending moment without increasing peak strain at the root.
- With the current structural configuration, the strain in the unbalanced outboard cover panels is not sufficient to twist the aerofoil to the same extent as an unbalanced inboard section, tempering the effectiveness of aero-elastic tailoring. This constraint has limited the twist achieved by the partially unbalanced case.
- In order to increase the effect of aero-elastic tailoring the configuration should be changed to increase the outboard strains.

Conclusions

The initial requirements for the ASM were:

- Analyse a wide range of aerofoil structural configurations;
- Full, six degree of freedom (translational and rotational) loading;
- An ability to simulate static, steady state and transient dynamic loads;
- Allow both linear and non-linear response;
- Perform rapid sizing iterations;
- Incorporate bespoke integrity assessment criteria;
- Allow simple modification to internal structural configuration (e.g. rib pitch).

Steady state dynamic analysis has not been implemented with the current ASM. However all other requirements have been attained. In order to make the ASM completely generic, enhancement of the aerodynamic component of the loading module will be required to accommodate specific applications.

An example has been used to show the potential benefits that could be achieved by utilising unbalanced composite panels, either in the whole of the covers or the outboard section of the wing. From this study the preferred configuration is to use balanced composite panels in the inboard section of the wing. By this means the internal loads are reduced through aero-elastic effects, but the strength is maintained in the inboard section of the wing.

It has been identified that in order to improve the effectiveness of aero-elastic tailoring, the outboard sections of the wing will need to exhibit a greater span-wise strain to generate a greater degree of induced twist. This could be achieved by tapering the thickness to chord ratio along the span, or reducing cover thickness subject to other constraints.

The ASM has a significant potential for improvement and extension. Enhancements include an optimisation routine to refine the structural configuration using the results from the sizing analysis. The aerodynamic module has a considerable scope for extension, including modelling the onset of stall. The ability to simulate steady state dynamic loading is top of the agenda for future development.

The ASM offers a rapid solution to understand the impact of configuration changes, including aero and inertial elastic tailoring at the initial design phase.

Acknowledgement

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Management's responsibility for lively structures

Abstract

New Recommendations for the dynamic performance of grandstands and stadia are addressed to management as well as to designers, since it is they who are responsible for the comfort and safety at each event. But the guidelines require that a specialist be employed to lead the dynamics assessment to liaise with the designers and to advise and report to the management.

Introduction

Stadia and arenas can all be made to vibrate by active people. Most of the time the vibration caused by crowd movement is imperceptible but when crowds co-ordinate their movement to a musical beat it can be noticeable, and for some facilities, quite uncomfortable. Different structures have different susceptibilities and it is the management's responsibility to ensure any vibration does not get out of hand. So what are the concerns and when and where might there be a problem?

The first concern is discomfort for the crowd, especially for those individuals who remain passively seated amongst a lively crowd who are standing, bobbing and jumping to a beat. They are going to be more aware of the vibration than their neighbours and more disturbed by it. Also, those relaxing in hospitality suites can become uncomfortably aware of room, table and crockery vibration caused by the crowd outside. The second is panic, which must be avoided at all costs. Panic for one individual is serious and potentially life threatening when many panic together.

The worst vibrations will occur when a crowd is both lively and co-ordinated, so the most serious situations occur where there is music accompanying rhythmic activity. At lively sporting events, even when there is no music being broadcast, supporters can sing and move rhythmically to their own chants or songs, causing their movements to be co-ordinated. Pop or rock concerts can be expected to produce the most vibration, the amount depending on the artists and the type of audience they attract.

Some facilities are not at risk; it depends entirely on the design of the structure. Frames with long spans and tiers that are cantilevered many rows out from the supporting structure are the parts most susceptible to undesirable vibration from crowd activity.

Design and management of structures such as grandstands that are used for crowds and events where vibrations and crowd reaction can be a problem, is the subject of recent Recommendations by a Joint Working Group of the Institution of Structural Engineers, the Department for Communities and Local Government and the Department for Culture Media and Sport. The Group started its work in January 2000, published Interim Guidance based on the knowledge available at that time, in November 2001 and has now produced its Recommendations, some of which follow from laboratory research, full scale testing and the development of new theory all undertaken since 2000. The new document, 'Dynamic performance requirements for permanent grandstands subject to crowd action – Recommendations for management, design and assessment' provides the most comprehensive coverage of these issues available. The Recommendations replace the Interim Guidance and have been adopted by Government in the context of Building Regulations and Safety Certification. A letter sent out jointly by DCLG and DCMS in November 2009 to regional and local authorities in England and Wales recommended, as good practice, the use and application of the new recommendations so those concerned about commissioning new or modified facilities, or contemplating staging an event likely to be more lively than those previously staged, should now be working to the new Recommendations.

They offer advantages over earlier guidance:

- More options at the design stage;
- Management versatility in the use of their facilities and, as a consequence, some structures considered unsafe under earlier recommendations can be shown to be acceptable; either (a) always or (b) for particular events and operational arrangements.

The new Recommendations have been written for everyone who has responsibility for grandstands, including owners, operators, managers, architects, insurers and engineering designers, as well as local authority staff dealing with building control and safety issues, all of whom are interested in ensuring events are run with an expectation of comfort and safety. The detailed technical matters relating to structural dynamics are confined to the appendices, leaving the main document as a handbook for management, requiring:

- Management to take on-going responsibility for the dynamic performance of their facilities, keeping records of all dynamic assessments for future reference and maintaining documentation describing events, and recording stewards' comments and any complaints of discomfort;
- Dynamic testing of new or modified facilities by a competent testing house to confirm the results of calculations;
- A 'Listed Engineer', who has the necessary experience and competence, to lead the dynamics assessment, liaise with the designers and to advise and report to the management. The Institution of Structural Engineers keeps a list of such engineers.

It was considered that there was a need for listed engineers because, the combination of structural dynamics and rhythmic crowd action is a specialist field and it is important that dynamic assessments are carried out under the direction of an engineer competent to do so. Management should take special care in the appointment of the listed engineer, whose role is defined in the Recommendations and who has the responsibility to keep management informed about the process. Those on the list are self-certified, so management needs to be satisfied that the listed engineer they choose has appropriate credentials and experience and will be personally available to management for the duration of the task.

The new Recommendations mark a major step forward in managing lively structures. Based on recent research, they can be used to predict dynamic performance more accurately and with more confidence than before. With the requirement that a Listed Engineer should be given the responsibility for overseeing the structural dynamics aspects of design and assessment and to advise and report to management throughout the process, management can expect to be fully acquainted with the dynamic capabilities of their structures and the types of event they may host together with any palliative measures that may be necessary.

Acknowledgements

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Design of Aberdeen Channel Bridge, Hong Kong

Abstract

Aberdeen Channel Bridge will form part of the new South Island Line (East) of Hong Kong's Mass Transit Railway (MTR). The 225m long post tensioned concrete box girder bridge will cross the Aberdeen Channel linking a section of viaduct on Hong Kong Island to a tunnel on Ap Lei Chau Island. The local topography and the presence of existing infrastructure tightly constrained the arrangement of the bridge. This paper describes some of the challenges in designing a durable structure which fits into the local environment.
To reduce maintenance requirements bearings were eliminated from the pier tops by providing split integral piers. The bridge is to be cast in situ as balanced cantilevers; during this construction stage the split piers will need to be braced. A number of measures were taken to ensure the long term durability of these piers, including adopting Eurocode crack width requirements, installing cathodic protection from the outset and specifying higher performance, marine environment grade concrete. The bridge is in a heavily populated area and to reduce the local impact full enclosure noise barriers are to be provided. These increase the wind loads on the structure and this, combined with a short back span required to meet site constraints, led to large uplifts at the abutments. Measures taken to reduce this are outlined. The overall sustainability of the design was assessed using a recently developed in-house bridge sustainability index tool. The methodology and results of this tool are discussed.

Keywords: Post- tensioned concrete, balanced cantilever, sustainability

Introduction

The proposed Aberdeen Channel Bridge (**Figure 1**) is a 225m long post tensioned concrete box girder bridge forming part of the South Island Line (East) Mass Transit Railway (MTR) extension in Hong Kong. The new MTR line is approximately 7km long and will link South Horizons on Ap Lei Chau Island to the south with Admiralty station on the north side of Hong Kong Island. The line extension comprises a mix of tunnels and viaducts. The railway will cross Aberdeen Channel onto Ap Lei Chau Island to the south adjacent to the existing Ap Lei Chau highway bridge on the west side. The existing bridge was constructed in 1978 with a second parallel crossing added to widen the road in 1992. On the Hong Kong Island side to the north, the new bridge will connect to viaducts on a rocky outcrop. On the Ap Lei Chau Island side the line will run directly into a tunnel on leaving the bridge, under the rocky outcrop and existing tower blocks.

The detailed design of the bridge was undertaken by Atkins as part of a traditional contract covering the bridge, sections of viaduct works and certain stations for the Client, MTR Corporation. The construction contract was put out to tender in summer 2010 with construction of the bridge programmed to begin in 2011.

Form of bridge

The bridge consists of three continuous spans of lengths 58.5m, 115m and 73m (**Figure 2**). At the south abutment the bridge leads straight into a tunnel, at the north abutment the deck rests on a short abutment shared with the viaducts. The two intermediate piers are aligned with the existing Ap Lei Chau Bridge piers to minimise disruption to the sea channel and for aesthetic reasons. The bridge is required to clear the navigation channel, which is 70m wide and 14m high above sea level in the main span. Due to the sloping vertical alignment of the tracks, the south pier is 13m tall and is founded on a pile cap to be constructed on the existing sea wall. The north pier is 17m tall and founded on a pile cap in the sea channel. The main span to side span ratio, particularly for the south span is larger than would have been preferable to maximise the efficiency of the bridge, but could not be altered due to the topological and aesthetic constraints.

The deck is an internally post-tensioned single cell concrete box girder. Two MTR tracks cross the bridge deck which is 10.8m wide. A single cell box section was selected (**Figure 3**) as the deck is relatively narrow. The box girder varies in depth from 6.5m at the piers to 3.0m at midspan and abutments, with a parabolic curved soffit. A simple uniform cross section was selected for ease of construction. The noise assessment for the scheme has identified the need for full enclosure noise barriers in some locations. A continuous upstand for the anchorage of noise barriers is provided on each deck outstand over the length of the bridge. The full height noise barriers will not initially be installed on the full length of the bridge, but the design was required to allow for any possible retro-fitting. The bridge is to be cast in situ using the balanced cantilever construction method.

The bridge is integrally connected to the two intermediate piers with bearings only provided at the two end abutments. The integral connection at the pier tops was provided to eliminate the

Figure 1. Proposed new Aberdeen Channel Bridge

maintenance liability of bearings at the top of the piers which would be difficult to inspect and replace. The integral connection is also more stable for the construction of the balanced cantilevers, reducing the need for temporary works at the pier heads.

At the initial design stage options considered for the bridge form included a cable-stayed bridge, a fin-back bridge and an extra-dosed bridge. The post tensioned concrete box option was selected as it was viewed to harmonise with the existing bridge, which has a similar form, and was the most economical option.

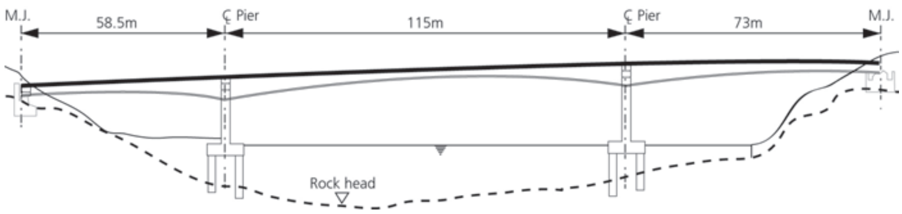

Figure 2. Bridge elevation

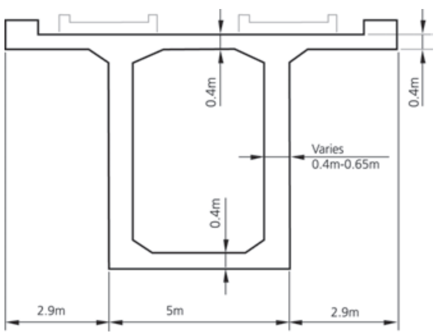

Figure 3. Typical cross section

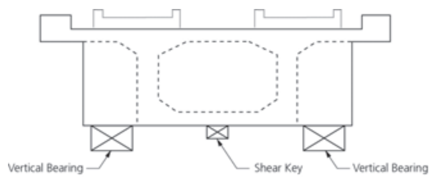

Figure 4. Widened abutment diaphragms to minimise uplift at bearings

Deck design

Design standards

The bridge was designed to MTR's in-house design standards[1]. These standards are based on the Structures Design Manual for Highways and Railways (SDMHR) published by the Hong Kong Highways Department[2]. The SDMHR is largely based on BS 5400[3] but with modifications appropriate to local conditions in Hong Kong. For some aspects of the design, reference was also made to the new Eurocode BS EN 1992-2 Design of Concrete Bridges[4].

Global analysis

The global analysis of the bridge was carried out using a 3-D space frame line beam model. A separate shear flexible grillage model was used to determine the distribution of load effects between the two webs. The space frame model allowed the construction sequence and tendon stressing operations to be taken into account and automated the creep, shrinkage and loss calculations for the post tensioned box.

The train live loading on the bridge followed the requirements set out in the MTR design standards, which are based on BS 5400 loading with modifications for the specific MTR trains. One of the most critical load effects on the bridge was the wind loading. The Hong Kong region is affected by typhoons and the SDMHR makes allowance for this by specifying high Ultimate Limit State (ULS) factors for wind loading. The overall effect of the high wind loads was also increased by the requirement to allow for the presence of the noise barrier in the design. The barrier presents a greatly increased total wind area, more than doubling the transverse wind loads.

During the construction of the bridge, the most critical stage is when the majority of the balanced cantilever decks have been cast but the midspan stitch has not yet been installed (**Figure 5**). At this point the wind loading can generate large transverse forces at the piers and any out of balance construction loading can generate large moments in the piers. The out of balance construction loading considered was a Uniformly Distributed Load (UDL) on one side of the deck representing either the concrete pours out of balance or the temporary formwork out of balance, but not both. Other out of balance effects such as unintentional deviation of concrete thicknesses and construction surcharges/storage loads were also considered. Such considerations are now more formally given in BS EN 1992-2, although this was not specifically used for the design of Aberdeen Channel Bridge.

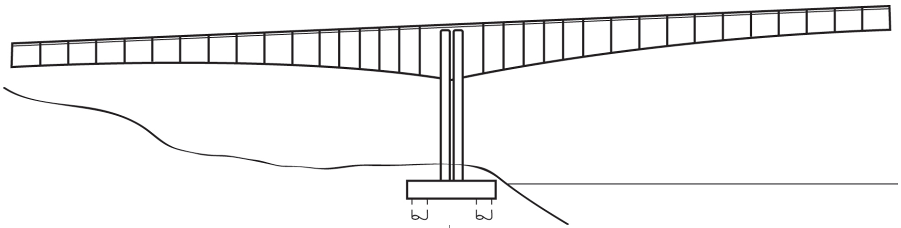

Figure 5. Balanced cantilever construction prior to stitching to adjacent deck sections

Uplift

The short side spans meant that under some loading conditions, uplift was generated at the bearings. The transverse wind loading on the noise barrier also generated a twisting at the deck end which gave more uplift on one side. A number of options to eliminate this uplift were investigated. Increasing the mass of the deck at the side spans by increasing the wall thicknesses and extending the solid diaphragm section was considered. However this did not provide sufficient additional mass to fully eliminate the uplift. The uplift generated by the eccentric transverse wind loading was minimised by widening the diaphragm at the bearing location (**Figure 4**). This both increased the total dead load and increased the spacing of the bearings, thereby increasing the lever arm for the bearing reactions, reducing the final uplift force.

In the final design the uplift at the north abutment was eliminated and uplift bearings do not need to be provided here. At the south abutment where the side span is shorter due to the need to link into the adjacent tunnel, uplift could not be eliminated and special uplift bearings will need to be provided. At both locations the transverse loads from the wind at the deck ends can become very high and so a separate longitudinal guide bearing is provided to resist these loads (**Figure 4)**.

Diaphragm design

The deck diaphragms at the piers and abutments are required to transfer the loading from the box section into bearings at the abutments or into the piers. These elements were designed using strut and tie analysis. The checks of the compression struts, ties and nodes were carried out in accordance with the guidance given in BS EN 1992-2. Limits on compressive stresses and tension at ULS are given in this code. At the Serviceability Limit State (SLS) the tension stress in the bars was limited to 200MPa to ensure strains under service loads and thus cracking was limited. "Suspension reinforcement" to transfer the shear forces from the box webs into the top of the diaphragms was provided based on the guidance by Leonhardt[5]. Separate models were developed for the maximum torsion and maximum shear load cases.

Tendon arrangement

The post-tensioning tendons are arranged in the box in accordance with the guidance in Appendix D of BS 5400 Part 4. In order to simplify the construction process, as much repetition of detailing as possible was adopted. The box section requires a significant amount of post tensioning to be installed. Ensuring that the tendons were arranged to minimise curvature (to minimise losses) and ensuring sufficient cover is always provided to prevent any pull out failures, required careful consideration of the 3–dimensional layout of the tendons. An existing in-house clash detection software package was modified to ensure the optimum layout of the tendons. The 3-dimensional box dimensions and the tendon layouts were input into a model and the software checked that the limits in Appendix D were met for every tendon at every location along its length. **Figure 6 and 7** show the visual output from this clash detection software.

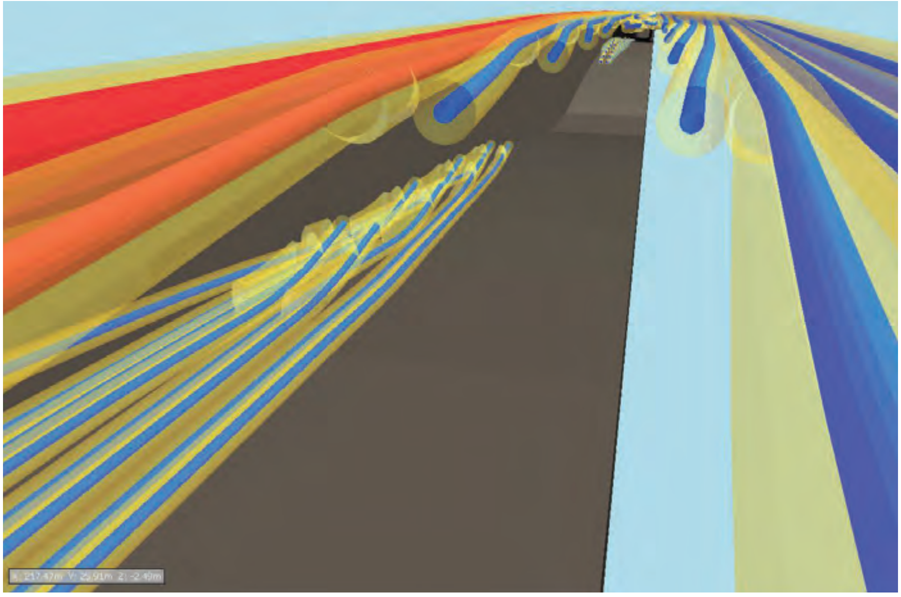

Figure 6. Clash detection for tendon geometry including minimum spacing

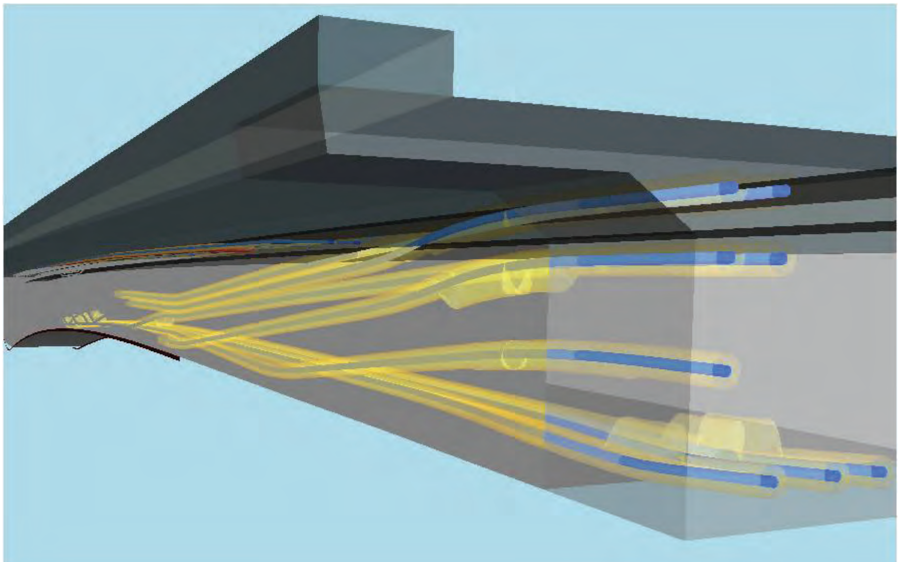

Figure 7. Clash detection for tendon geometry including minimum spacing

Pier and foundation design

Split pier arrangement

The piers are integrally connected to the deck without bearings. This is intended to reduce the maintenance costs of the bridge in service, but generates issues for the pier design. Expansion and contraction of the deck due to temperature changes, generate longitudinal forces at the top of the piers as the piers deflect. The size of these forces depends on the stiffness of the piers. Rigid piers will restrain the deck from changing length and so will generate very large longitudinal forces at the top (and bottom) of the piers. More flexible piers will provide reduced restraint, thereby reducing the longitudinal loads generated in the piers

Initial design checks showed that a single pier design, either solid or hollow that provided sufficient strength for most load combinations would be too rigid, generating a greater longitudinal force at the top of pier than could easily be resisted in bending. A split leaf pier design was thus proposed. The two separated leaves have considerably less longitudinal stiffness than a single pier, whilst retaining a similar effective strength.

Piers during construction

During the construction of the bridge the most critical stage for the piers will be once the majority of the deck has been cast, but before the two balanced cantilevers have been joined by the stitch in the main span. The moment and shear forces exerted on the pier at this stage greatly exceed those experienced whilst the bridge is in operation. Rather than design the split piers to resist these critical short-term, temporary construction forces, temporary bracing of the piers was specified during the construction phase. By temporarily connecting the split leaf piers their stiffness is increased and the second order moments generated in the piers are reduced. Consequently the final reinforcement quantities required in the piers are also reduced.

Serviceability design of piers

Initial crack width calculations for the piers indicated that design crack widths may exceed the simple limits specified in BS5400. This could have been solved by providing extra reinforcement in the face of the piers. However the piers were already heavily reinforced and there was a concern that additional reinforcement, although allowing the design to meet the code requirements, would actually reduce the durability of the structure due to the adverse effect such congestion may have on placing the concrete, with perhaps a reduced quality of concrete and cover.

The new Eurocodes take a different approach to BS5400 in the consideration of crack widths in design. Larger peak crack widths under short-term loading are permitted, even in aggressive environments and crack width limits are considered only under quasi-permanent loads (i.e. the time-based average bridge loading). There are a number of reasons for these differences. Crack widths are limited in structures for three main reasons: to prevent corrosion of reinforcement, to prevent unsightly appearance and to prevent loss of service, e.g. by leakage in a liquid containing structure.

For the piers on this bridge only the first two of these requirements apply. BS EN 1992-2 permits wider crack widths as research into durability has found that below 0.3mm the width of cracks has little influence on the durability of the structure. The quality of the concrete cover has the greatest influence on overall durability. Eurocode 2 bases its crack width requirements on the quasi-permanent load combination as they recognise that it is not the widest crack that opens during service of a structure, rather the time averaged mean crack width. This means that for road and rail bridges no traffic loading is included when crack widths are checked. Cracks that open and close quickly under peak service loading will have little time to affect durability. A crack which is permanently open however does provide a sustained path to the reinforcement.

The limit on crack width for appearance reasons are based on studies that have shown that members of the public are generally unaware of crack widths of 0.3mm and below, but start to notice larger cracks. This is based on a viewing distance of 3 metres. A value of 0.3mm has therefore been taken as the upper bound to acceptable crack widths where the concrete surface is visible, irrespective of durability requirements.

It is recognised that additional measures such as special coatings or cathodic protection need to be taken to enhance durability for some situations. Horizontal surfaces subject to de-icing salts is a clear example and vertical surfaces subject to salt water spray such as the piers on Aberdeen Channel Bridge close to the tidal splash zone also require further consideration. In conjunction with the Client, it was decided that cathodic protection should be provided for the piers to enhance their long term durability, despite not being strictly necessary to BS EN 1992-2.

Cathodic protection system

Initially, a sacrificial anode system submerged below sea level and attached to the pier foundations was considered. However the beneficial effects would be limited by the height of influence above sea-level that could be achieved and which would be limited to approximately 2m. Another solution was to embed sacrificial anodes in the concrete piers. However their effective life is relatively short (approximately 10 years), and they would then need to be replaced by breaking out the surrounding concrete and then repairing it. This in itself is expensive and would affect the long-term durability of the piers, especially with their inherent difficult access.

An impressed current cathodic protection system was therefore chosen as the preferred option for the piers on the Aberdeen Channel Bridge. This system can be installed at the time of original construction and consists of ribbon anodes installed on top of the reinforcement cages in the piers and pilecaps using isolating clips. Cabling for a data monitoring system and power supply system will also be installed in the piers with junction boxes in the hollow box girder deck at the pier locations. Data about the condition of the reinforcement will be collected and monitored, using a suitable interface system to detect the onset of corrosion at an early stage. The low powered current supply to the ribbon anodes would then be switched on to actively prevent corrosion of the reinforcement. This system has a design life of 120 years, and when installed at the construction phase of a structure, does not significantly increase overall costs as it is relatively easy to install.

Additionally, marine grade concrete was specified for the piers to further improve the long-term durability of the structure and minimise maintenance requirements in the difficult location.

Foundations

The pier foundations are both formed of 4 no. 2.0m diameter piles, with a rectangular pile cap. The piles are founded in 2.0m deep rock sockets. The pile cap within the sea channel will be fitted with a fender system to protect it from ship impact and will be cast above highest water level to ensure it is visible from sea channel vessels. The bridge was designed to resist ship impact, however the vessels in the channel are of limited size and so this was not generally found to be a critical global load case.

As the rock head level in the vicinity of the foundations for the piers is relatively shallow, various construction options were considered for the marine pier. Spread foundations were considered at the feasibility stage, constructed either by de-watering a temporary cofferdam and excavating to acceptable bed-rock, or using precast caissons, sunk onto the seabed with in situ mass concrete. The former option was used for the adjacent Ap Lei Chau Highway Bridge pier foundations, so this solution has already been trialled successfully in this location.

Various construction methods for a piled solution for the pier foundations were also investigated. Options including construction of a temporary island, construction of a temporary cofferdam or using a barge or jetty were considered. The final construction method proposed for the detailed design was to use a temporary cofferdam to create a piling platform, which also provides a dry area for constructing the pilecap. This option minimises disturbance to the seabed and the plan area of the construction works for the marine pier foundation in the sea channel.

The south pier is located to ensure that the main span of the new bridge is in line with the existing highway bridge main span. This places the foundation for the south pier on top of the existing seawall. During construction of the south pier and foundations, the seawall will need to be temporarily removed and a piling platform constructed over the foundations of the seawall. Once the piles and pilecap are cast, the seawall will be rebuilt in the same location and the rock revetments will be reinstated to the same extent in the sea channel.

Abutment and foundation design

North abutment

The north abutment is located on the steep slopes of the rocky outcrop with the Seminary building on top. It provides the end supports and shared interface structure for the Aberdeen Channel Bridge and the adjacent viaducts. The articulation of both structures requires free mechanical bearings at this abutment, therefore two sets of bearing shelves are needed to locate the bearings and jacking positions for future bearing replacement. A suitable gap and expansion joint was designed to allow for the relative movements of both structures.

A relatively small bank seat arrangement is used in this location due to the topography. The rock head is relatively close to ground level but falls steeply in the transverse direction. Therefore a spread foundation, 3m deep to account for the dipping rock head, was designed for this location.

A requirement of the Hong Kong Fire Department is to provide emergency access off the Aberdeen Channel Bridge. Generally, MTR evacuation routes are from the end of the train carriages between the rails of the track, keeping the deck cross-section relatively narrow. However it was agreed with the Hong Kong Fire Department that the route off the bridge will be accommodated by widening the deck at the north end to provide an emergency walkway on one side of the tracks. This walkway leads to a set of stairs at the north abutment that connects to the existing adjacent highway bridge, which is at a lower elevation at this location. Emergency access steps from the highway parapet to the pedestrian walkway on the highway bridge will also need to be installed to complete the emergency access route.

South abutment

The south abutment is located on the steep slopes of the rocky outcrop with the Sham Wan Towers building on top. It provides the end supports and shared interface structure for the Aberdeen Channel Bridge and the tunnels of the adjacent contract.

A relatively small bank seat arrangement is used in this location due to the topography. A bearing shelf with discrete bearing plinths is included to accommodate the mechanical bearings and jacking positions to allow for future bearing replacement. A suitable gap and expansion joint is designed to allow for the movement of the bridge. The abutment will also have an inspection gallery as required by the HKDSM with a curtain wall behind to support the earthworks at the back of the abutment. The rock head in this location is at the approximate level of the abutment foundation but also dips down steeply in the transverse direction. Therefore to minimise excavation, a spread foundation 3m deep has been designed for this location.

Sustainability

The Atkins sustainability index tool

Atkins has recently developed an in-house tool to determine and measure various sustainability aspects for bridge schemes. The objective is to use the tool during both outline and detailed design phases of a scheme to understand the effects of changing different aspects of a structure on its overall sustainability. The Sustainability Index raises awareness of sustainable aspects of design and construction of a scheme to all members of the team, including the Client.

Aspects considered by the Sustainability Index Tool include:

- Economy (Initial Cost, Whole Life Cost, User delay during construction)
- Environment (Extent of loss of habitat, Noise during construction, Noise during service)
- Society (Aesthetics, User delay during construction)
- Resources (Use of recycled material, Consumption of natural

resources, ease of modification or demolition)

- Climate Change (Carbon footprint (construction), Carbon footprint (maintenance))

The tool allows the user to calculate a weighted score of between 1 and 10 for each aspect of sustainability listed above, with the lowest score being most favourable. The overall score for the bridge is the total score from the sum of the scores of each aspect, with the lowest overall score being most favourable. The scores for each aspect are also represented visually by the tool using a 'spider chart' to highlight aspects with relatively high scores.

The use of the tool depends on assumptions made by the user based on best information available at the time of undertaking the sustainability assessment. The score can be adjusted to match the more accurate information available as the scheme progresses.

Use of the sustainability index tool for Aberdeen Channel Bridge

The sustainability index tool was used during the detailed design of the Aberdeen Channel Bridge to understand the implications of decisions on design aspects and construction methods, some of which could be varied, while other choices were required due to either physical or client requirements. **Figure 8** shows the completed Sustainability Index for Aberdeen Channel Bridge.

Sustainability aspects that are influenced by the design process include the use of natural resources, which has a high score for this structure. This is due in part to a heavy concrete deck profile relative to the area of the deck, as the bridge is relatively narrow but has relatively deep sections. Another aspect relates to the extensive use of prestress in design, making demolition or modification of the structure complex

Bridges Sustainability Index - Output
Project MTR - South Island Line (East)
Structure: Aberdeen Channel Bridge

Attribute	**Measure**	**Grade**
Economy		
Initial cost	<2500, >2250 (£/m^2)	6
Whole life cost	<3250, >2750 (£/m^2)	4
User delay during construction	1 out of 13	1
Environment		
Extent of loss of habitat	3 out of 16	2
Noise during construction	7 out of 10	7
Noise during service	-1 out of 4	0
Society		
Aesthetics	7 out of 14	5
User delay during construction	1 out of 13	1
Resources		
Use of recycled materials	7 out of 11	7
Consumption of Natural resources	>2.50 (tonnes/m^2)	10
Ease of modification, demolition	6 out of 10	6
Climate change		
Carbon footprint (constructive)	<3, >1.5 (tonnes $C0_2$/m^2)	8
Carbon footprint (maintenance)	<0.05 (tonnes $C0_2$/m^2)	0

Overall index score = 0.427

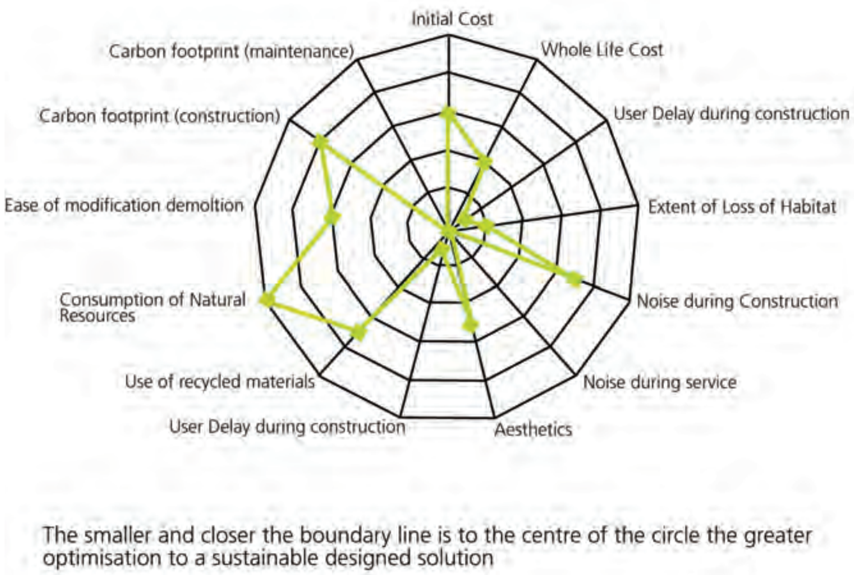

Figure 8. Bridges sustainablity index ouput for Aberdeen Channel Bridge

Structures

in the future. The initial cost of construction of the bridge is relatively modest based on the deck area and this keeps the score for this aspect at a moderate level. The design choices for the structural form will keep maintenance requirements to a minimum, thereby achieving a low score for this aspect of sustainability too.

As this scheme uses a traditional form of contract, where the detailed design is completed and then contractors will tender to construct the bridge, various assumptions were made in relation to construction aspects. The main assumptions are given below:

- It is unknown if recycled materials will be used during construction so it is assumed that only a limited amount will be used
- Distances to be travelled by materials have been conservatively assumed
- There will be a high use of natural resources
- Relatively large amounts of excavation will be required with associated removal of waste materials
- Night working will be used which may affect local residents and businesses.

These assumptions are generally conservative, and they particularly affect the carbon footprint of the scheme during construction, giving a high score for this aspect. This demonstrates how the construction process can improve the sustainability of a scheme further by choices of material types and sources and methods of working.

Continually using the Sustainability Index tool throughout the design of Aberdeen Channel Bridge highlighted that a low maintenance, economical structure that did not adversely affect the environment was developed. It also highlighted how the choice of materials and construction processes are important and affect the overall sustainability of the bridge.

Acknowledgements

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Bearing replacement and strengthening of Forth Road Bridge approach viaducts, UK

Abstract

The existing Forth Road Bridge spans the Firth of Forth in Scotland. Construction of the bridge was completed in 1964. The main structure is a three span suspension bridge with a central span of 1006m and side spans of 408m. At each end of the bridge there are two multi-span approach viaducts leading up to the main crossing. The deck of the approach viaducts comprises a pair of longitudinal steel box girders supporting a series of transversely spanning steel girders, both act compositely with a reinforced concrete deck.

The steel girders of the approach viaducts are supported on steel roller and rocker bearings. The bearings are fixed to reinforced concrete portal piers which are founded on rock. These piers vary in height between 11m and 40m. An initial study of the bearings identified that the rollers had locked up due to corrosion and distortion, and the concrete beneath the bearings and elsewhere on the pier tops had deteriorated due to chloride contamination. Assessment showed that structural deficiencies in the pier were exacerbated by both the concrete deterioration and change in articulation. These factors lead to the decision to replace all the bearings on the viaducts.

The original structure was not designed to facilitate replacement of the bearings so the structure had to be strengthened and modified by the addition of jacking stiffeners and corbels to the pier tops. Initial design was carried out to BS5400 leading to substantial amounts of strengthening being required. This was then repeated using Eurocodes to gain better structural efficiency and reduce the amount of strengthening required to the deck and piers. This paper outlines the design of the strengthening and modifications to the bridge to facilitate bearing replacement, together with a detailed description of the design of the temporary works needed to maintain the bridge's articulation during jacking. Lessons learnt during construction are discussed and the structural benefits of design to Eurocodes are emphasised.

Keywords: Roller bearings, bearing replacement, steel box girders, steel strengthening, Eurocodes, virtual reality modelling.

Introduction

The Forth Road Bridge (**Figure 1**) spans the Firth of Forth and was completed in 1964. The main structure is a three span suspension bridge. At each end of the bridge there are two multi-span approach viaducts comprising a pair of longitudinal steel box girders with cross girders supporting a concrete deck slab as shown in **Figure 2**. The approach viaducts carry two carriageways, each with two lanes, and extend from the abutments to the side towers, which are shared with the main suspension bridge. The total width of the structure is 36m.

The box girders rest on steel roller and rocker bearings on reinforced concrete portal piers, varying between 11m and 40m tall, founded on rock. The articulation of the two viaducts is shown in **Figures 3a** and **3b**. Locations with roller bearings allow for horizontal movement through movement of the roller while the locations with pinned bearings allow for movement through flexing of the piers.

During inspections and displacement monitoring, the existing roller bearings were found to exhibit little or no movement and varying amounts of corrosion. At the north side tower, the only roller bearing on the north viaduct, the roller was found to be nearing the limit of its movement range. **Figure 4a** shows a typical roller bearing, and **Figure 4b** shows the roller at the north side tower. Structural assessment of the rollers bearings to BS5400-9-1:1983 and BS EN 1337-4 showed that the original bearings did not meet modern geometrical limits and were significantly overstressed to the codes.

The rocker bearings were generally found to be in better condition than the rollers, though some corrosion was present. A typical rocker bearing is seen in **Figure 5**. A structural assessment was also performed on the rocker bearings, which generally found that the bearings complied with the requirements set out in BS EN 1337-6:2004.

An inspection of the pier tops showed concrete delamination occurring at many of the pier tops with patches of spalled concrete in the regions directly below the bearings. Therefore, the pier tops were tested for carbonation depth and chloride contamination which showed that many of the piers had high chloride contents and were at risk of further deterioration. Due to the poor concrete condition around and below the bearings, it was decided to replace all the bearings including the rockers (which would need to be lifted in any case for concrete repairs beneath them). To ensure the long term adequacy of the structure, a cathodic protection system was also installed as part of the bearing replacement scheme as discussed later in this paper.

Figure 1. Forth Road Bridge, viewed from south

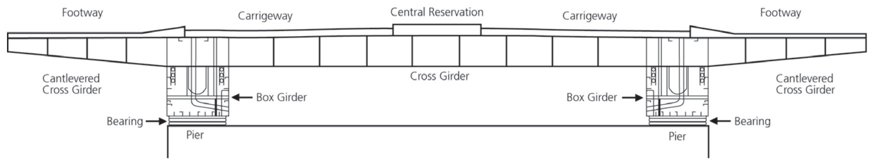


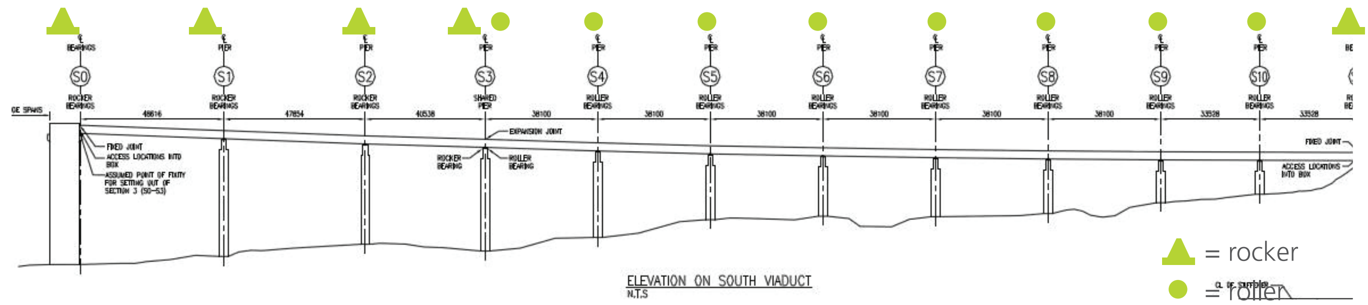
Figure 2. Cross section through deck

The original structure had no provision for bearing replacement so it was necessary to consider how best this was to be done through local modification and strengthening of the existing structure. The steel box and concrete substructure were first assessed to determine the existing resistances of the relevant components. The remainder of this paper describes the structural assessment, the options for the new bearing types and the design and implementation of the bearing replacement solution.

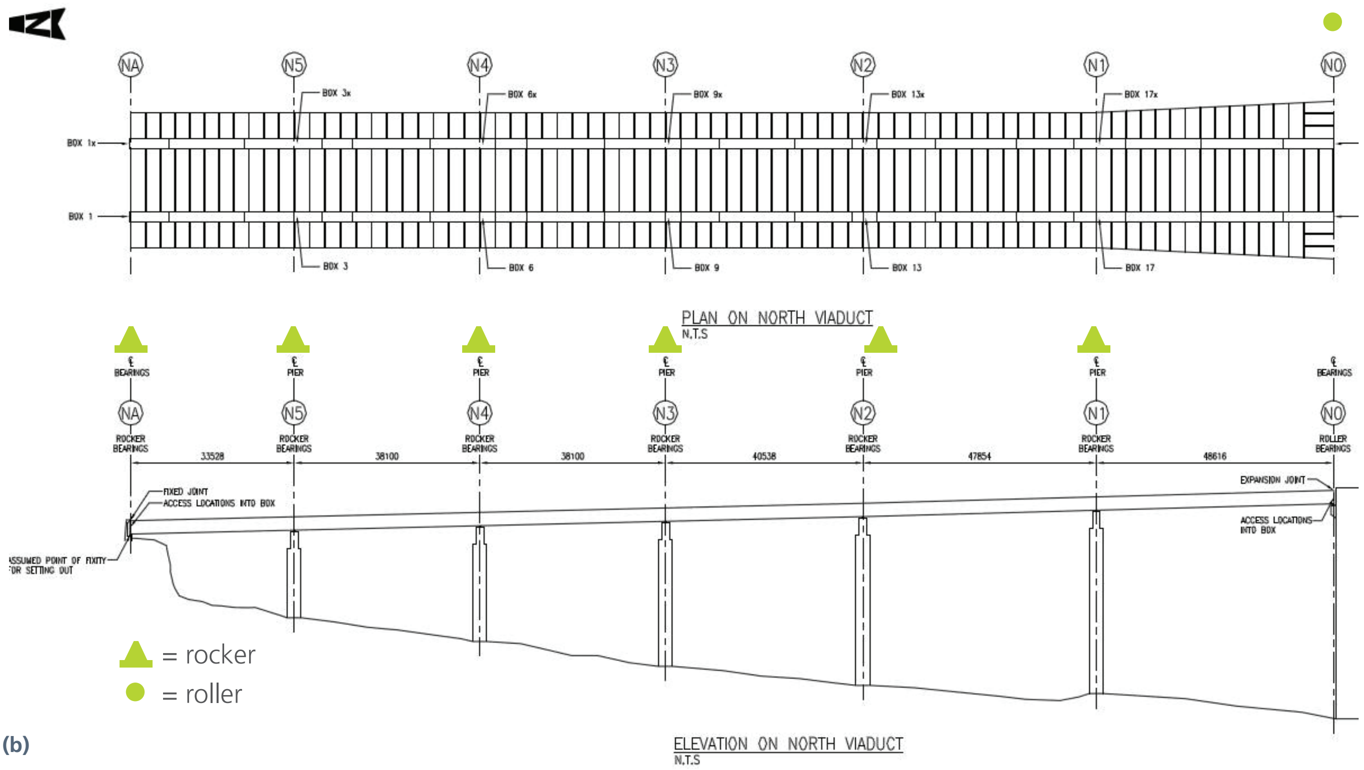
Bearing replacement options

Several options were considered for the bearing type to use in the replacement of the existing roller and rocker bearings, but the overarching intention was to restore the intended articulation of the bridge.

The most feasible options for replacement of the rollers were pot bearings, sliding bearings or replacement roller bearings. A pair of pot bearings was eliminated early on as an option because there was insufficient room to achieve an adequate lever arm between them to resist the torsional moments attracted by the boxes. A single pot bearing was also considered, occupying the full width of the diaphragm, but this would have required significant widening of the pier top to accommodate it. Using smaller pot bearings, partially loading the width of the diaphragms, would have been geometrically feasible but would have required strengthening to all the diaphragms; this would have been very difficult because of the extensive services passing through them. Moreover, a single pot



(a)



(b)

Figure 3. (a) Articulation at south viaduct, (b) Articulation at north viaduct

(a)

(b)

Figure 4. Roller bearings, (a) Typical roller bearing, (b) Roller at north side tower

Figure 5. Typical rocker bearing

bearing could not transmit torsional reaction from the box girders, necessitating strengthening to the cross girders at supports.

Replacing the current bearings with new roller bearings was also considered. This would involve no modifications to the diaphragm and there would be no change to the current articulation of the structure. However, the new bearings would have required a larger diameter than the original bearings in order to comply with BS EN 1337-4 and would not, therefore, fit within the available vertical space between the pier top and the box girder soffit. A higher material grade (with yield strength equal to or exceeding 690 MPa) was also considered to enable the diameter to be reduced, but sufficient confidence could not be obtained in the toughness of the steelwork that would be obtained.

The final solution therefore was to adopt a sliding bearing. This minimised the changes to the original articulation, whilst avoiding the potential problems with brittle materials. To minimise the effect on the existing structure, the bearing was detailed with the sliding surface on the lower surface so all the eccentricity occurred on the pier and not on the box girder diaphragms; the previous roller bearings shared the eccentricity between the pier and box. This had a negligible effect on the overall pier load effects but was beneficial to the box girder diaphragms. It was recognised that this detail could lead to durability problems so, to prevent ingress of dirt on the sliding surface, the bearings were detailed with a protective gaiter.

For the rocker bearing, a similar process was completed for the selection and replacement rocker bearings were selected. The new bearing types are shown in **Figure 6**.

(a)

(b)

Figure 6. New bearings, (a) Rockers, (b) Sliders

Assessment of box girders and piers – as built

To replace the bearings, the structure had to be modified to allow for new jacking points. Therefore both the steel box and the concrete substructure were assessed to determine the existing resistances of the relevant components.

Steel box girders

A typical internal arrangement of a steel box is shown in **Figure 7**. The steel box girders were initially assessed using BS5400-3 and BD 56, which found significant overstress in the existing structure. Particular areas for concern were the resistances to shear and shear-moment interaction. In addition, the shape limits for the stiffeners (to prevent torsional buckling) were not met. To avoid strengthening the boxes, Eurocode 3 was used to reassess the boxes, particularly BS EN 1993-1-5. This re-assessment eliminated the overstress under shear and shear-moment interaction, but the torsional buckling requirements for the stiffeners were not met. The overall amount of strengthening required to the box girders was correspondingly reduced, but not eliminated. As Eurocodes are based on specification of modern materials, it was important to investigate the impact of any departures from the material properties inherently assumed and take this into account during the assessment. Most relevant to this was steel ductility as the steel to BS968:1962 used in the construction of the Forth Road Bridge possessed lower ductility than modern steels. Coupon testing of steel from the boxes confirmed that the steel possessed adequate ductility to satisfy Eurocode requirements. Reference 1 gives further details of considerations for using Eurocodes for assessment.

Figure 7. Typical internal view of box girder

Concrete piers

The pier tops were also assessed using the strut and tie rules in BS EN 1992-1-1, as strut and tie analysis is not adequately covered by BS5400 Part 4 or BD 44. The existing pier reinforcement consists of mild steel plain round bars. Typical piers and the strut and tie models used for their assessment are shown in **Figure 8**.

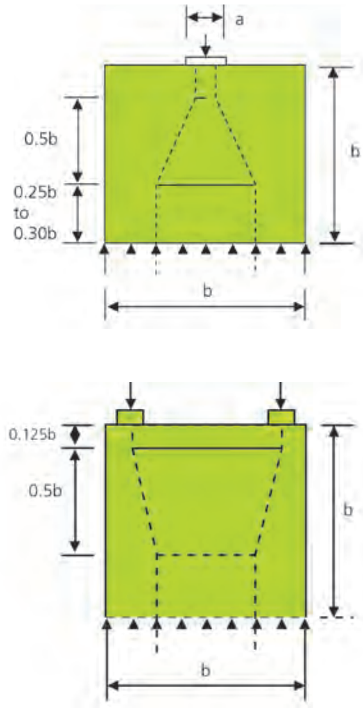


Figure 8. Analysis of typical piers with strut and tie analysis

This analysis showed that the existing reinforcement was not adequate to resist bursting loads in the pier tops, but the pier was adequate if the tensile strength of the concrete was invoked. Although this explained why there was no distress in the existing condition, it was not considered an acceptable permanent solution considering the ongoing concrete deterioration.

The piers were also assessed for global behaviour, including the longitudinal loading due to temperature expansion and contraction on the piers with pinned bearings. Adequacy of these piers was demonstrated by considering cracked behaviour in accordance with the recommendations given in BS EN 1992-1-1 clause 5.8.7; without considerations of cracking, the piers were stiffer, attracting too much load and their resistance was exceeded.

Solution for bearing replacement – corbels and steelwork

General requirements

The bridge is a Category A listed structure and is highly visible, so all parts of the strengthening work to facilitate bearing replacement had to be in keeping with the existing structure and were subject to approval by the two adjacent planning authorities in conjunction with Historic Scotland. However, all parties accepted that the visual considerations had to be balanced with the structural and safety considerations of the work as discussed below. A key aspect of the scheme design was that it should also facilitate future bearing replacements, should they become necessary.

Scheme outline

Replacement of the bearings necessitated jacking up the box girders to release the existing bearings, but there was insufficient space for jacking at the pier tops and no suitable location on the box girders to apply the jacking forces. The solution chosen was to add corbels to the tops of the piers to provide adequate space for the jacking equipment and to add jacking stiffeners to the box webs. Corbels were selected because they provided a permanent solution for bearing replacement, whilst also being more economic than options involving temporary propping. The corbels are discussed in later sections of this paper. In addition, the design needed to maintain the articulation of the bridge throughout the replacement process, which meant providing longitudinal fixity at rocker bearings and a controlled release of unintended force at roller bearing positions.

A key feature of the jacking scheme was the use of four jacking points at each box girder support. This was necessary for a number of reasons including lack of available room at the pier top in line with the diaphragms (because of the limited available clearance to the permanent moving maintenance gantry) and the presence of cross girders in line with the diaphragms limiting the available height for jacking stiffener addition in line with existing diaphragms.

A typical sequence for a bearing replacement is as follows:

a) Install internal steel strengthening

b) Install jacking stiffener

c) Cast new corbel

d) Install restraint system

e) Install jacks and jack up to remove load from bearing

f) Remove existing bearing

g) Repair concrete under bearing as required

h) Position new bearing

i) Lower box and transfer load to new bearing.

As the construction sequence was complex and the Designers'

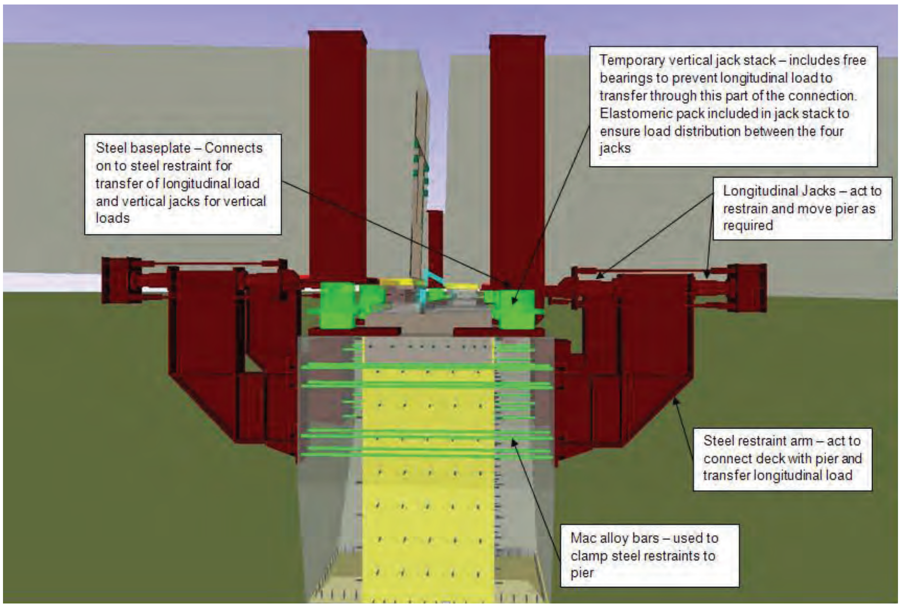


Figure 9. Scheme features in VR model

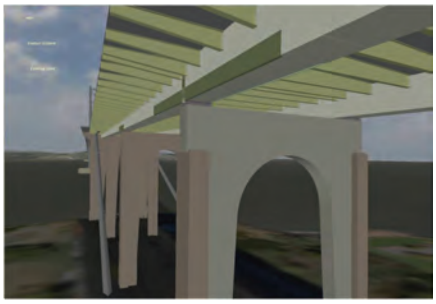
Risk Assessment identified non-compliance with it as a significant risk, a virtual reality (VR) model was used to highlight the different construction stages in the project in order to minimise the risks of errors being made in the construction sequence. This followed successful use of a similar model on a previous bearing replacement project[2]. The main features of the bearing replacement scheme are shown in **Figure 9**, extracted from the VR model.

Concrete corbels

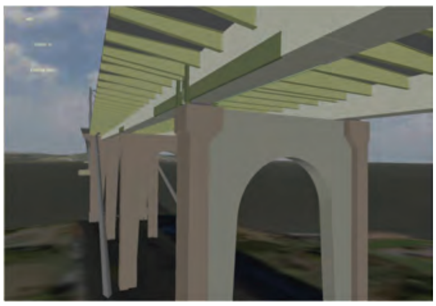
The concrete corbels were designed to be permanent extensions to the existing pier and served two purposes:

- To provide adequate space to position jacks
- To facilitate installation of permanent bursting reinforcement in the pier tops.

The corbels had to carry the load from the temporary jacks to allow the permanent bearings to be replaced whilst also allowing the pier top concrete to be broken out and recast where needed to rectify the deterioration. For this reason, the corbels were cast 300mm below top of pier level so that pier top breakout would not undermine the new corbel reinforcement. Space and aesthetics were the major constraints in the design of the corbels. Their width in the longitudinal direction of the bridge was dictated by the need to tie in with a pre-existing concrete



(a)



(b)

Figure 10. Corbel addition to tie in with existing verical pier feature, (a) Before corbel addition, (b) After corbel addition

feature extending up the sides of the piers as shown in **Figure 10**. Their width perpendicular to the bridge span on its outer face was limited by the clearance to the existing maintenance gantry.

A typical final installed corbel is shown in **Figure 11**.

The corbels were designed using strut and tie models of the generic form shown in **Figure 12**, which necessitated the anchoring of new reinforcement into the existing pier. This required holes to be drilled into the existing structure and new reinforcement resin grouted in. **Figure 12** shows typical reinforcement resin anchored through the thickness of the pier prior to drilling for the addition of the pier transverse reinforcement. This transverse strengthening reinforcement extended up to five metres into the pier because, in addition to functioning as part of the corbel, it was also designed to replace the existing transverse top mat of reinforcement while this was exposed during the repair work of the concrete at the top of the pier. The corbel reinforcement also provided bursting reinforcement for the permanent condition, which had been found by the assessment to be deficient.

It was accepted that some bars would inevitably be damaged during coring operations, so a coring protocol was developed in advance of construction to set out the numbers and locations of bars that could be damaged without remedial work being required. This allowed site staff to make the decision on what to do when an existing bar was hit and to update their strategy for the remaining cores.

The main reinforcement in the corbel needed to pass through the full width of the pier and be fully anchored at each end. The bars could not be detailed with a bend at both ends as they could not then be threaded through the core hole.

Figure 11. Corbel addition on site

Corbel width was very limited and, at some locations, there was inadequate room to couple onto the end of the straight bar with a bent bar to form the bar anchorage. In these locations, the bars were fitted with end plates via a threaded connector. On each side of a pier at these locations, main bars were alternately anchored by reinforcement bend and by end plate as shown in **Figure 13**. This detail suited the corbel strut and tie arrangement well but could only be used in areas of no or low fatigue due to the relatively poor fatigue performance of threaded coupling devices. Some limited fatigue testing was carried out on the T-head connection to ensure that it was equivalent in performance to the S-N curve provided in BS EN 1992-1-1 for "splicing devices", which was adequate for the fatigue stresses in this region.

Ahead of construction on site, a full scale mock-up corbel was constructed to trial the proposed self-compacting concrete and to check the bond at the interface with the existing pier. This highlighted some improvements to make to surface preparation, concrete mix and initial reinforcement layout, proving that the trial was a valuable undertaking.

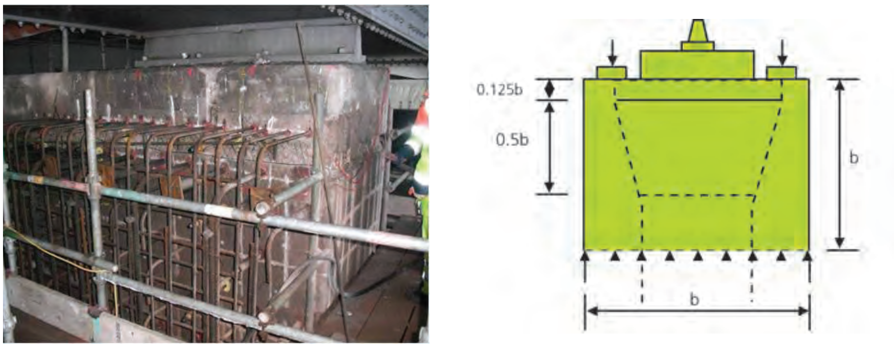


Figure 12. Typical corbel reinforcement and corresponding strut and tie model

Figure 13. Bars with T-head connection

Steelwork

To minimise the hazards associated with working in the confined space environment of the steel box and to avoid damage to the extensive existing services within the box, the main components of the strengthening needed for jacking were added to the outside of the steel box girder. This comprised single–sided box section jacking stiffeners (**Figure 14**) similar to those developed in reference 2 where the design methodology is described. A number of alternative arrangements were provided for these box stiffeners to give flexibility in the location of the bolted attachments to the webs. Weathering steel was used for the jacking stiffener because of the lack of access to its internal surfaces. The only internal steelwork strengthening was additional bolted longitudinal stiffeners (to strengthen the webs before drilling the holes for the jacking stiffeners) and some additional plating to existing longitudinal stiffener angles (required to prevent torsional buckling). The new longitudinal stiffeners were rolled steel angles connected to the webs through one leg. Welding was not used for the connections because of concerns over the ability to weld to the existing steel without laminating it and because of the confined space environment inside the boxes.

The steelwork design was carried out in accordance with Eurocodes, specifically:

- BS EN 1993-2 (Bridges)
- BS EN 1993-1-5 (Plated structural elements)
- BS EN 1993-1-8 (Design of joints)

At the end supports, the services were found not to be ducted where they passed through access holes in the box webs and diaphragms. The risks of carrying out work adjacent to these services, comprising fibre optic cables, was considered too great so it was decided to move the new longitudinal stiffeners to the external faces of the webs at these locations.

Strengthening of the existing longitudinal web stiffeners on the internal web faces was also replaced by additional new longitudinal stiffeners on the external web faces. Moving the stiffeners to the outside of the box in this way increased the steelwork quantities but improved buildability as the plates did not need to be brought into the box. External stiffeners were considered elsewhere but proved to be less convenient because of the difficulties of getting continuity across the jacking stiffeners at intermediate piers, as was needed in the design.

Figure 14. Typical box jacking stiffeners

Restraint systems

The articulation of the bridge needed to be maintained during bearing replacement. Since the longitudinal forces at fixed bearings were too large to take in shear on the jacks, separate temporary restraint systems were provided to connect the bridge superstructure to the substructure during the bearing replacement process. The form of the restraints varied according to the location but at the intermediate piers, longitudinal restraint was provided by four steel brackets positioned in pairs each side of a box, one pair either side of the pier, anchored to the pier using Macalloy bars passing through holes in the pier. Transverse restraint (to the smaller forces from wind and skidding loads) was provided by the jacking stacks themselves. This required the use of a guided temporary bearing on one of the jack stacks at a box support location to resist the shear, with free bearings at the other jack locations. Similar systems were designed for the abutments and side towers; these are not described here.

The purpose of the restraint system at rocker bearing locations was to fix the bridge piers to the superstructure throughout the bearing replacement whilst the fixity of the permanent bearings was temporarily released. The steel brackets at intermediate piers were of the form shown in **Figure 15** with a horizontal jack connecting the top of each restraint to the adjacent jacking stiffener base plate. The jacks allowed the differential horizontal movement between the superstructure and substructure to be controlled under the small deflections occurring in the steelwork itself. The system in **Figure 15** was the arrangement for Pier S3 where two simply supported box ends landed on a shared pier and hence an additional set of jacks and tie bars was provided so the restraint system could both push and pull. These additional features were unnecessary at continuous box locations as the restraints only needed to carry forces in one direction. The horizontal restraint jacks were engaged before lifting the superstructure and remained engaged during the bearing replacement until the girders were lowered down onto the new bearings.

The restraint systems at roller bearing locations were similar but were provided only to control the release of force locked into the roller bearings. It was known such forces existed from earlier pier monitoring. Lifting of the superstructure under these conditions could have caused a sudden release of horizontal force and a springing back of the pier in a potentially dangerous sudden movement. The provision of jacks allowed this force and movement to be released in a controlled and safe manner.

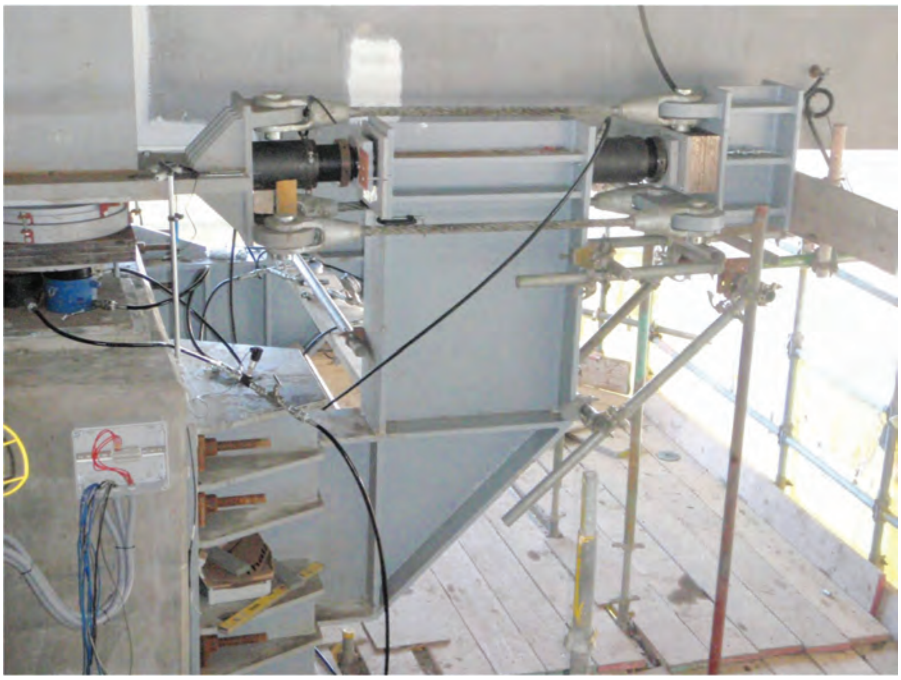


Figure 15. Typical temporary longitudinal restraint

In all cases, generous allowance for eccentricities was made in the design of the steelwork because aligning the temporary restraint brackets, whose position was fixed by the locations of the core holes for the Macalloy bars, with the jacking stiffener base plate stiffening had significant tolerance.

As a result of this provision of temporary fixity, no limit was set on the number of piers which could be jacked up at any one time.

Design of jacking system

The jacking scheme utilised two jacking points per bearing at each abutment and side tower and four jacking points at other pier locations. The jacking system at each jacking point comprised three jacks supporting either a transversely guided or free pot bearing; the initial concept utilised only one large jack at each jacking point but this was modified once on site to suit the equipment available – **Figure 16a**. This ensured that the relatively small transverse forces from the deck (due to wind and skidding) were taken through the guided bearings and then through the jacks to the piers. Longitudinally, the bearing transmitted no loading to the jacks (other than from friction at the roller bearing piers). Longitudinal forces at rocker piers were carried by the temporary longitudinal fixities described previously because the shear forces were too large to be carried by the jacks and additionally, the uneven bearing pressure resulting on the jacks from the longitudinal force would have been too great for the concrete.

At locations with four jacking points, the load at each jacking point after lock off was potentially very uneven under live loading due to the torsion attracted to the boxes and the tendency to uplift on the "back span" jacks where only one span was loaded. To prevent uplift at a jack position and to distribute the loads between them more evenly, thus minimising the strength requirements for jacking components and box strengthening, the stiffness of each jacking system was reduced by incorporating an elastomeric pad between the jack and temporary bearing within a further bespoke bearing capable of transmitting transverse shear. The assembly formed by these three components was referred to as the 'jacking stack' and is shown in **Figure 16b** undergoing a load test.

The determination of the required stiffness for each jacking stack was carried out by linear finite element analysis of the entire south viaduct. Box girders, transverse beams, diaphragms, internal stiffeners and concrete slab were all modelled with thick shell elements, while the jacking stiffeners were added as beam elements. **Figure 17** shows the distribution of forces between the four jacking points as a function of elastomeric pad stiffness at pier S4. The jacking sequence included the jacks being hydraulically linked for dead loads and subsequently locked-off for live loads. A maximum stiffness of 10^7 kN/m was needed to prevent uplift at a jack. The upper line shows the total force in all the jacks. It can be seen that below a stiffness of 10^5 kN/m, the forces in the jacks were very even but the flexibility is such that load was shed to adjacent piers, which was undesirable. The vertical flexibility under live load would also have compromised the ability to grout below the new bearings. Full 3D modelling was essential to model the redistribution of torque also caused by the softer bearing support; this slightly increased the stress resultants in the support cross girders.

The final design stiffness of the jack stack was tuned to a value between 10^6 kN/m and 10^7 kN/m so that uplift could not occur under any load combination and load was shared economically between jacks, but the vertical displacement under load would not be so great as to disrupt grouting beneath the permanent bearings. That latter was set at 0.2mm and this proved to cause no problem during grouting.

Cathodic protection

The inspection and concrete test data indicated that the cause of concrete deterioration was chloride induced corrosion, possibly from the initial use of de-icing salts in the first twenty years of the bridge's life. The chloride ion content at the majority of pier tops and both abutments was in excess of the threshold value of 0.3% by mass of cement and the 'diffusion' calculations suggest that the critical level at the depth of reinforcement would be reached in the next 5 to 8 years. Action was therefore required to arrest continued deterioration. Various repair options were considered but the application of cathodic protection (CP) to the piers, side towers and abutments, together with minimal concrete repairs solely to the delaminated areas, was evaluated to be the most cost-efficient solution for the sub-structure.

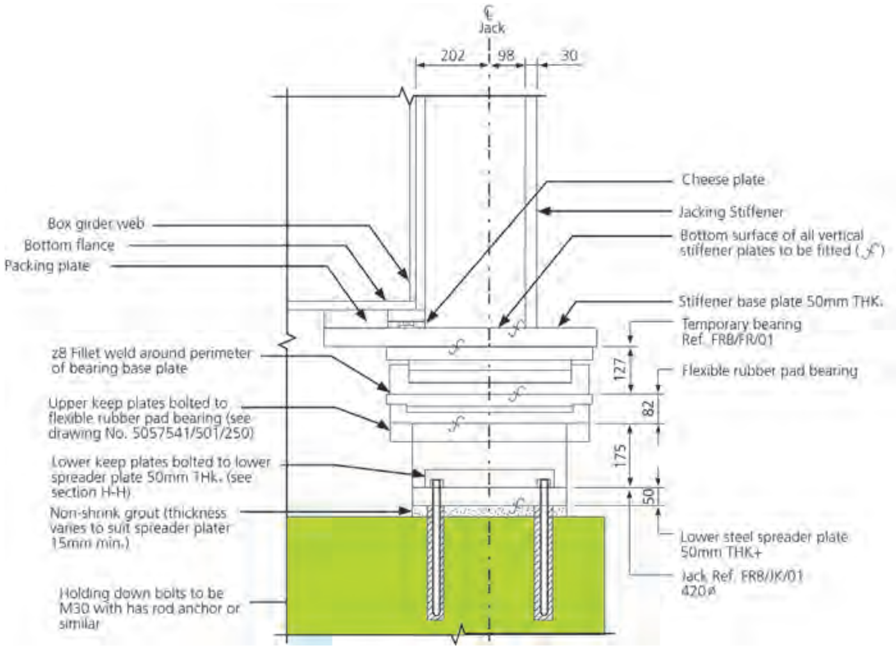
An impressed current (ICCP) system was selected with design undertaken in compliance with the recommendations of references 3 to 5. Three types of anode systems were used in the design: titanium mesh based anode to be installed at the interface of concrete substrate and new corbels; titanium ribbon based anodes to be installed within slots cut into the concrete cover; and titanium ribbon based discrete anodes designed to be installed within holes drilled into the structure. Reference electrodes for monitoring the performance of each anode zone were also included.

The area covered by each anode zone was designed to be independently powered by an integrated power supply and data acquisition unit, which will provide the facility for controlling the voltage and current to each zone; and monitor/control the performance of the CP system. All units are installed within two networks (one for each approach viaduct). On each network all units were detailed to be operated from a single main control unit either locally on site or remotely via a modem connection.

Conclusion

The replacement of the bearings on the Forth Road Bridge is a very significant undertaking because of the lack of provision for this eventuality in the initial bridge design. The solution developed, involving modification of the structure to provide permanent jacking points on superstructure and substructure, now makes any future bearing replacement or refurbishment a much simpler, quicker and cheaper operation. The permanent modification to the structure, by addition of corbels to the piers and external stiffening to the box girders, was carried out sympathetically to the original design and under the scrutiny of Historic Scotland.

A number of measures were taken to minimise the cost of the modification works and could be considered for future bearing replacement schemes. These included the use of Eurocodes for the assessment of the existing structure and design of the new works, the use of external stiffening to improve buildability and the use of elastomeric pads incorporated in the jacking system to give a better distribution of jack loads after lock off and hence reduced demand on the new stiffening and concrete corbels.



(a)

(b)

Figure 16. Jacking Stack, (a) Concept with single jack, (b) Final three jack system undergoing load test

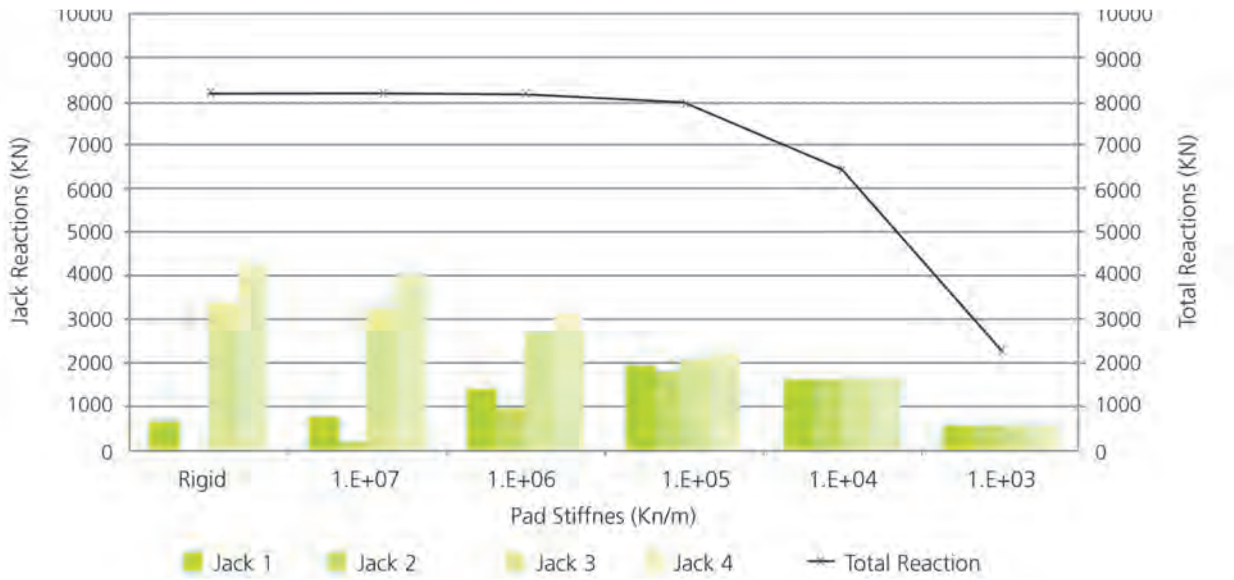


Figure 17. Results of sensitivity analysis on elastomeric pad stiffness

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Design of cross-girders and slabs in ladder deck bridges

Abstract

Deck slabs in ladder decks span longitudinally between transverse cross-girders and the primary function of these cross-girders is to support the deck slab. The girders may however need to perform the secondary function of preventing the slab from buckling in compression. The concrete deck slab of a ladder deck can have a very large transverse span between main girders. This large unsupported width can lead to buckling of the slab in compression unless it is prevented from doing so by transverse girders. Where transverse girders are required to prevent buckling of the slab, they need to be designed to be both stiff enough and strong enough to perform this function in addition to resisting vertical loading. If the spacing of the cross-girders is large, it is still possible for second order bending moments to develop in the slab under the effects of global compression. This paper sets out guidance on the limiting spacing of main girders and cross-girders to avoid consideration of second order effects and also the means of determining second order effects in slab and cross-girders when this becomes necessary.

Introduction

In ladder deck bridges, such as those illustrated in **Figure 1**, the deck slab performs two principal functions: it carries permanent and variable loads in plate bending, spanning between the transverse cross-girders and between the main girders; and it acts as the flanges of the composite main girders and cross girders. In sagging regions, the deck slab is in compression longitudinally and its stability against plate buckling needs to be verified. The cross girders provide regularly spaced out-of-plane restraint to the slab, in addition to their role in supporting the deck slab between the main girders; the stiffness of the restraint that these cross girders provide must be considered and any forces arising from the restraint that they provide must be included in their design. Even when restrained by the cross girders, the slab itself is relatively slender, in terms the ratio of cross girder spacing to slab thickness, and second order effects need to be considered in its design verification.

The rules in Eurocodes 2, 3 and 4 do not explicitly deal with these design considerations, although there are rules that may be applied. The purpose of this paper is to show how the Eurocode rules may be applied to the design of cross-girder and slab and to provide simplified factors to account for second order effects.

In hogging regions, the deck slab is in tension and there are no buckling effects to consider; there are no second order effects in the slab and no restraint forces imposed on the cross girders. Nevertheless, the dimensions of the slab and the cross girders are generally chosen the same as in the sagging regions and thus the verification in the sagging regions determines the sizes for the whole bridge.

Figure 1. Typical ladder deck geometry (prior to casting deck slab)

Structures

Plate buckling action of the deck slab

For the concrete slab to buckle under longitudinal compression, it must bend out of plane both longitudinally and transversely. If there were no cross-girders, the behaviour would be like that of a plate – see **Figure 2**. Schlaich et al[1] recommend that, to prevent any significant second order effects from occurring in the slab due to such buckling, the transverse span to depth ratio should not exceed 30; the authors of this paper consider, based on precedent, this to be a reasonably pragmatic limit for typical concrete grades up to C40/50 but this limiting ratio might be a little too large for higher strength concrete grades.

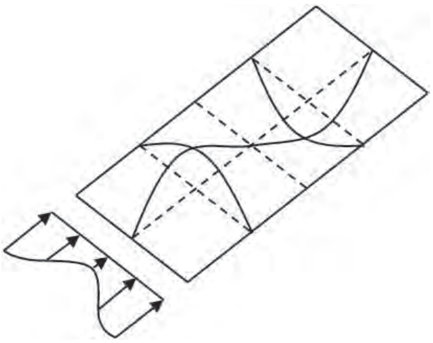


Figure 2. Slab buckling without restraint from cross-girders (plate-like behaviour)

When the slab transverse span/ thickness ratio is within the above limit, it is sufficiently stable without any additional restraint from the cross girders. Consequently, the design of the cross-girders does not need to include any restraint forces. However, in practice, when the slab thickness is typically 250mm, the above limit is equivalent to a limiting spacing of the main girders of 7.5m (slightly less if the concrete is stronger than C40/50). Most ladder deck construction places the main girders at greater spacing than this (up to about 18m) and then the slab will need to be stiffened transversely by cross-girders.

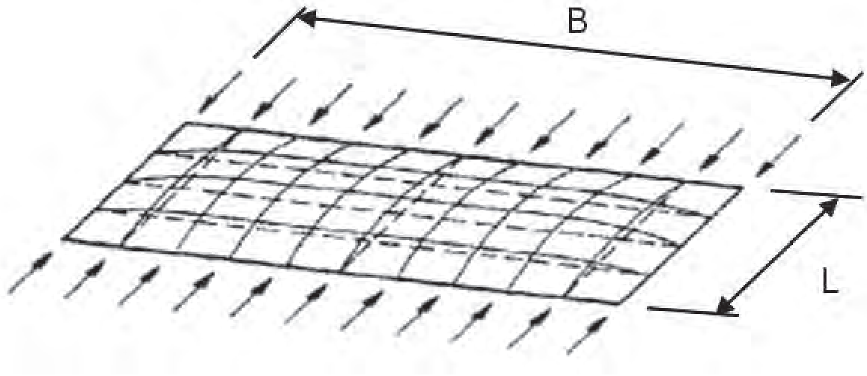


Figure 3. Slab buckling between cross-girders

Where a wide slab is stiffened by the cross girders, the mode of buckling tends toward that of simple strut buckling – the greater the aspect ratio, the closer the buckling is to strut-like behaviour – see **Figure 3**. To consider whether second order effects in the slab are significant and, if they are, to evaluate the design effects, it is conservative to ignore plate action (i.e. buckling of a rectangular plate supported on four sides) and to treat the slab as a column, with a half wavelength of buckling equal to the span between cross girders.

Limit on cross-girder spacing to avoid considering second order effects in the slab

In PD6687-2[2], clause 6.1 provides a simple rule for externally prestressed members that second order effects need not be checked when the spacing between deviators does not exceed 10 times the section depth. This simple rule is considered appropriate for the slab of a ladder deck in the sagging region. The rule is less conservative than the simple rule for isolated compression members in clause 5.8.3.1 of EN 1992-1 but even so, in practice, with a 250mm slab thickness the consequent limit of 2.5m on cross girder spacing is much lower than the preferred spacing of between 3m and 4m. In practice, therefore, second order effects in the slab will need to be considered.

Second order effects in deck slab

Where second order effects in the compressive region of the deck slab need to be accounted for, this can by achieved using EN 1992-1-1, treating the slab as a simple compression member. Three methods are given for determining second order effects in reinforced concrete compression members, in clauses 5.8.6, 5.8.7 and 5.8.8. The clause 5.8.6 method requires non-linear analysis and is thus not readily applicable; the clause 5.8.8 method is not suitable for members with transverse load, such as the deck slabs; thus the clause 5.8.7 method is used.

The clause 5.8.7 method uses a nominal stiffness of the compression member that takes account of the level of stress in the member, the concrete strength and the member slenderness. In doing so, it is important to use the actual design compression force in the slab (to avoid over-conservatism) and the eccentricity of that force from the centre of the slab (as this causes slab moment and curvature which will also be amplified by the compression). Generally, the slab force is unlikely to be at the level of its design compression resistance, as the design situation will be with combined longitudinal and transverse loading on the slab (and thus there will be significant local bending moments to be considered). Additionally, Eurocode 4 requires shear lag to be considered at ULS and this will normally mean that the design compression resistance is not that of the gross slab. Even if the full width of the slab can be mobilised, its large width makes it unlikely that it will be fully utilised in compression.

The clause 5.8.7 method derives a magnification factor that is applied to the first order moments. The factor is expressed in clause 5.8.7.3 as:

$$M_{Ed} = M_{0Ed}\left[1+\frac{\beta}{(N_B/N_{Ed})-1}\right]$$

Where

M_{0Ed} is the first order moment, which should be taken as $M_{slab}+N_{Ed}(e+w_0)$

N_{Ed} is the design value of the axial force in the slab, coexisting with M_{Ed}

N_B is the buckling load, given by $N_B=\pi^2EI/L^2$

M_{slab} is the moment in the slab due to transverse loading

e is the eccentricity of the axial force relative to the centroid of the slab

w_0 is the geometric imperfection in the slab (= slab span/400, based on the value of θ_o = 1/200 in EN 1992-2, clause 105 and the UK NA)

EI is the nominal stiffness, given by clause 5.8.7.2.

Second order effects may be ignored if they are less than 10% of the first order effects, according to clause 5.8.2(6).

The calculation of stiffness EI according to clause 5.8.7.2 depends on a number of parameters, including the area of reinforcement, the effective creep ratio φ_{ef} and the relative axial force n. The relative axial force is a significant parameter: its value is defined as N_{Ed}/A_cf_{cd}. As noted above, the value of N_{Ed} should be based on the actual slab force calculated allowing for shear lag and the crushing resistance A_cf_{cd} should be based on the whole gross width of slab ignoring shear lag.

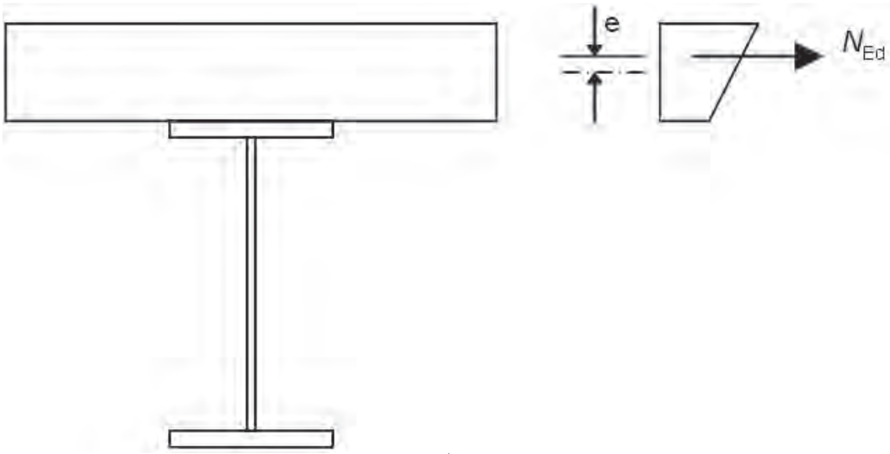


Figure 4. Eccentricity of slab force

(Note that f_{cd} should be calculated in accordance with EN 1992-2 rather than EN 1994-2). The effective creep ratio φ_{ef} is determined in accordance with clause 5.8.4(2) of EN 1992-1-1. A value of zero implies all the slab moment is due to non-quasi-permanent loading.

Values of the magnification factor for a range of n values are given in **Figure 5**, for a 250mm thick slab spanning 3.0 and 4.0m. It will be noted that the curves for 4m spacing have been curtailed at n = 0.6; beyond this point the curve changes gradient according to clause 5.8.7 because the parameter k_2 reaches a limiting value of 0.2. Clearly there would be no step change in reality and calculations beyond this value of n could either be conservatively performed to clause 5.8.7 or non-linear analysis used.

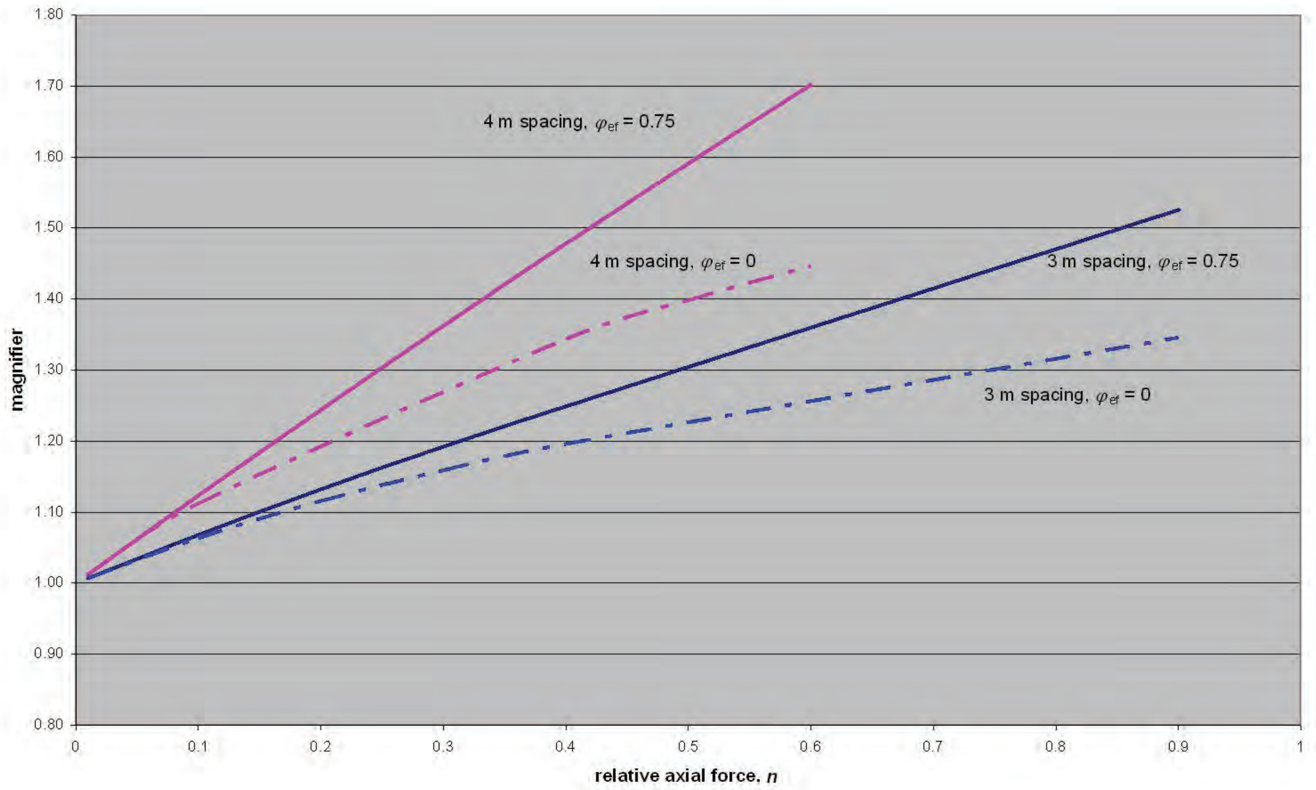
The situation where the main beam spacing is not large relative to the cross girder spacing and the behaviour is partially plate-like, rather than solely column-like, could be accommodated in the process of determining N_B by substituting the buckling load for a plate, $N_{cr,p}$ of side B equal to the main girder spacing and length L between cross girders.

$$N_{cr,p}=\frac{k_\sigma\pi^2Et^3}{12(1-v^2)B}=\frac{k_\sigma\pi^2EI}{(1-v^2)B^2}$$

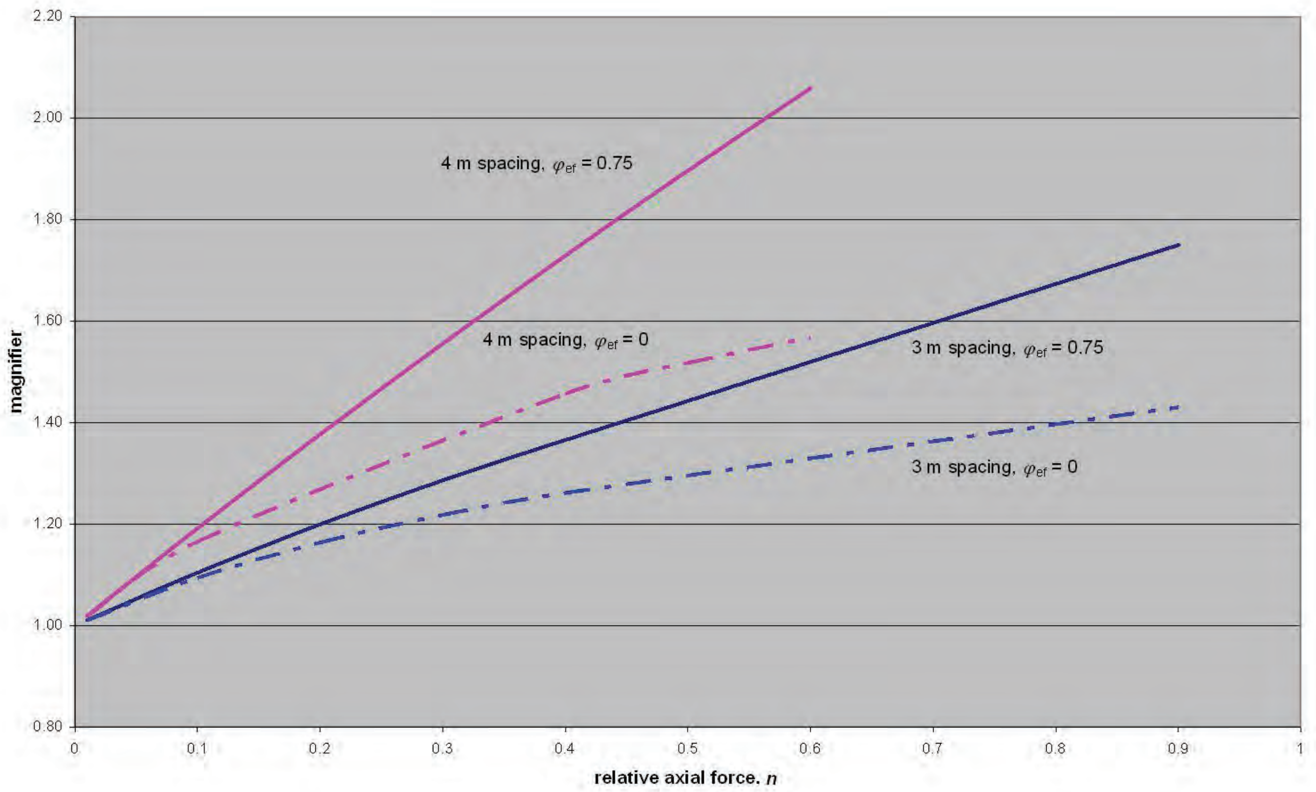
in which $k_\sigma=(B/L+L/B)^2$

and $v=0$ for cracked concrete.

In this case, EI will not be constant in the two directions, even if the reinforcement provision is the same, because the transverse direction typically has zero compressive force. A conservative approach would be to take the smaller value of the cracked stiffness derived in accordance with clause 5.8.7.2 in either the longitudinal or transverse direction. This is, however, likely to be overly conservative since, for normal slab support aspect ratios, the behaviour is dominated by the stiffness in the longitudinal direction, so the slightly unconservative approach of basing EI solely on the longitudinal direction will be acceptable. In addition, the moment amplification method itself becomes more conservative the more plate-like the behaviour becomes as, for plate-like behaviour, the elastic critical buckling force is not an upper bound on strength, whereas it is for column-like behaviour. The benefit of using the plate critical force will be small but will increase as the ratio B/L reduces. Utilising this approach is likely only to be of benefit in the



(a)



(b)

Figure 5. Magnification factor, to allow for second order effects (250 mm slab, C40/50), (a) H20 bars at 150 mm centres, top and bottom, (b) H16 bars at 150 mm centres, top and bottom

situation where a small overstress was produced using the column buckling approach.

An example of the calculation of the magnifying factor, for both simple strut buckling and for plate panel buckling is given below.

Assume a slab thickness 250mm, cross girder spacing 3500mm and main girder spacing 12000mm. The slab concrete is C40/50 and the reinforcement is H16 bars at 150mm centres (take distance between top and bottom bars = 160mm, symmetrically placed about η the slab centroid). Take $\varphi_{ef} = 0$ and $= 0.20$ and $f_{cd} = \alpha_{cc}\, f_{ck} / \gamma_m = 22.7$ MPa with $\alpha_{cc} = 0.85$ and $\gamma_m = 1.5$.

Properties per metre width of slab
$I_c = 1.302 \times 10^9$ mm^4
$I_s = 17.2 \times 10^6$ mm^4

Material properties
$E_s = 200\,000$ MPa
$E_{cd} = E_{cm}/\gamma_{cE} = 35\,000/1.2 = 29\,200$ MPa

The effective stiffness is given by clause 5.8.7.2 of EN 1992-1-1 as:

$$EI = K_c E_{cd} I_c + K_s E_s I_s$$

$$K_s = 1$$

$$K_1 = \sqrt{f_{ck}/20} = \sqrt{40/20} = 1.41$$

$$K_2 = \eta \frac{\lambda}{170} = 0.2 \frac{3500/72.2}{170} = 0.2 \frac{48.5}{170} = 0.0571$$

$$K_c = \frac{k_1 k_2}{(1+\varphi_{ef})} = \frac{1.41 \text{ x } 0.0571}{(1+0)} = 0.0805$$

$EI = 0.0805$ x 29200 x 1.302 x 10^9 + 1 x 200000 x 17.2 x 10^6 = 6.50 x 10^{12} Nmm^2

The design value of axial load per metre width is:

$N_{Ed} = \eta A_c\, f_{cd} = 0.2$ x 250000 x 22.7 x 10^{-3} = 1133 kN

The critical force per metre width is:

$N_B = \pi^2\, EI/L^2 = \pi^2$ x 6.50 x $10^{12}/3500^2$ x 10^{-3} = 5237 kN

Conservatively assume that the distribution of the first order moments is parabolic, this gives a β value of 1.0.

The moment magnification factor is:

$$m = \left[1 + \frac{\beta}{(N_B/N_{Ed}) - 1}\right] = \left[1 + \frac{1}{(5237/1133) - 1}\right] = 1.28$$

Alternatively, N_B can be replaced by $N_{cr,p}$

$k_\sigma = (B/L + L/B)^2 = (3500/12000 + 12000/3500)^2 = 13.84$

$v = 0$ for cracked concrete.

$$N_{cr,p} = \left(\frac{k_\sigma \pi^2 EI}{(1-v^2)B^2}\right) = \left(\frac{13.84 \text{ x } \pi^2 \text{ x } 6.50 \text{ x } 10^{12}}{12000^2}\right) \text{x}$$
$10^{-3} = 6166$ kN

And then

$$m = \left[1 + \frac{1}{(6166/1133) - 1}\right] = 1.23$$

This reduces the moment magnification factor.

Destabilizing effects on transverse cross girders

When the presence of cross-girders is needed to prevent buckling of the slab in compression, they must be designed for stiffness and strength so that they do in fact restrict the buckling length of the slab to the distance between cross-girders. This is achieved by designing them as transverse stiffeners to EN 1993-1-5 clause 9.2.

The stiffness of the composite transverse girder may conservatively be based on its fully cracked section properties but it will usually be adequate to use uncracked properties for the slab, as the permanent cross girder moment will usually be sagging. This sagging moment will be increased by the destabilising effect of the slab if the cross girder being checked has a net downward deflection compared to the adjacent girders. If an adjacent girder is loaded, the destabilising effect would tend to push the cross girder up and relieve the sagging moment (and does not therefore need to be considered in verifying the strength of the cross girder).

The Designers' Guide to EN 1993-2[3] gives a method of calculation to combine the destabilising effect of the deck slab and the transverse moments acting on the transverse members in the strength check.

Stiffness requirement

Based on the requirement in EN 1993-1-5, 9.2.1 the stiffness requirement may be expressed as:

$$\delta + w_{Ed} \leq \frac{B}{300}$$

where

w_{Ed} is the deflection of the cross girder under transverse load

δ is the extra deflection arising due to the compression in the slab and, in the absence of transverse axial force in the cross girder, is given by:

$$\delta = w_0' \left(\frac{\dfrac{EI_{st}}{\sigma_m B^4}}{\pi^4} - 1\right)^{-1}$$

where

I_{st} is the second moment of area of the composite section

$$w_0' = w_0 + w_{Ed}$$

w_0 is the initial imperfection which, according to EN 1090-2, Annex D1.6(5), for one cross girder level relative to the adjacent girders, is 2L/400 = L/200

σ_m is the destabilizing effect of the load in the slab, which can conservatively be taken, for the case of cross girders, as:

$$\sigma_m = \frac{2N_{Ed}}{BL} = \frac{2\sigma_c b t_{slab}}{BL} = \frac{2\sigma_c t_{slab}}{L}$$

Structures

In which σ_c is the mean longitudinal stress in the slab. This value can either be determined according to the actual design effects or to the value if resistance is fully mobilised.

Strength requirement

The destabilising load on the cross girder depends on longitudinal imperfection and its peak value (at the centre of the cross girder) is $(w_0' = \delta)\sigma_m$. Assuming that this is a sinusoidal imperfection, this slab applies an additional moment on the cross girder of:

$$(w_0' = \delta)\frac{\sigma_m B^2}{\pi^2}$$

This additional moment must be included in the ULS verification of the cross girder.

Acknowledgement

The authors thank Mabey Bridge for the supply of the photograph in **Figure 1**.

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Resolving complexity: Why systems engineering works

Abstract

In recent years the pitfalls in the design and delivery of complex systems have been well documented. The £12billion national NHS computerisation programme[1], designed to give doctors access to patients' records wherever they are in the country failed on the first installation. Consequences were potentially loss of life had patients been incorrectly identified. The procurement of Mark 3 Chinook Helicopters has resulted in the purchase of machines that may well be perfectly safe, but have so far proved impossible to certify. The contract failed to specify that software and avionics codes should be analysed according to stringent UK defence standards. The result has been that, since delivery in 2001, the Mark 3 is restricted to limited flight trials and only in cloudless conditions[2]. The overarching problem has been identified as a lack of adequate control, leading to the misconception that programme management is to blame. In reality, delivering a complex system is a multi-disciplinary undertaking of which programme management is only one aspect and complex systems require an integrated approach covering technical, commercial and management activities. A well proven methodology which provides guidance through design, delivery, operation and retirement is required. The methodology would provide coherence and facilitate specialist disciplines to work together to assure a successful outcome i.e. that the system meets stakeholder needs and is resilient to unwanted emergent properties in an uncertain environment.

This paper describes how the ISO/IEC 15288 systems and software engineering standard defining system life cycle process and work products, can be used as a framework to support an holistic solution when used in conjunction with a model based system engineering environment. The paper will show the life-cycle of a conceptual naval vessel, detailing how development progresses in an integrated and coherent way, from stakeholder needs through to requirements specification, covering the system through-life. It will also show how ISO 15288 processes are executed in a model driven environment using the UK Ministry of Defence's (MOD) Architectural Framework (MODAF), to specify a coherent solution across all lines of development (training, equipment, people, infrastructure, process, organisation, information and logistics). The integrated design repository facilitates reuse and coherence across all supporting disciplines such as safety, security, human factors and supportability.

The benefits of the approach are significant improvements in performance, cost, time and risk. Increasing the effort in the early phases of acquisition of complex systems drives out errors downstream. The author recognises that the adoption of the approach will require significant courage of budget holders to modify their spend profile. The promise of avoiding project outcomes defined in the first paragraph should provide plenty of incentive.

Note that some of the diagrams included in this paper are purposely blurred as they have been derived from MOD sources.

Introduction

The ultimate goal of the design and definition process for a system of systems or system, is to improve quality within the constraints of cost and time. What is meant by quality? The system behaves in a predictable, error-free way in every situation, predicted or not see **Figure 1**.

Consistent use of systems engineering processes significantly reduce the number of unknowns within a project at design time and therefore reduces the probability of system failure. The process begins by consideration of predictable behaviour. The discovery of unknown states is achieved through communication with stakeholders with the objective of uncovering operational and support scenarios, identifying additional system of systems dependencies and further scoping the system of interest. Despite a rigorous approach, unknowns will still remain in the form of unspecified states due to environment events that cannot be predicted at design time. Specific mitigation solutions cannot be defined at design time and will generally be discovered through system usage in a complex environment. In most cases, they represent the largest set of unknowns. The mitigation of risk emerging within these states is to apply advanced engineering skills to design flexible, agile systems which have a high degree of run-time robustness. There is, of course, a trade-off between quality and cost.

Reducing complexity

There are number of contributing factors influencing the quality of delivered systems. There is no doubt that systems are becoming more complex. Our most successful approach is to reduce complexity but how? An example of complexity is the maritime environment which

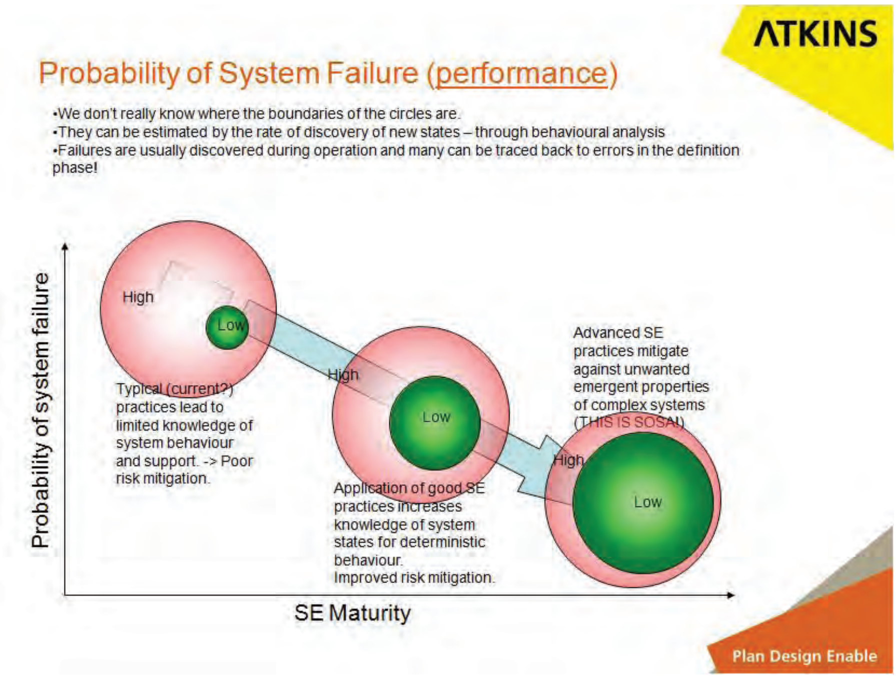

Figure 1. Probability of system failure

places even greater constraints on the systems we design and operate. We are frequently asked to design a system which must operate in a hostile environment, be capable of war fighting, providing humanitarian aid, transporting goods and people, and provide hotel facilities for thousands of people – all in one system!

Product breakdown

The traditional approach to complexity reduction is product breakdown. The system is partitioned into physical sub-systems and each one approached inevitably in isolation. This creates a stove-piped work breakdown structure at the outset, where specialist disciplines use their own techniques (which are actually similar) and develop their own language and cultural identity. The majority of engineers and managers are comfortable with this approach as the artefacts are tangible and well scoped. Customers, however, are not well served by this approach. Consider a product breakdown for a surface vessel, see **Figure 2**.

The stakeholder requirements map is not clear because requirements are defined in terms of the physical components of the system and this decomposition does not support a clear set of needs for the business and end user/maintainer. User stakeholders need to be proficient in a number of specialist domains to understand how disparate systems will come together to form their system needs.

Effect on the development process:

- Stove piped teams working in isolation with ill-defined integration planning
- How are the needs of the stakeholders validated? Delivery of a propulsion system in itself does not deliver any complete end-to-end system function!
- The systems that are being delivered are in specialist domains, which cultivates communication barriers to stakeholders
- There is an inherent risk of high integration cost.

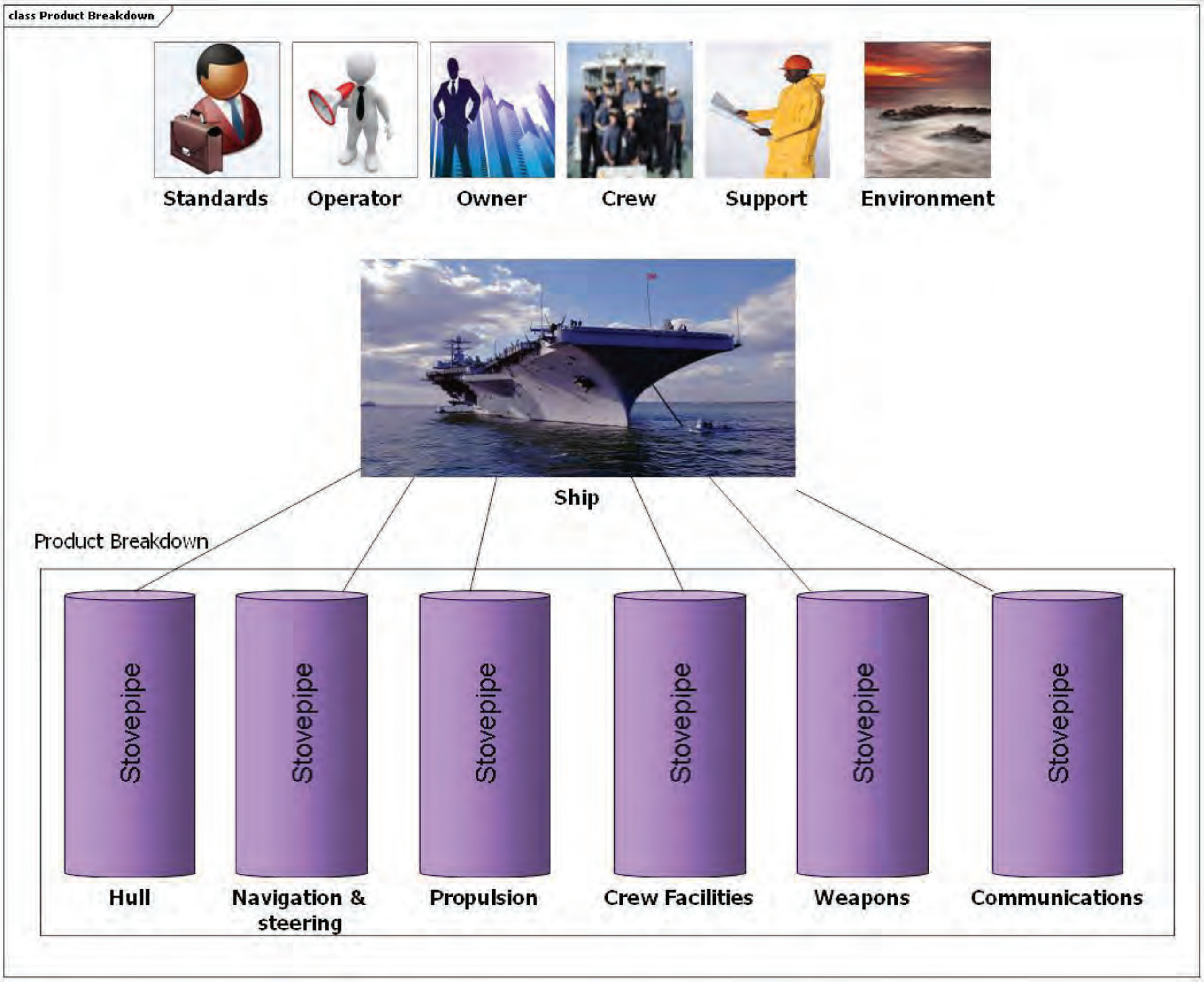

Figure 2. Product breakdown

Functional breakdown

A more cohesive way of reducing complexity is the use of functional breakdown see **Figure 3**. The system definition process continues with behavioural analysis of the system at a number of architectural levels of abstraction.

Stakeholders have a clear communication of the services that the system will provide and how their functional needs will be met.

Effect on the development process:

- Behavioural analysis provides a functionally cohesive definition of the system across all domain areas which can be decoupled

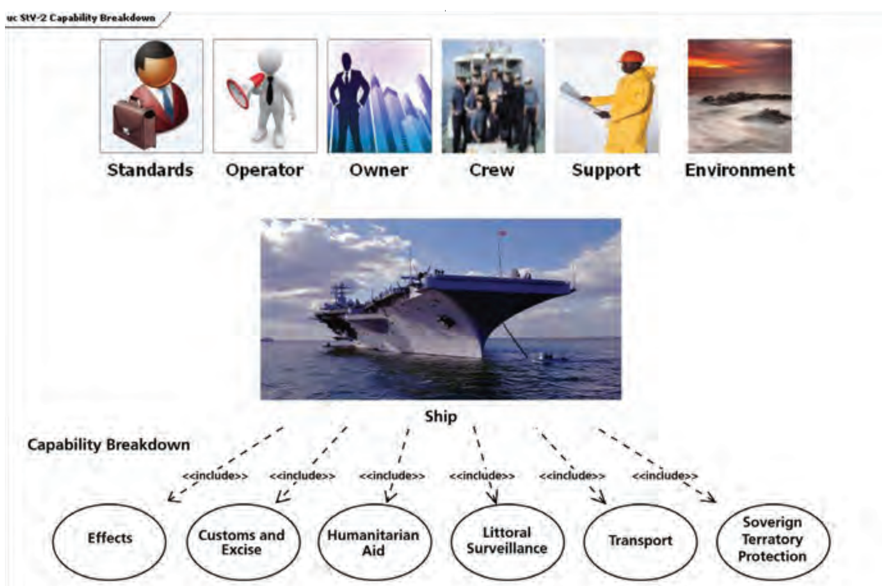

Figure 3. Strategic View-2 (StV-2) Capability / Functional breakdown

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from architectural and solution constraints, further reducing complexity early in the development life-cycle.

- Dependencies on services from other specialist domain areas can be specified together with interface requirements. This necessitates the establishment of a language that all stakeholders can understand.
- Functional analysis significantly reduces the probability of system failure. Behavioural definitions, clearly communicated to engineers and stakeholders, reduce the risk of omission of functional blocks and improve the success of integration of disparate systems. This ultimately reduces the probability of operational failure.

Non-functional analysis

What of the constraints imposed by commercial and technical business goals? The functional breakdown approach deals with these by leaving their impact as late in the design process as possible. This allows a number of unconstrained designs to be contrasted and the best selected.

Although constraints may be applied late in the process, their impact must be considered at the outset, with a clear compliance plan defined for each.

Advantages:

- Decoupling from behavioural analysis yields a number of candidate solutions for consideration
- Reuse of legacy systems can be planned early and the impact (risk and cost) assessed
- Stakeholders have a degree of confidence that constraints will be met.

Total cost of ownership

The proposed functional approach will produce a coherent definition of the operation of the system and therefore provide a better and more thorough analysis with justification for selection of physical solutions. From the enduring statement of requirement in the common language, a number of physical solutions can be considered and the selection of the "winner" justified against requirement compliance, implementation risk and cost.

Procurement costs are often the sole consideration in the selection of equipment. For most projects, particularly low volume high cost maritime systems, the total cost of ownership has significant financial commitments in the operation and disposal phases. These costs can, and should, be considered in the solution decision making process. The behavioural definition of the operation of a system should be extended to support and disposal. Sustainment requirements may be derived by defining the support scenarios for the system. This provides key decision makers with a more comprehensive view of the through-life costs.

Systems engineering lifecycle

ISO/IEC 15288-2008 is the international standard on Systems and Software Engineering. The Lifecycle Processes advocate the use of integrated agreement, business, project and technical processes to achieve effective and coherent system definition throughout its life.

Definition processes

The system definition processes on the left hand of the vee (the systems engineering life-cycle process are typically drawn as a V diagram) are crucial to the through-life success of the system by defining and meeting stakeholder and business requirements. The life-cycle definition stack is shown in **Figure 4**.

- Enterprise Analysis - Analysis of the system of interest within the context of the rest of the enterprise. The output from this phase is the contextual definition of the system, its scope and dependencies on external systems and agencies. Analysis of the black box system produces a set of coherent stakeholder/user requirements.
- System Definition - Implementation-free statement of the logical components of the system, their architecture (static definition of component content and the associations between components) and behaviour (interaction between components to deliver a function or capability). This is an enduring design which remains stable throughout solution selection. This phase provides a common language which is used to communicate with all speciality disciplines and the system stakeholders.
- System Development - Solution architectures and specifications of physical components which satisfy the System Definition. Selection and justification of best solution architecture. The output from this phase is a set of configuration items with their specifications and interfaces.

System engineering technical process design patterns

There are at least three core systems engineering design patterns which may be applied throughout the definition life-cycle. The patterns follow the same activity flow but act on information at different levels of abstraction. Refer to the outputs of the processes in **Figure 4**.

- Behavioural analysis – This provides a definition of system operational, support and disposal scenarios. Several disciplines use these scenarios to assess the system from their specific viewpoint. Scenario analysis provides the coherence which binds together the functions of architectural components at any level of abstraction. The inherent communications properties of behavioural analysis and their dissemination among stakeholders, results in a significant increase in the understanding of the system and its environment. This in turn reduces the risk of error propagation through the definition process and therefore failure of the delivered system. The information generated from scenario analysis is augmented by each specialism and managed in a common design repository.

- Architecture design – This activity defines the active and passive elements of the system and shows their associations and communication paths. Good architectural design relies on knowledge and experience of fundamental techniques such as:
    - Abstraction
    - Object oriented design
    - Coupling and cohesion
    - Separation of concerns
    - Open systems and open interfaces.
- Quality – Design review activities throughout the life-cycle to reduce the number of errors "leaking" into the next phase of development and therefore increasing cost and reducing quality.

Figure 5 shows how the process design patterns bind the engineering disciplines to produce a coherent, high quality solution. The integrated solution is founded on a common design repository.

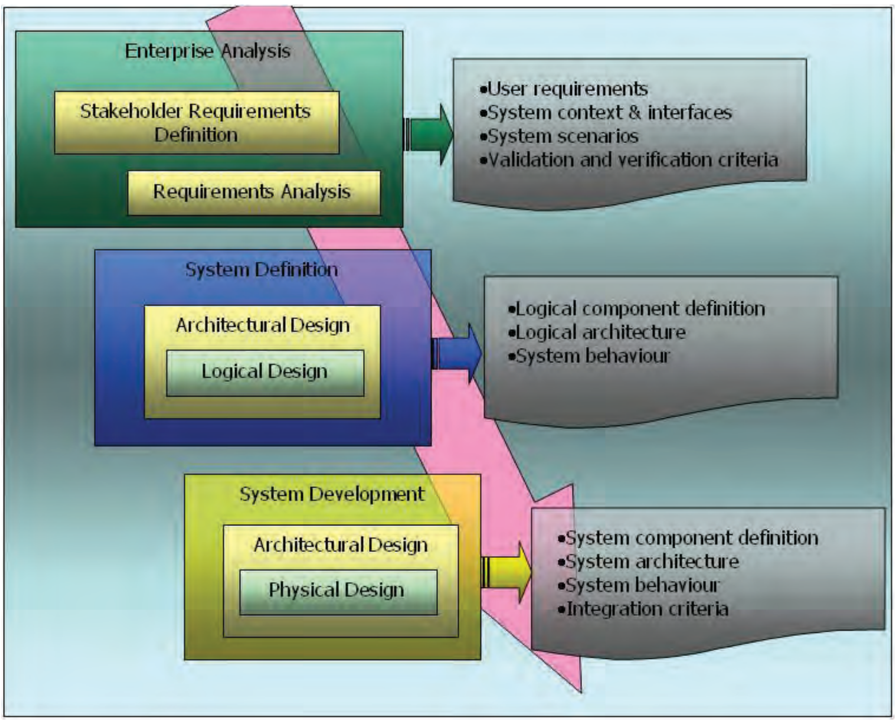

Figure 4. System definition stack

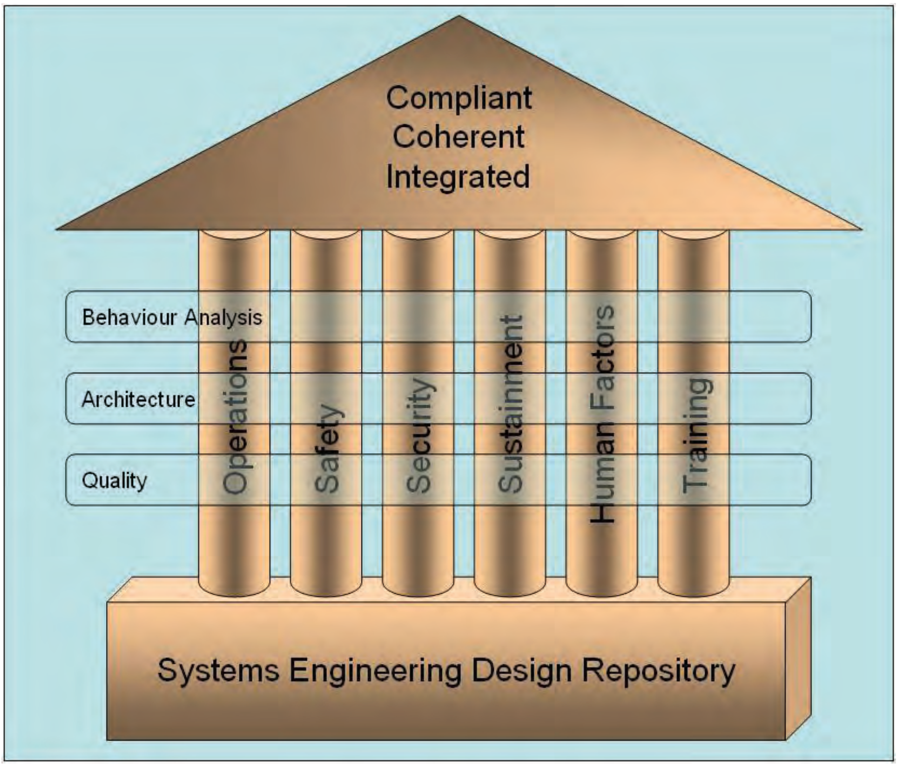

Figure 5. Integrated system viewpoints

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Model driven engineering and MODAF

Model Based Systems Engineering was developed as an approach to reducing the complexity of Software systems. It focuses on the definition of models of abstract components which describe the operation and configuration of a software system. The stated advantages[3] are:

- Maximise compatibility between systems
- Simplify the process of design
- Promote communication between developers and stakeholders.

The Object Modelling Group (OMG) initiative on Model Driven Engineering (MDE) or Model Driven Architectures, not surprisingly, has much in common with ISO/IEC 15288. It has resulted in the development of a number of standards such as the Unified Modelling Language (UML) for software intensive systems and, more recently the Systems Modelling Language (SysML)[4] which supports many of the ISO/IEC 15288 technical processes.

These are shared objectives with the wider systems engineering community (of which software is but one discipline) and that system engineers could and should consider MDE for holistic system definition.

MODAF[5] has a strong underlying association with UML/SysML and many of the fundamental elements and views have UML at their core. The objectives of MODAF are to provide the MOD with a common framework for definition of military systems. It has the capacity to fulfil the objectives of compliance, coherence and integration stated in **Figure 6** by facilitating behavioural and architectural viewpoints accessed by all. The abstracted system is captured as a collection of connected, coherent and consistent military artefacts.

Common definitions of views of behaviours and architectures exist at different abstraction levels. There are obvious parallels with the systems engineering life-cycle technical processes in ISO/IEC 15288

- Strategic views define the desired business outcome and what capabilities are required to achieve it
- Operational views define (in abstract rather than physical terms) the processes, information and entities needed to fulfil the capability requirements. The logical views may include a service oriented representation of the capability
- System views describe the physical implementation of the operational and service oriented views and, thereby, define the solution.

The MOD estimate of savings through the use of MODAF is between £0.75Bn and £3Bn[6]. The use of Enterprise Architecture, process design patterns, and MODAF should significantly reduced rework. More effort is required at the early phases of the system definition phases but this has significant payback through life, see Figure 6. In order to routinely gain these benefits, some changes are required.

- The culture of working in the comfort of the solution space and the detail of physical components needs to be addressed and a more abstract, function based approach adopted.
- More successes are needed to build a belief in the approach. This would result in increased confidence of budget holders and programme managers.
- The competency levels of system engineers needs to be improved.

To date, the introduction of MODAF has been disappointing. Ironically, given the rationale of MODAF, the MOD is still struggling with a diversity of approaches and integration across the enterprise. The following are contributing factors:

- No consistent use of a common system engineering framework with MODAF at its core
- Stated model objectives at the outset of a programme of work are essential. What questions will the model answer through delivery of coherent design[7]?
- Little or no abstraction. The Operational Views, for example, are intended to be “logical” yet the Operating View 1a (although intended for non-engineers) invariably contains solutions and raises expectations accordingly.
- Varying interpretation of MODAF views and their purpose.
- The tools commonly used do not support the concept of an integrated design repository. Design information is disjointed and often repeated.
- Document centric engineering is still prevalent and there is little understanding of the strong association between architecture, behaviour, and requirements.
- MODAF, in fact any framework, can not cover all of the needs of system definition. Strict adherence to the underlying MODAF Meta Model increases design costs and provides little value to the decision makers.

Although MODAF is not perfect, its underlying principles are essential to the success of the enterprise. It has the potential to become the integrated environment and design repository with all of the richness of associations between elements. However, a more pragmatic approach to MODAF compliance will produce the benefits and minimise the costs.

Maritime capability

Enterprise analysis

Enterprise analysis of a system to satisfy business goals includes the key Strategic View-2 diagram in **Figure 3**, which defines the high level capabilities. These, together with technical, commercial and business constraints, shape the architectural design. This is the starting point for decomposition of the system through behavioural analysis. Stakeholder needs are elicited and analysed by setting the system in context for the known operational and support capabilities. Behavioural analysis of the system as a "black box" gives a coherent list of stakeholders and functional dependencies as well as the system connectivity context which defines external agencies and information exchange, see **Figure 7**. The key benefits of this analysis are:

- The scope of the system has been identified leading to a better understanding of acquisition, operation and disposal costs
- Technological and dependency risks can be identified and managed at an early stage
- User requirements can be formulated into contractual agreements
- Validation criteria can be seeded through the use of behavioural scenarios
- The impact of technological, commercial and business constraints is well understood and a compliance plan has been established
- The acquisition life-cycle can be scheduled and costed.

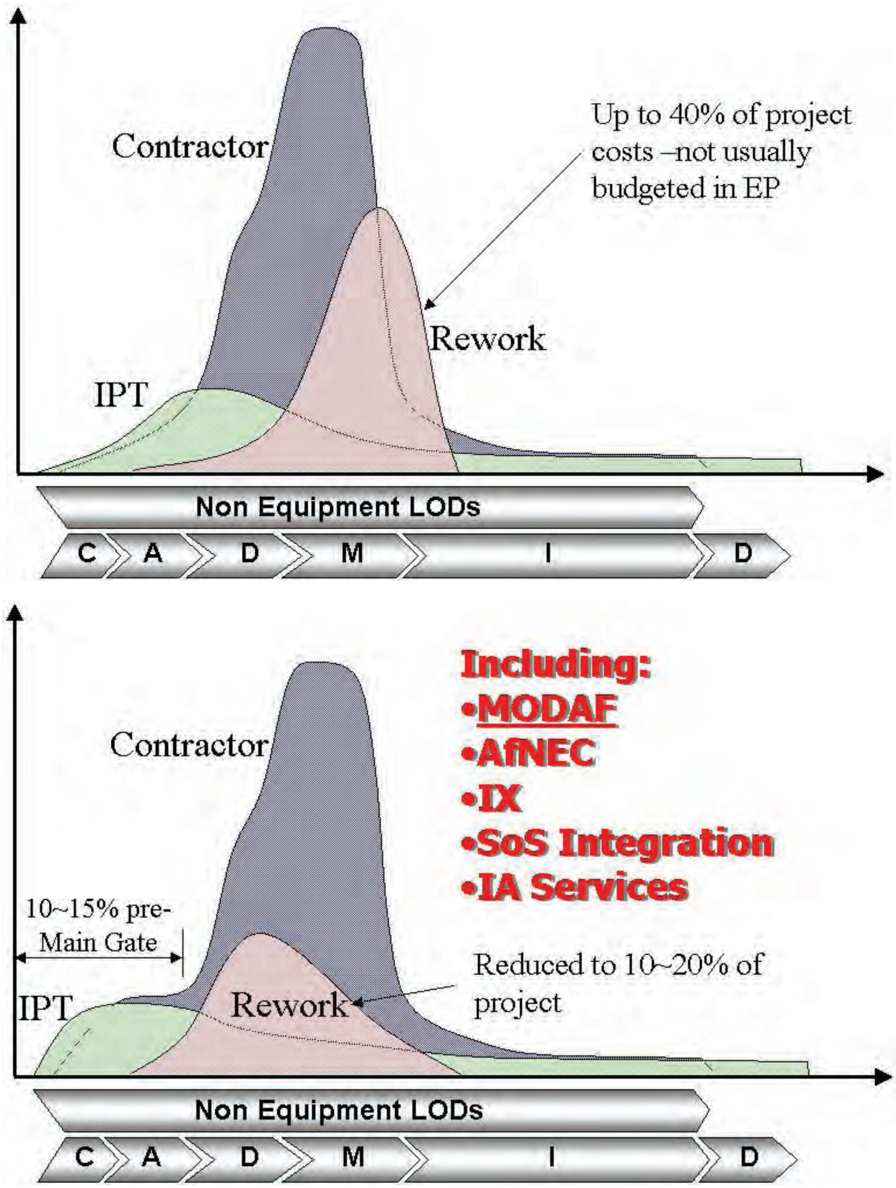

Figure 6. Potential benefits of using MODAF

Note: Figure 6 contains the following abbreviations:

- IPT: Integrated Project Teams – MOD requirements development and commercial team for equipment procurement
- EP: Equipment Programme – MOD's standard procurement process (as opposed to urgent operational requirements)
- LODs: Lines of Development (Training, Equipment, Personnel, Information, Concepts and Doc trine, Infrastructure, Logistics),
- C, A, D, M, I, D (Concept, Assessment, Demonstration, Manufacture, In service, Disposal)

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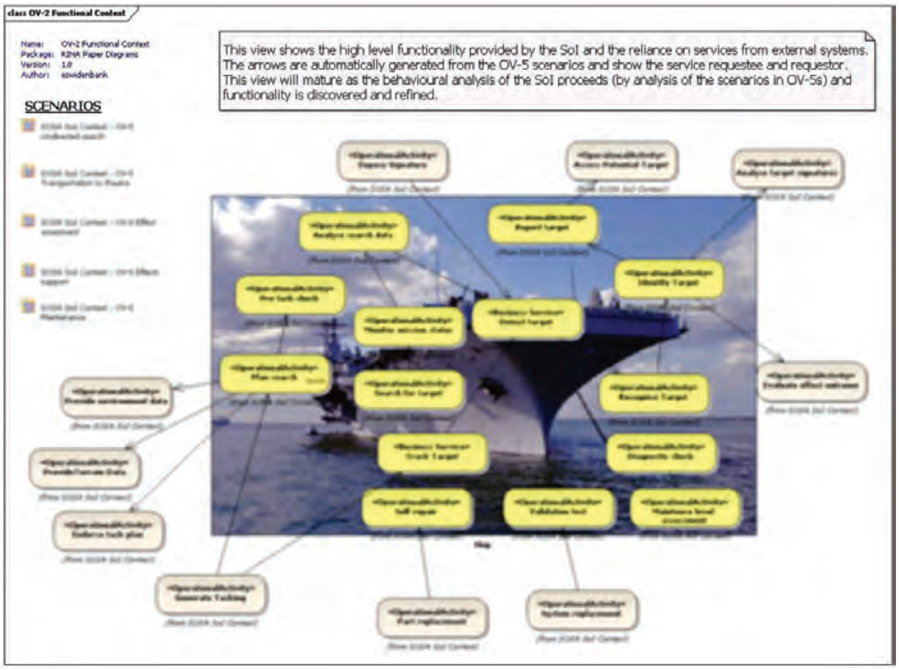
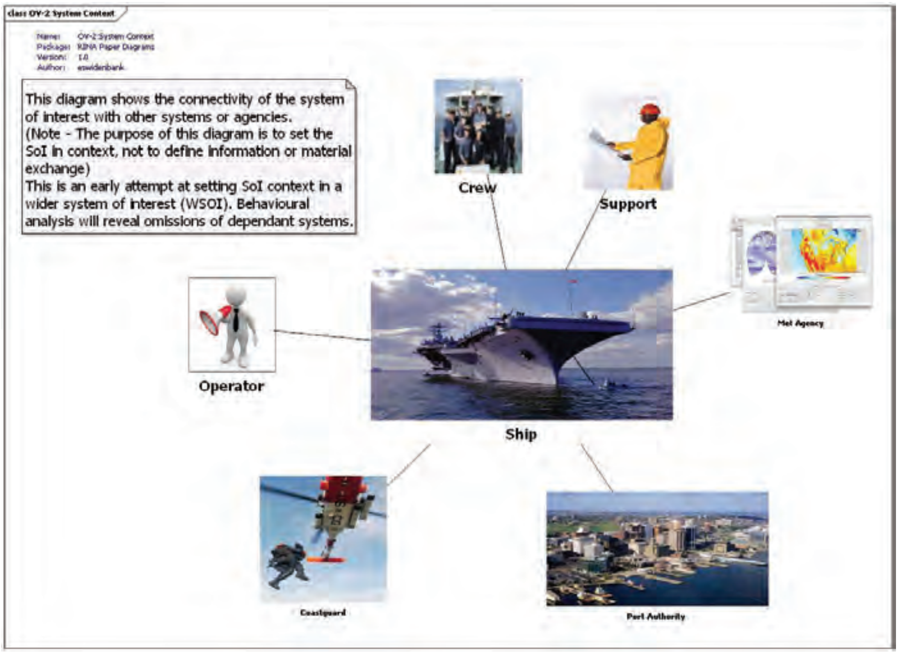


Operational View -2 Functional context



Operational View -2 System context for System of Interest (SoI)

Figure 7. System context

System definition

This process opens up the system as a "white box" although it must remain consistent with the context defined during enterprise analysis. Complexity is reduced by breaking the system into functionally cohesive components and defining the services or functions of those components, see **Figure 8**. The behavioural analysis design pattern is used to achieve this. The behavioural description uses logical components and there is no commitment as yet to solution. The approach must be pragmatic in that the constraints of potential physical solutions are considered. The design should be enduring throughout the lifetime of the system, given stable user requirements. **Figure 9** shows how a common behavioural definition provides a coherent baseline for analysis by speciality engineers.

The benefits of a logical definition are:

- Behavioural threads using logical components are constructed in a language that can be understood by both the stakeholder and developer communities
- An holistic view of the system is produced which includes all components required to support a well defined task
- System functional requirements and quality of service can be formulated so that a number of solution configurations (including re-use of legacy systems) can be considered at the system definition phase
- Provide a common definition (central design repository) for analysis of:
    - System and system-of-system safety
    - Operator competency expectations leading to training needs analysis
    - Validation criteria seeded by behavioural scenarios
    - Human factors analysis
    - Information management (exchange, consistency, quality and re-use/exploitation).

System development

The core activity in this phase is the creation of one or more solution configurations that satisfy the functional requirements defined during system definition. Non-functional requirements have a major impact on the architecture,

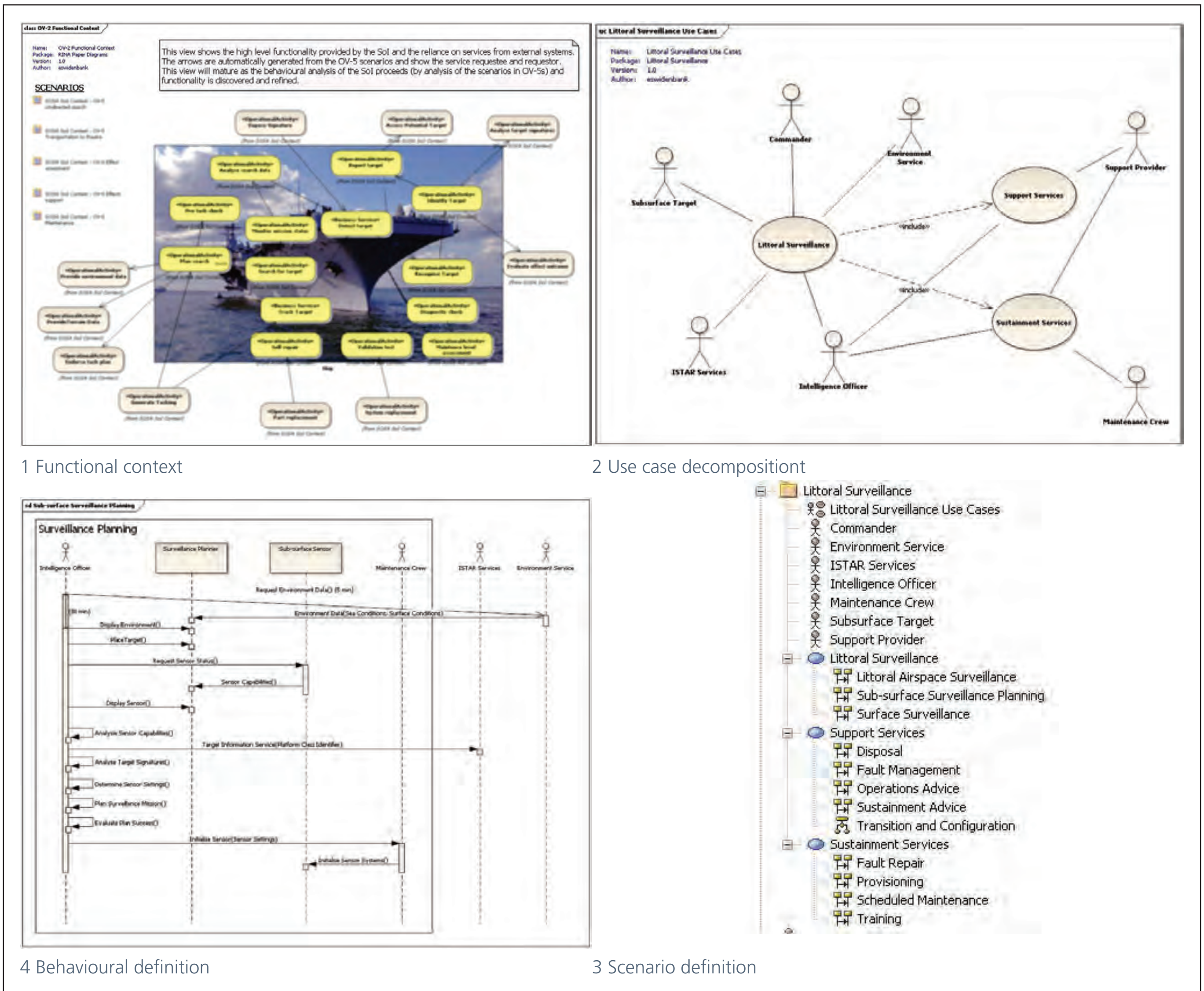


Figure 8. System functional decomposition

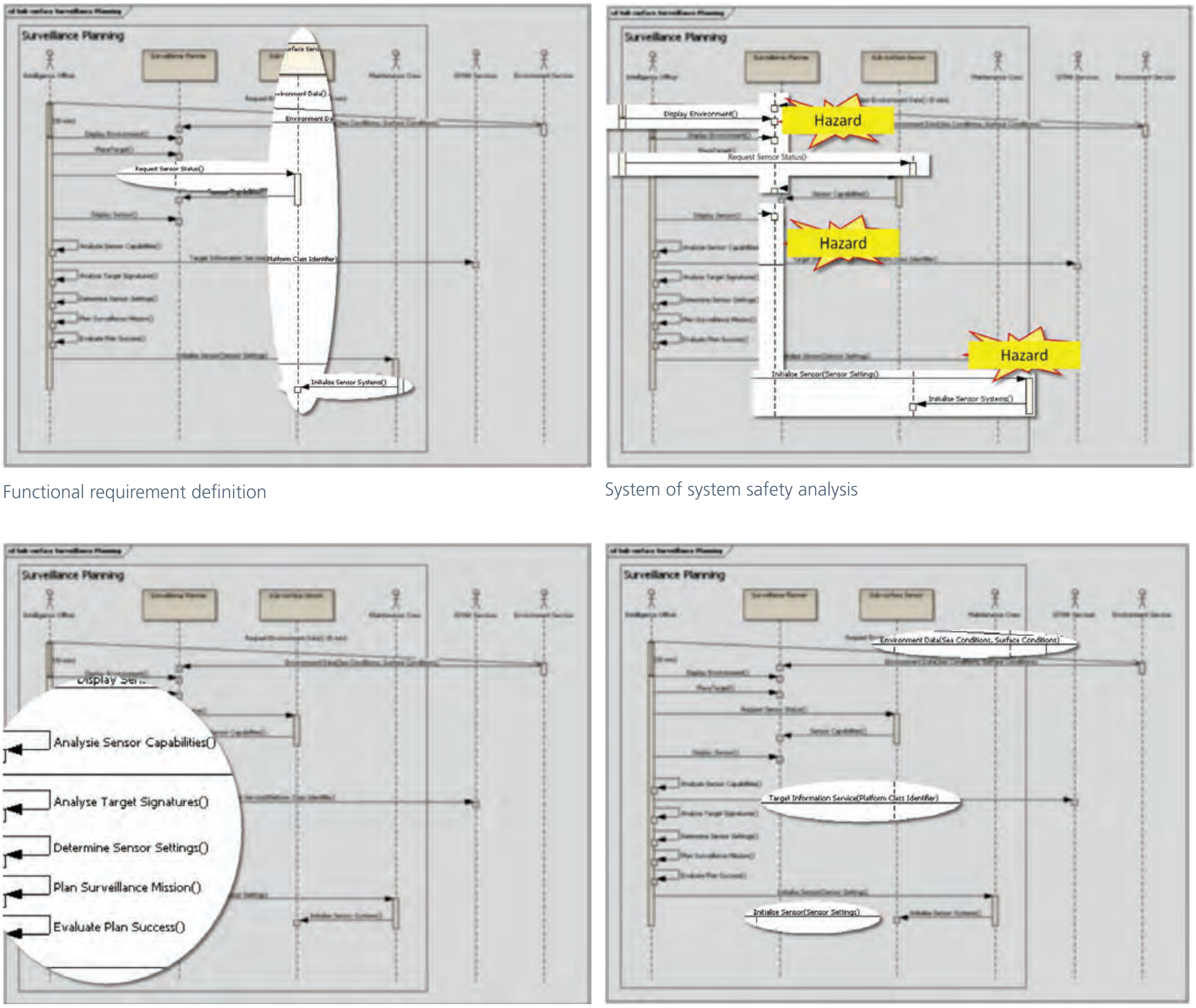
as do technological and operational constraints.

Figure 10 proposes two solutions to the sub-surface surveillance capability. Both satisfy the requirements but will have a significantly different through life cost, for example the inclusion of additional crew for the manned patrol boat solution will have an impact on the crew support and well-being.

Non-functional equipment specifications can be derived at this time. There will be performance budgets e.g. weight and size that must be managed. These constraints impact and define the specifications of the equipment configuration items.

The benefits of the solution architecture are:

- Choice and justification of solution architectures against an enduring logical definition baseline
- Integration tests can be seeded using the physical architecture
- Through-life cost and risk analysis
- Generation of coherent technical contractual requirements
- Phased delivery options from initial operating capability to retirement.



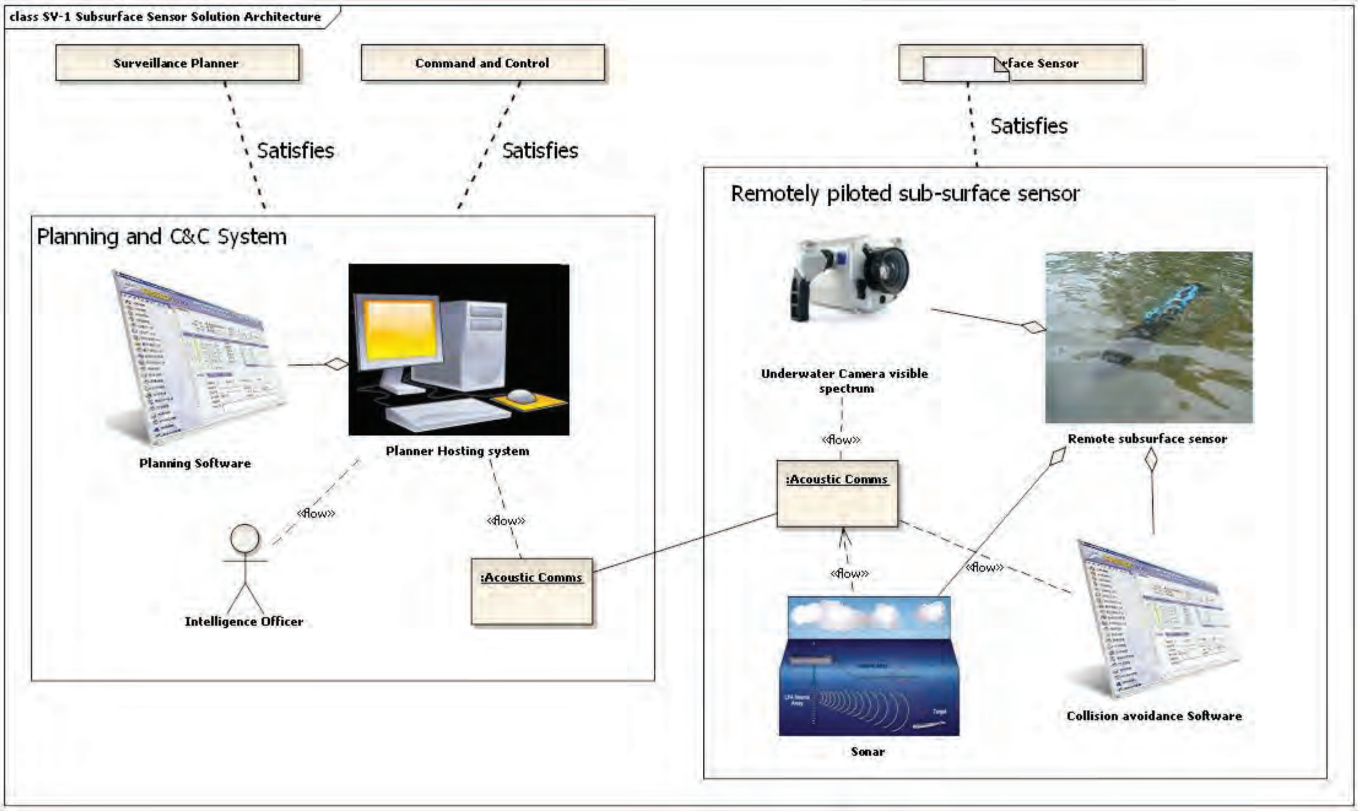
Functional requirement definition

System of system safety analysis

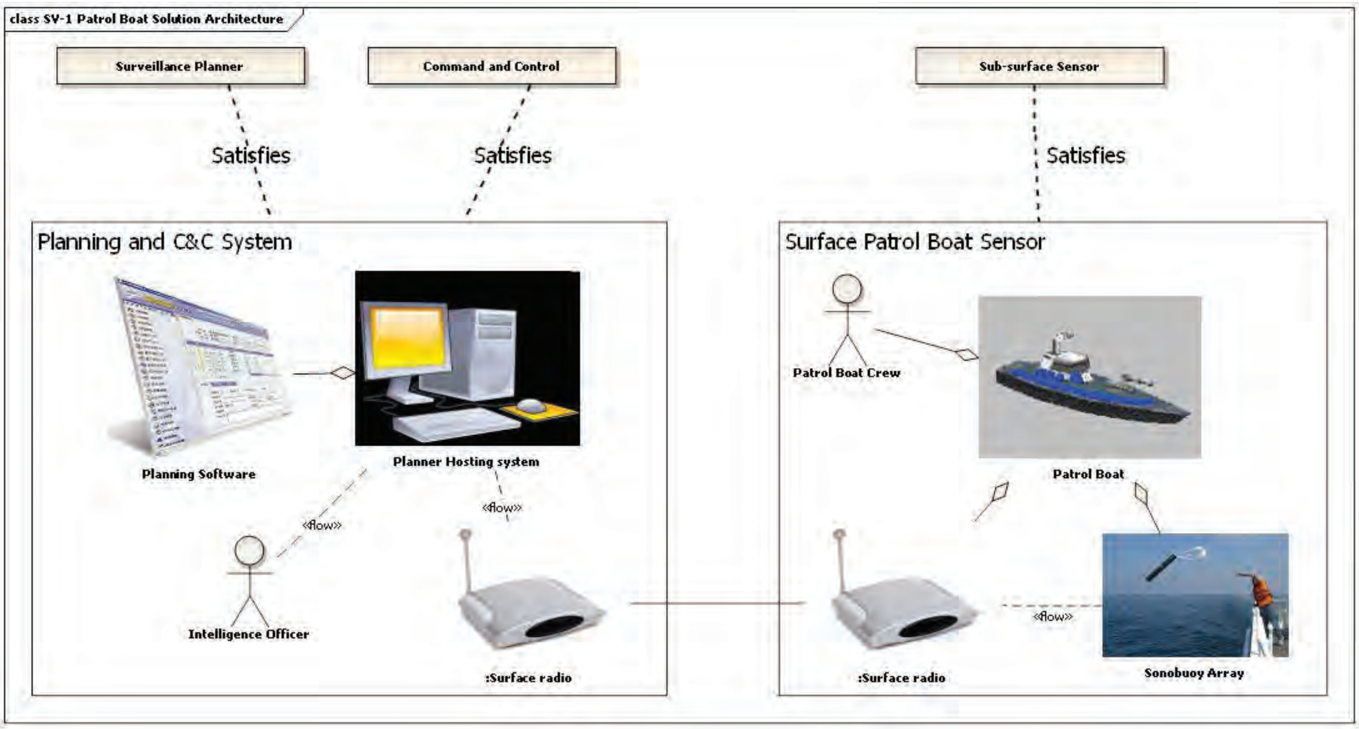
Operator competency and training needs

Information management and exploitation

Figure 9. Analysis of operational and support behaviour



Remotely piloted sub-surface sensor



Crewed patrol boat sensor

Figure 10. Solution architecture options

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Conclusion

This paper has proposed the use of functional breakdown for reduction in complexity of marine systems. This has many advantages such as a quality improvement through an holistic view of system capabilities. The systems engineering standard (ISO 15288) has been used as a framework for the system definition phase of the life-cycle. When this is teamed with a model driven approach and a modelling framework (MODAF), the result is a coherent set of system and support specifications which have been optimised through constant stakeholder review.

Acknowledgement

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Somerset Highways' mobile highways management solution

Abstract

Using advanced mobile highways management solution Atkins Inform to manage highway defects, Somerset highways has seen a 52% cost and efficiency saving without having to increase resource levels. This paper describes the system which was developed to achieve this.

Introduction

With over 30,000 road defects reported each year and local authorities expected to 'do more with less' in the face of public spending cuts, how do you manage highways effectively and efficiently? Somerset Highways, the highways maintenance partnership between Atkins and Somerset County Council, have developed a mobile solution to allow for field based management of highway defects.

Since 2008, Somerset Highways has adapted the Inform system to become an advanced mobile highways management solution. The all-electronic, end-to-end solution enables seamless, field-based management of highway defects between Somerset's highway inspectors and its highway contractor, Atkins. The improvements made have involved the Somerset County Council System (Confirm) and the Atkins System (Inform) speaking to each other without the need for paperwork or manual input in the information flow with the systems.

The Inform Mobile solution won the 2011 Highways Magazine Excellence Award for Most Innovative Highway Authority project of the year[1].

Judges commented that "with most authorities working towards mobile working, Atkins' partnership with Somerset is a good example of using technology to help improve highway efficiencies."

How does it work?

As shown in **Figure 1**, Somerset Highways' inspectors report defects in situ on mobile devices and send this data, including the exact location of the problem back to Somerset's central Confirm system. An Atkins interface extracts records set as 'committed' by SCC directly from the Confirm database. The data is then formatted into a standard Confirm interface file and placed on the Inform FTP server. These files are picked up every 15 minutes and uploaded into the Atkins system. The files create a work order and job in Atkins' finance system, CPA, and operational system, which are loaded onto the defect repair gangs' mobile devices.

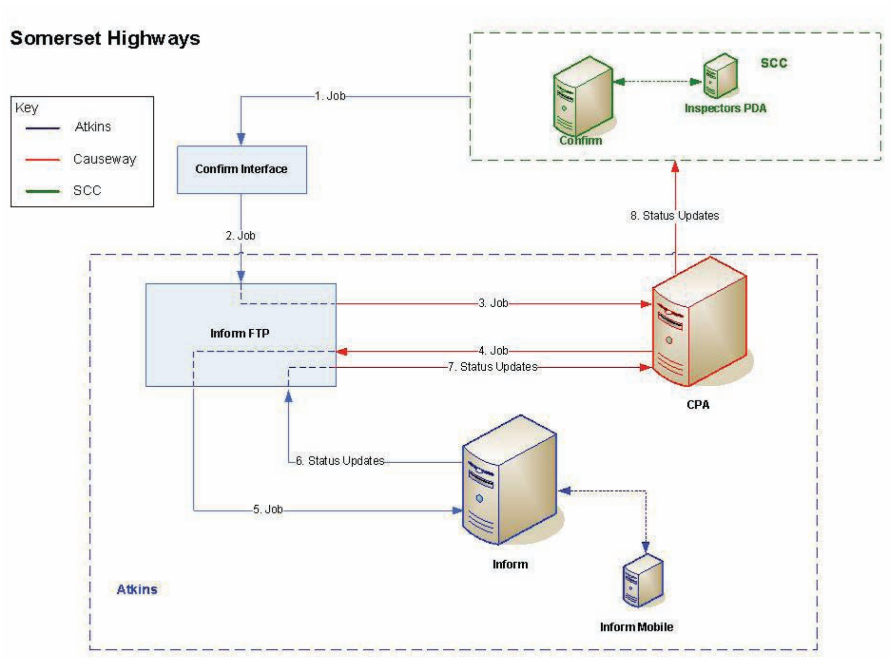

Figure 1. Confirm-Inform system

The gangs are deployed through the mapping of works which is done by the schedulers in Somerset Highways. Software called SDI Spatial Data Infrastructure Mapping is used, shown in **Figure 2**. It shows the position of all the defects and it highlights the best routes for each of the gangs. Once the route is planned the schedule uses a new innovated scheduling tool to "drop" the works into a gang diary on their Mobile devices.

Members of the public are also able to start the electronic process themselves by pinpointing the location of a road defect on an interactive map on Somerset County Council's website, shown in **Figure 3**, which then triggers an automatic inspection of the problem.

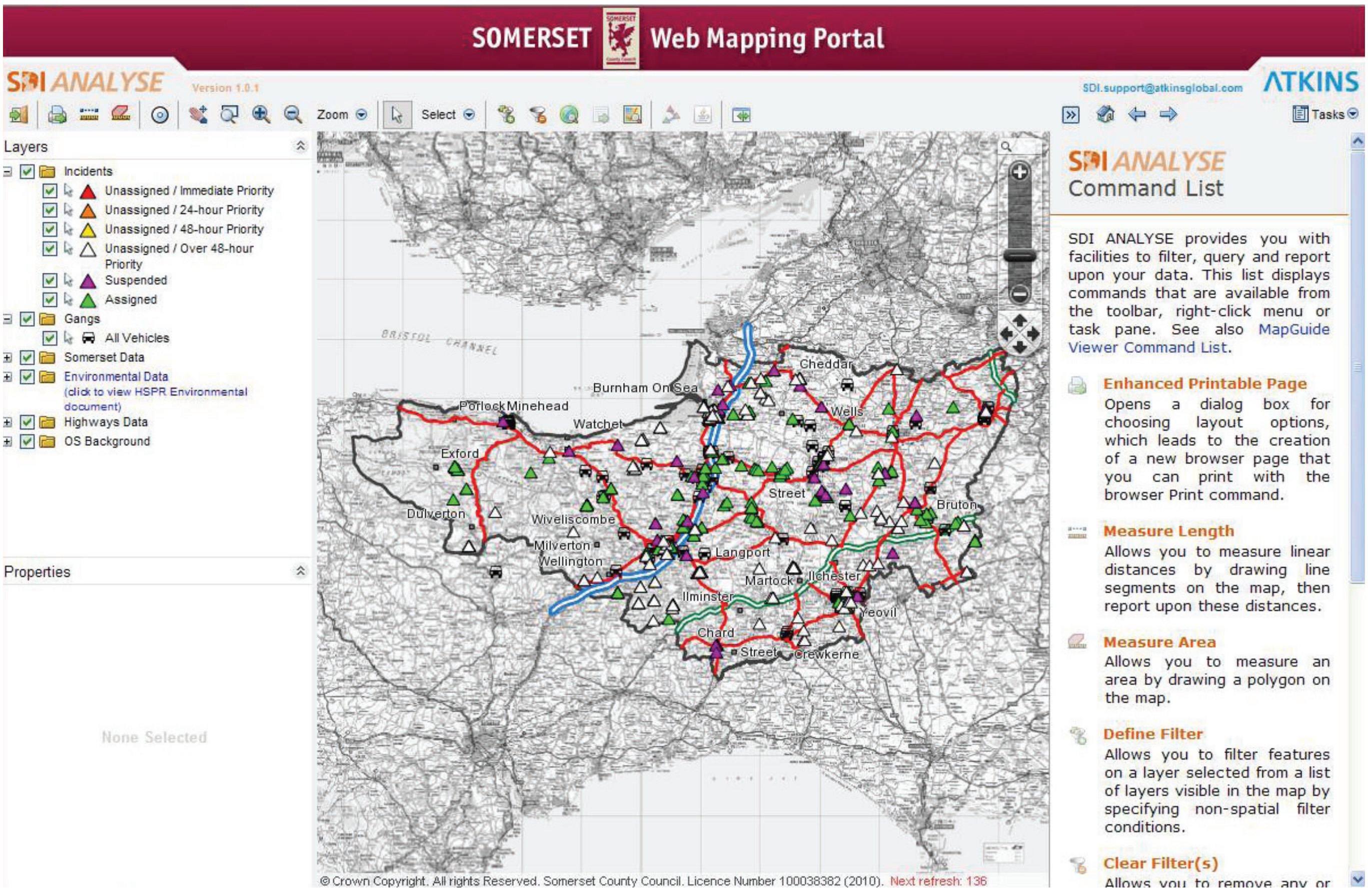

Figure 2. SDI Spatial Data Infrastructure Mapping.

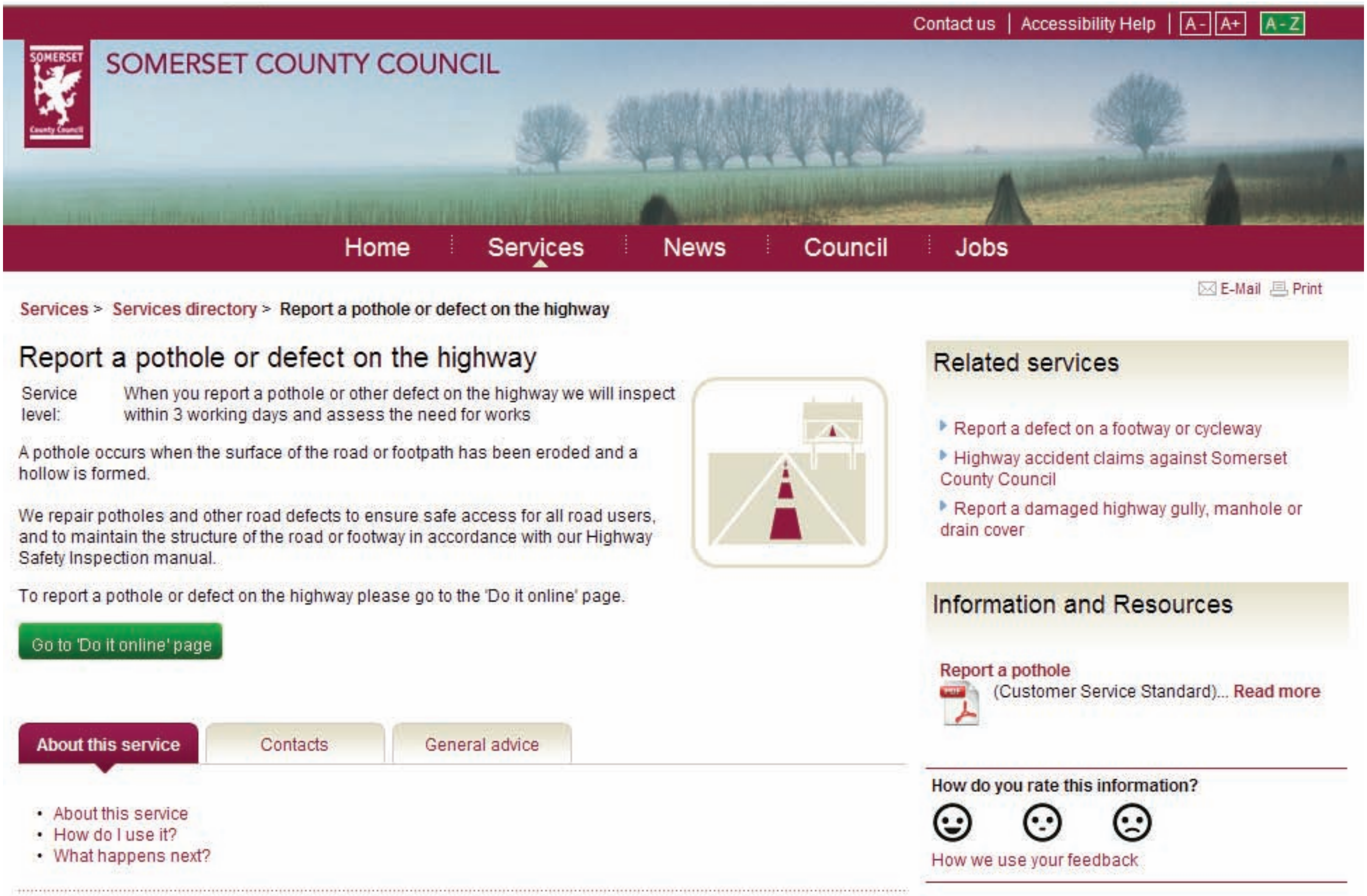

Figure 3. Public reporting system for highway defects.

The mobile device uses SatNav technology so once the information gets transferred into their device it guides the defect gangs directly to the defect. As shown in **Figure 4**, once on site the device enables the gangs to input a range of data:

- Start and finish times
- Dynamic risk assessments for repairing the defect
- New schedule of rates if the defect has used more material or is larger than when the inspector reported it
- 'Before and after' photos to ensure it can be signed off by county inspectors.

Upon completion, the inputted data files are picked up from the Inform FTP server by CPA and processed. Status updates are sent at 15 minute intervals to the Confirm System Agent to be uploaded to Confirm. These include the key dates which originated from Inform, statuses such as 'depot allocated' and 'work complete (measured)' and the jobs' measures, which are initiated from CPA. As photos of the finished job have been loaded the County Inspectors can sign the job off with a single click.

Training

Training on the entire system, including the device, took two weeks. Each of the gangs was trained on the device in a specially designed session, where the gangs practiced using the devices on test jobs until they felt comfortable with its functions. Additional back office support was provided for a month after the go-live date.

Benefits

While many highways inspector/ contractor systems are partly electronic, the Somerset solution is believed to be the most advanced and comprehensive in terms of its scope. Using the Confirm system has yielded cost and efficiency savings of 52% by helping the defect repair teams to do their job faster and more accurately. The all-electronic process has enabled more inspections to be carried out using the same inspection team and seen a 100% increase in defects being repaired, without the need to increase staff resources. The mobile transmission and receipt of information between inspector and gangs speeds up the maintenance process, while the all-electronic system enables highway works to be programmed and planned in advance rather than being purely

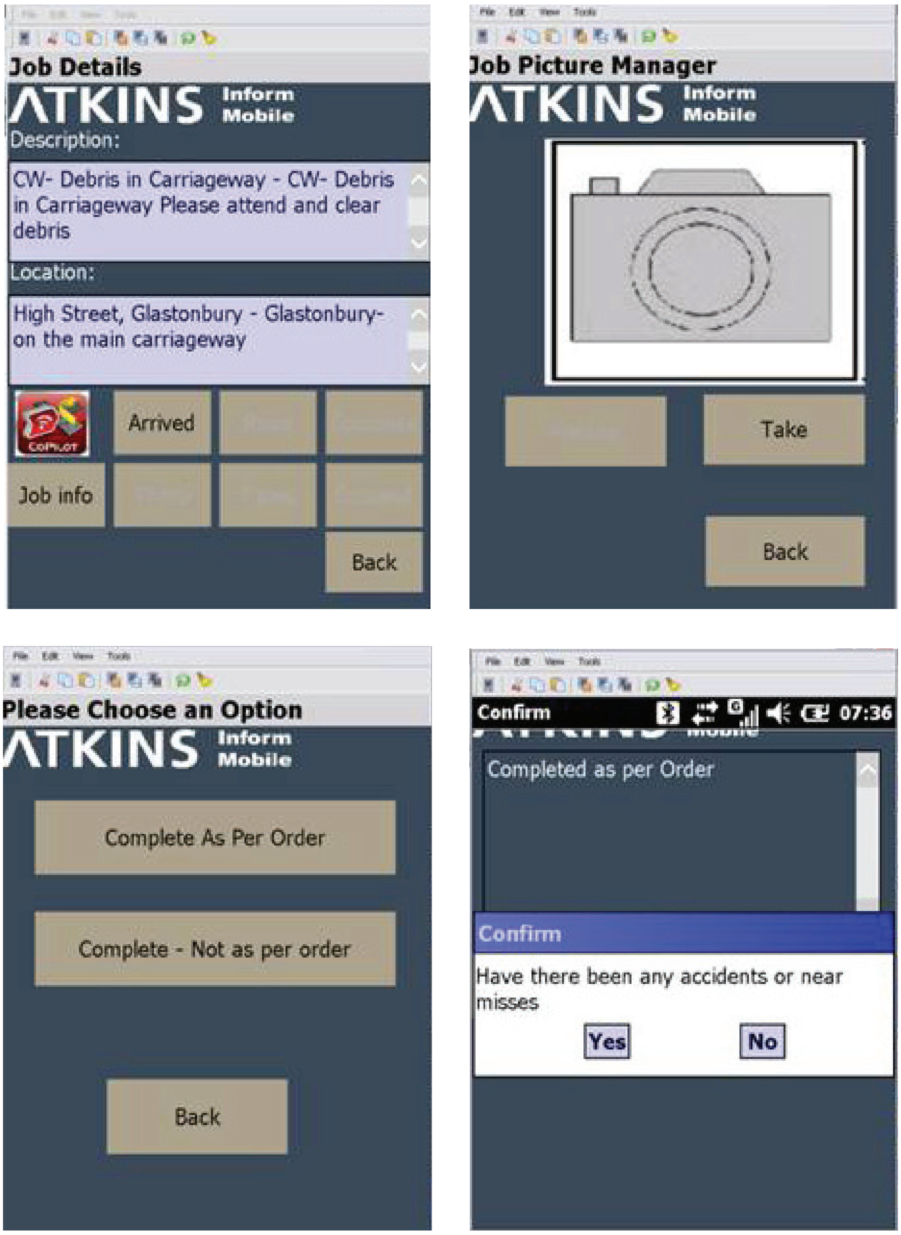

Figure 4. Mobile device data input fields

reactive. Atkins' work gangs are now able to respond more quickly and efficiently to logged faults, resulting in 100% of safety defects being repaired within their target response time. In addition it has significantly reduced time-consuming paperwork, saving approximately 78,000 sheets of paper a year, (equivalent to nine trees).

The benefits of end-to-end management of street works for Somerset's citizens are also clear: faster and more accurately targeted repairs mean fewer traffic jams and a safer built environment. **Figure 5** shows the results of the 2010 National Highways and Transport Network survey, which ranked Somerset the seventh local authority and the second county in the country in terms of the public's satisfaction with roadworks management.

Being able to report and log defects directly from the actual location of the problem is clearly the way forward for highway inspection teams, but the seamless, end-to-end nature of the Confirm-Inform based solution developed at Somerset means that the resulting job stays within the electronic system between Atkins and Somerset County Council right up to completion.

What's next?

Going forward, Atkins and Somerset hope to extend the system even further into the public realm, by sending regular updates to individual citizens who have reported faults, including information about when the repair will occur and confirmation of its completion. The all electronic system is also beginning to be used for other larger work schemes across Somerset.

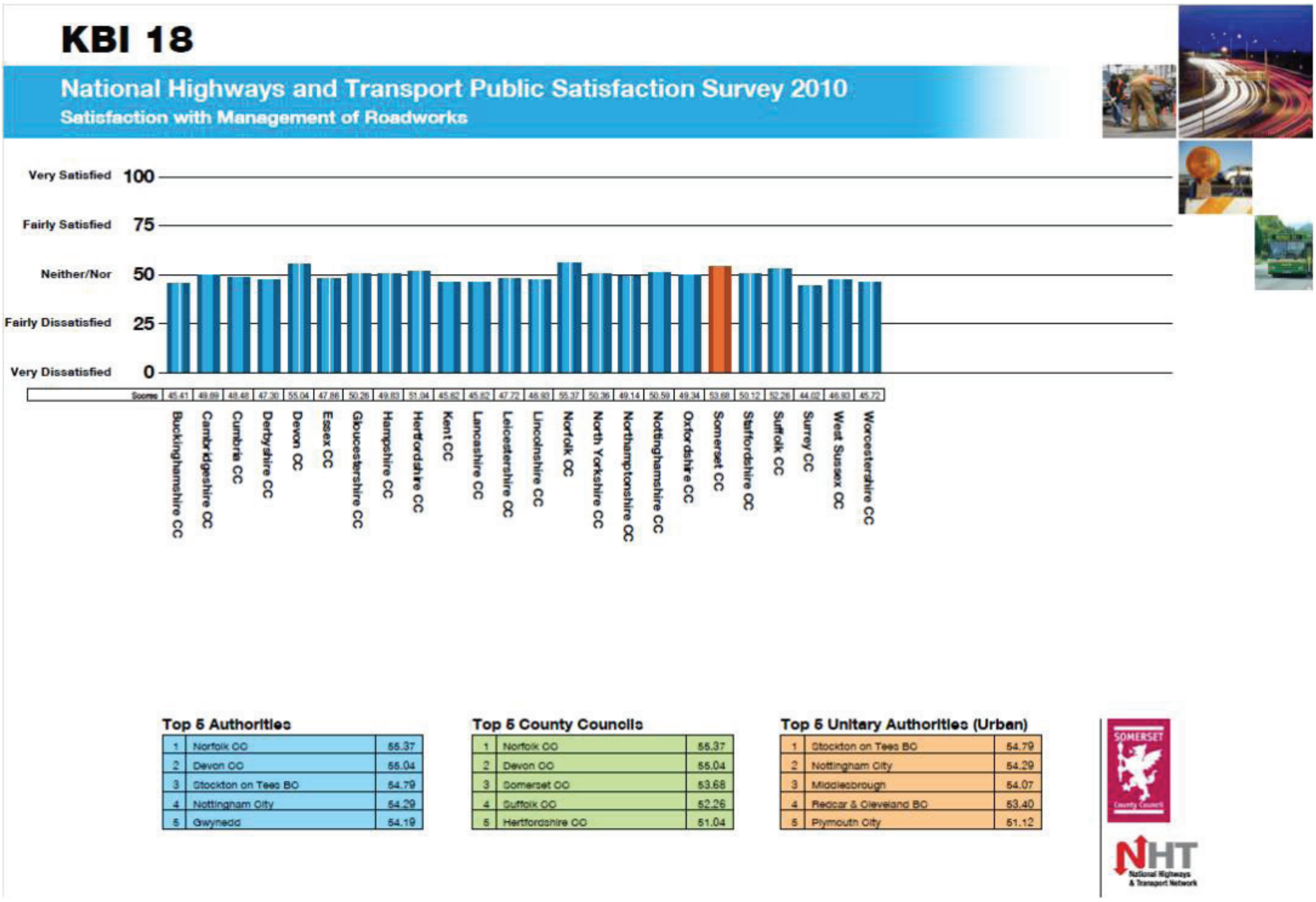

Figure 5. Results of the National Highways and Transport Public Satisfaction Survey 2010[2].

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Systems Engineering

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Industrial innovation management: A systems approach

Abstract

This technical paper presents a practical and systematic approach for the measurement and assessment of industrial innovation systems. It presents a new systems thinking approach to analysing and managing innovation across organisational boundaries in complex industrial systems. Drawing on best practice from numerous sectors, proprietary research and empirical experience, a set of interdependent frameworks are described which can be swiftly deployed and easily configured to bring a fresh perspective to a challenging and often-misunderstood problem space that confronts all organisations seeking to identify new sources of value and reduce cost and complexity. Senior stakeholders in these organisations and industrial systems at business unit and executive board level would benefit from understanding this systems thinking approach when considering how to transform their strategy to support an innovation-led growth strategy.

Introduction

What is innovation?

Although the term 'innovation' is widely used to describe almost any aspect of industrial change, its definition is still the subject of debate between academics and industrial experts. In general, although the word is defined in many different ways, definitions are broadly classified in two main categories: those that see innovation as the final event – 'The idea, practice, or material artefact that has been invented or that is regarded as novel, independent of its adoption or non-adoption'[1] and those who see it as a process 'which proceeds from the conceptualisation of a new idea to a solution of the problem and then to the actual utilisation of a new item of economic or social value.'[2] Since the subject of this paper is to discuss innovation in industrial systems, we will adopt the latter definition, where innovation is primarily considered as a scientific and systematic staged process (Technology Maturity Cycle / Technical Readiness Level) for simultaneous engineering, or for the development of new methods, products, services, systems and capabilities – and the bringing of change based upon them. Nonetheless, it continues to be absolutely essential to understand the presence of multi-dimensional causal factors at work and not to overly simplify an incredibly complex process, cutting across a broad range of activities, institutions and time spans.

Economic significance of innovation

It is well understood that the growth of economies depends on the growth and survival of industries and firms and that innovation plays a powerful role in revolutionising economic and business activities in the industrial world. A number of developed countries have placed innovation firmly at the heart of national industrial strategy and policy objectives. This strategy aims to create an environment that enables innovation and supports its adoption and the spreading of its benefits to all industry sectors. Presently, with an increasing number of countries and industries adopting innovation, the discussion focuses on how best optimum results can be obtained so as to maximise the benefits to business and society. If this is to be successful, it is imperative to develop and implement an effective innovation measurement, assessment and control system to transform industrial objectives into innovation-led growth strategy.

The 2009/2010 INSEAD global innovation index states that "Recent economic history has shown that, as developed countries approach the recovery, growth and competitiveness frontier, innovation is crucial for them to continue innovating in their processes and products and to maintain their competitive advantage. Equally important, innovation has proven instrumental for enabling developing and middle-income economies to leapfrog to higher stages of development and fostering economic and social transformation."[3] This indicates that innovation not only provides a competitive edge to the industry, but also acts as a game changer. It also concludes that one clear feature of successfully innovating industries/countries is that in the face of adversity, they deliberately play to their strengths and act boldly, with an eye on new opportunities. In the UK, these principles were encapsulated in the 2010 Hauser Review by the Department of Business, Innovation and Skills (DfBIS)[4].

Innovation value and risk management

"What we measure affects what we do. We will never have perfect measures – and we need different measures for different purposes". Prof. Joseph Stiglitz, Columbia University

In the spirit of the above quote, this section discusses the significance of having a methodological tool that allows a quantitative measure of innovation system structure and

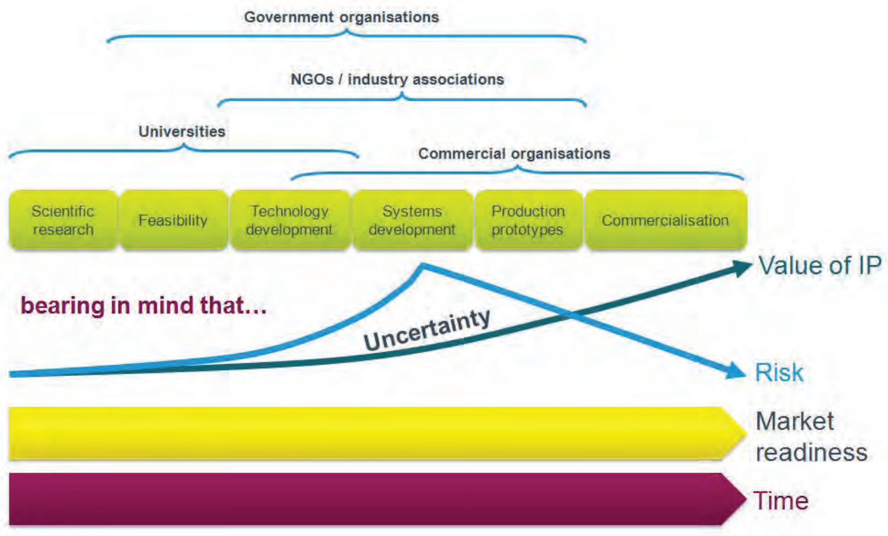

Figure 1. Innovation risk and value over time in a complex industrial system

the value created by it. Innovation is often incorrectly depicted as a smooth, well-behaved, linear process, completely ignoring the complex nature and direction of the causal factors. In fact, it is ephemeral, dynamic and subjective, gloriously messy and constantly evolving in nature, thus making it difficult to measure simultaneously in terms of both structure and causation. Therefore it is essential to understand both the structure of innovation systems and their capability to create value over time in order to effectively measure, assess and influence the key inputs and outputs to the innovation process.

The value of innovation is often assessed in terms of its financial returns over time, which involves the management of investment risk in creating this value, usually involving some element of intellectual property. Ideally, all commercial organisations want their innovation Intellectual Property (IP) for free, without taking the risk of investing in it over the "mountain of uncertainty" (see **Figure 1**). There is reluctance to get involved at the early stages before the value has been created, but, obviously, this is subject to contingent pricing later in the process. Conversely, universities are intensely involved in the creation of IP from fundamental research but frequently lack the means to enter the market on their own without commercial partnerships. Moreover, academics are less concerned with results as long as funding remains available. The result is an uncertainty gap between where universities' drive peters out and where the drive for commercialisation begins in earnest.

The situation is exacerbated when, as in transport, there are so many stakeholders that no one organisation can act in isolation. The qualitative aspects of that gap – the process of creating the value inherent in an innovation platform across an industrial system – are, however, not well understood. Similarly, there are significant gaps in understanding about the people, process, structure and culture of organisational behaviour in relation to innovation, especially in regard to what drives organisations to decide to invest – or not – in innovation and how they view the creation of value beyond their own locus of control and influence.

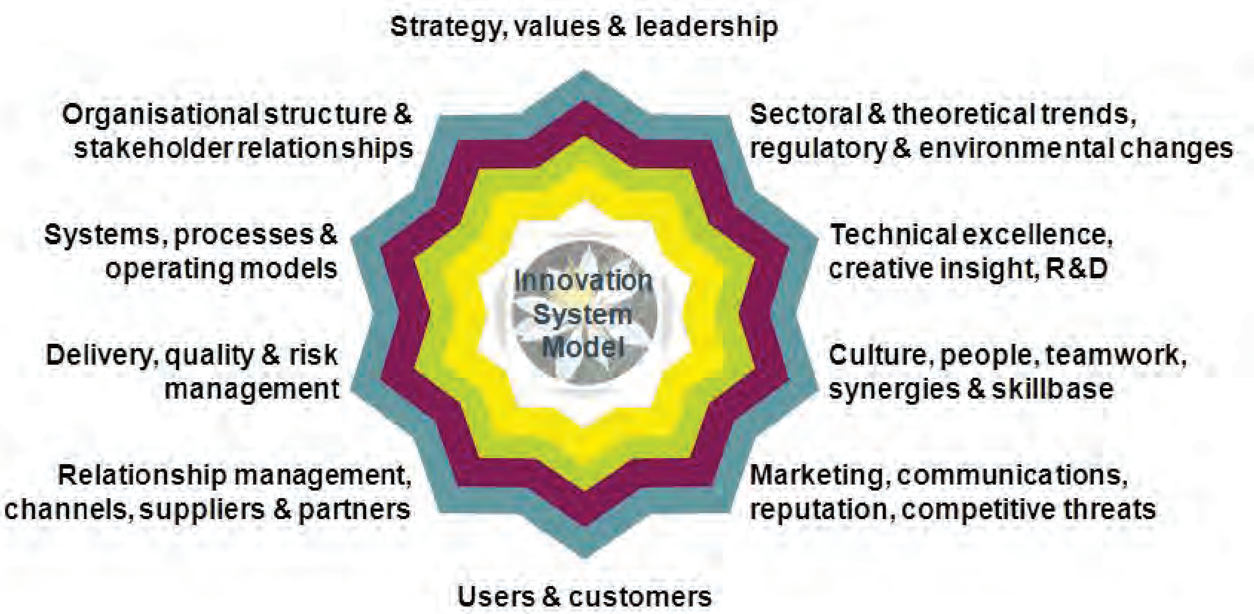

Figure 2. Innovation system structure model configured for the GB Rail innovation system

Measuring innovation – a systems approach?

This section discusses the models and techniques used to measure and summarise complex, multi-dimensional innovation structures, with a view to enabling decision-makers to compare the effectiveness of innovation value creation and progress in industries over time. The models discussed below are capable of measuring innovation so as to estimate where the industry would move to if all the causal factors were fully implemented. Users may also find these techniques easier to interpret, increasing the transparency of overall processes and accountabilities of both investors in and beneficiaries of the innovation systems.

The Atkins Rail Business Improvement team has developed a measure of the effectiveness of research, development, testing and innovation, capable of identifying trends over time and suitable for benchmarking and analysis of industry innovation systems and causal factors. The base content comprises two existing "open-source" best-practice structural and process maturity analytical and diagnostic frameworks:

- Innovation System Structure (ISS)
- Innovation Process Maturity Spiral (IPMS)

Innovation system structure framework

The Innovation System Structure model (ISS) displays a system-level view of organisational or industrial activity (see **Figure 2**). It is based upon an "idealised design" holistic systems approach where the aim of the system is to maintain productive dynamic tension, with no subsystem exerting undue influence, carrying undue risk or distorting the overall aim of the system. It is a diagnostic framework that has been developed and refined over a number of years and exhaustively stress-tested for its applicability to innovation and change in numerous industry sectors, subsystems and organisations, including IT, payments, professional services, fast-moving consumer goods, healthcare and latterly engineering and transport. It is important to realise that any subsystem, capability or combination thereof can support the creation of an innovation platform, as Virgin use values and leadership, as BMW use strategic supplier management or as Unilever use consumer insight. It can be used as a tool to view the entire innovation system which may be coterminous with the organisation, but may also stretch beyond it to a consortium or an industry. When deployed at industry level and configured appropriately, the ISS enables change and innovation challenges to be analysed, understood, challenged, localised in relevant subsystems and rebalanced. It can therefore be used to produce anything up to an industry-level innovation maturity profile which can be used to demonstrate the relationships and influences on the system as a whole, ensuring that the issue is comprehensively and systematically analysed to give a picture of "the industry as it is".

The ISS can also be used to analyse critical success factors or systemic risks. Individual organisational profiles can be benchmarked against an appropriate anchored scale, which can then be used to determine a picture of the overall industry profile. The ISS also supports the mapping of perspectives from different organisations to reveal mismatches between strategic objectives, tactical choices and organisational realities, as well as gaps in the industry as a whole.

Successful innovation requires that activities take place across all ten dimensions of ISS (see **Table 1** across) to a greater or lesser degree. The absence of activity on any one of these dimensions constitutes a risk to innovation delivery. ISS can be used to assess the effectiveness of an innovation system by means of qualitative anchored scales, producing an holistic view of its structural integrity.

Innovation process maturity spiral framework

Whilst the ISS is used to assess the content typology of the system, the Innovation Process Maturity Spiral (IPMS) is used to assess the maturity of activity within the system. Like the ISS, the IPMS is an idealised design approach which takes the journey of innovation from insight to reality, which is typically institutionalised within both organisations and industrial systems as a stage-gate process.

Innovation platforms

In the IPMS, embedded is the concept of 'Innovation Platform', which can be defined as a capability, asset or system which, once built, enables the speedy execution and wide extension of multiple market applications. Platforms facilitate and sustain long-term value. They take time to build, but are extremely difficult for competitors to mimic. The decision to develop a platform is a managed risk involving the investment of time and resources (as per **Figure 3**). Although value is created throughout the platform development process, tangible returns begin to appear when the first application is launched. The process of platform creation and the drivers of its various stages

Dimension	Relative innovation activities & subsystems
Policy, strategy and objectives	Strategy formulation, goal-setting, visioning, leadership, clarity of purpose
Trends (social, cultural, political, economic, regulatory)	Opportunity and market identification arising from knowledge strategy, environmental and regulatory considerations, horizon scanning, trend-watching, creativity and innovation theory, undirected research
Stakeholders and funding	Prioritisation, budgeting, resource allocation, investment strategy, sources of funding
Technical capability, skills & best practice	Technical/technological investigation, prototyping, experimentation, directed research, proof of concept, Intellectual Property (IP) generation
Value for money and operating models	Financial scenario modelling, cost-benefit analysis, investment return, breakeven analysis, sensitivity analysis, business case development, IPR modelling
Organisations, people and collaboration	Integration activities, change management, facilitation, internal/external collaboration and partnering, knowledge-sharing, organisational learning, skills development
Competition and communication	Proposition development, market research and market communications, branding, competitive differentiation, unique selling points
Delivery, standards and risk	Testing, project management, delivery process efficiency, cost control, risk management, quality and safety, supply chain management
Internal and external relationships	Channel strategy, route to market, purchasing/ procurement interface, contracting
User and customer needs	User experience modelling, customer journey analysis, user insight testing

Table 1. Dimensions of ISS and related innovation activities and subsystems

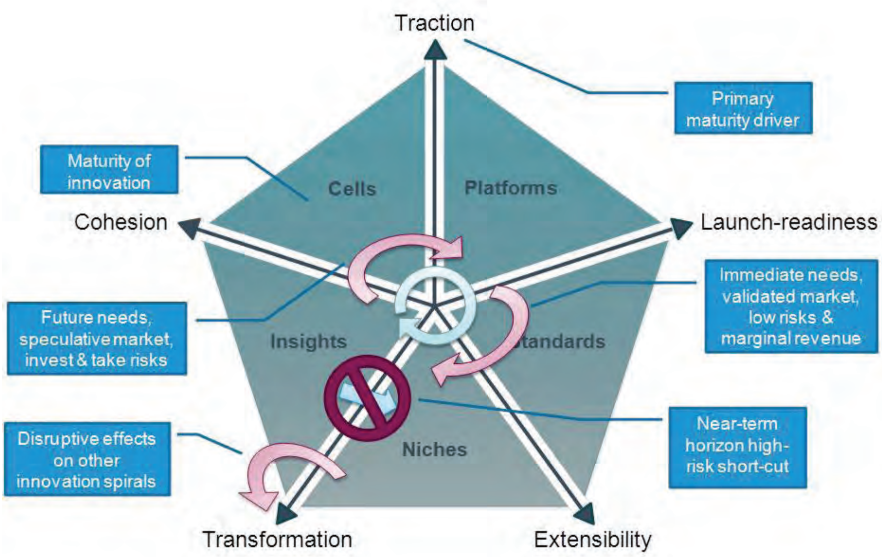

Figure 3. Innovation risk and value over time in a complex industrial system

Maturity level		Primary maturity driver	
Insight	A "germ", someone's bright idea, not fully thought through but worth exploring further with specific expertise	Cohesion	"Well-formedness", creativity, validity, degree to which it captures potential sponsors' imaginations
Cell	Limited exposure beyond a small circle who evangelise and build in an effort to engage key stakeholders	Traction	Believable viability to meet an identifiable need or proposition which can be verified via prototyping, testing and user research
Platform	Potential recognised by stakeholders and approval sought to invest in a launchable build	Launch readiness	Appropriate market conditions, available resourcing, compelling value proposition and clear delivery channel and user utility, mitigated risk profile
Standard	Clear, straightforward, scalable attractiveness to the target market and wide potential for growth	Extensibility	Scope for global applicability and significant profits or cost savings beyond the core user community or target market once firmly established
Niche	A particular application with strong adaptability to a limited market but limited potential beyond marginal growth	Transformation	Crossing boundaries, finding new applications which can disrupt established markets elsewhere or fulfil a previously unrelated function or user need

Table 2. Different stages of innovation process maturity and the drivers maturity at each stage

Capability	Driver activity	Driver activity		Customers	Value base
Insight	Pioneers of statistical radio channel modeling Charlie Bass, Kaveh Pahlavan and Phil Bello all recognised the need to create and maintain wireless networks	Cohesion	Validity of insight, given maturity of technology and likely future trends	ndustry gurus of electrical engineering "futurists"	Investing time in attending IEEE meetings/industry conferences
Cell	Socialisation of concept and agreement of standard through meetings of key industry stakeholders and research funders	Traction	Compatibility with platform architecture strategy of major FCC-member manufacturers and infrastructure	nfrastructure operator and device manufacturer technical strategists	Cost of investigations into compatibility with technical architecture, industry strategy
Platform	Adoption of support for networking standard on key industry technical platforms	Launch readiness	Development of interfaces, relevant hardware and software by device manufacturers and infrastructure operators	Hardware and software manufacturers, infrastructure and ISP operators	Significant development and testing costs, licensing and support contracts drawn up with ISPs
Standard	Incorporate wireless functionality into B2B/ B2C packages offered by internet service providers	Extensibility	Development of propositions, pricing, billing and customer service capabilities	Retail & business users of internet connectivity services	Proceeds of tariff billing for contracted services under user agreements
Niche	Resilient, military-grade applications e.g. MESH from Motorola, also form factor applications like Nintendo Wii	Transformation	Development of new business models specifically geared to these technical architectures and customer needs	Emergency services (MESH) and new gaming markets e.g. home exercise	Premium services for resilient performance and service infrastructure around keep-fit gaming

Table 3. Wireless networking: value creation across the innovation process maturity spiral

of maturity can be sequentially described as follows:

The process is also a cyclical "spiral"; it is apparent from the innovation process maturity model (that value increases over time with launch-readiness marking the point of greatest increase in value generation from the platform. As the number of applications diminishes over time and the limits of extensibility are reached, it is these limits that spur a new lease of life by disrupting or creating entirely new spirals.

As can be seen from the example in **Table 3**, the skills, structures, processes and decisions are very different depending on the maturity stage. In complex industrial innovation systems, no one entity or organisation has all of the skills, staff, processes and capabilities necessary to effect change and drive activity across the entire innovation spiral. For innovation to succeed across organisational boundaries, integration across the spiral stage interfaces is critical. The principal cause of integration failure is most commonly the lack of understanding of platforms – both people and organisations focus on their locus of [apparent] control and those closest to their immediate or personal priorities.

Combination of the IPMS and ISS

Whilst the ISS looks at the structures across an industry system, the IPMS is designed to analyse industry capability across innovation-led change processes. However, the combination of the two gives an extremely rich picture of both the structure of an industry and the system's maturity as far as innovation is analysed in terms of its subsystem areas and structural tensions. This approach, shown in **Figure 4**, combining both structural and process analysis at a system-wide level in two holistic and interdependent frameworks, enables specific interventions and portfolio management systems to be designed which directly address any capability gaps or systemic weaknesses to create end-to-end holistic integration. Recommendations for systemic interventions can be scored based on a configurable balanced scorecard approach, enabling ongoing monitoring and review.

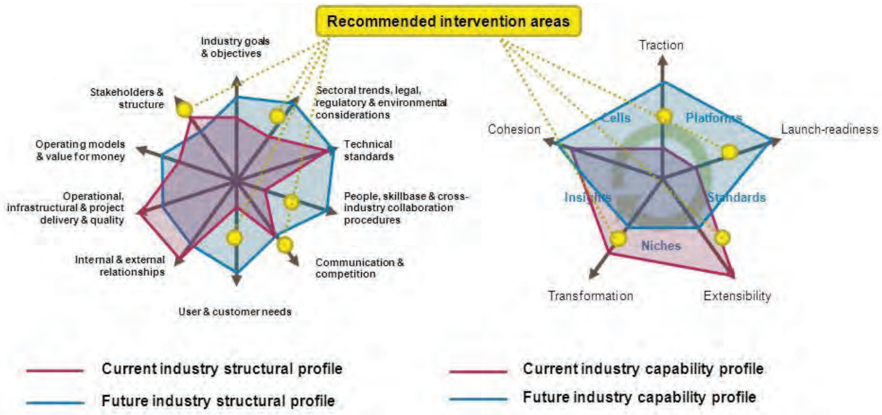

Figure 4. Combination of ISS & IPMS models

Caveats in using ISS/IPMS

ISS/IPMS are tools which are best used for systematic analysis and assessment of all relevant aspects of innovation systems structural integrity and process maturity. Anchored scales can be used to separate out levels of maturity, but should not be confused with quantitative measures of value. A score of 4 in an ISS/IPMS assessment is not, for instance, twice as good as a score of 2. ISS/IPMS scoring is most effective when used to identify differences and gaps, to draw comparisons and spot how an innovation system falls short of the ideal. This implies the development of consensus from stakeholders in the system of what precisely constitutes the ideal design of the system in their view. This idealised design approach allows the identification of "what ought to be" from an examination of "what actually is" by virtue of its inclusion into consideration of all possible elements of the system.

ISS/IPMS should be used to draw conclusions only with considerable care. Ideally both should be used to draw out hypotheses, contradictions and areas for further detailed investigation, as it relies on the accuracy and well-informedness, goodwill and honesty of contributors to a given instance of the models. Similarly, a great deal of care should also be taken in the configuration of the models and anchored assessment scales, the precise characterisation of its dimensions and the phrasing of the questions that are used to draw out insights. It will always remain possible to disagree with the configuration of a model or precisely whether a particular subsystem or system interface should be included in one or other dimension or quadrant.

Project case study: Use of ISS and IPMS within the Atkins rail business

The board of the Atkins Rail Systems Consultancy business commissioned the development of a change programme to embed innovation capability. The primary vehicle of this change programme is a one-day course developed in-house by the Technical Director and the Head of Innovation, known as the "Natural Innovation Seminar". A key component of building this innovation capability is the ability to deploy analytical frameworks for innovation systems and the key frameworks used are ISS and IPMS.

Both are used in exercises to analyse industry innovation systems and, similarly, provide the framework for developing systematic innovation interventions. The Natural Innovation Seminar is presently being offered to other Atkins businesses and can also be offered to clients as a project enhancement workshop, particularly at feasibility stage.

Both ISS and IPMS have been used within the Atkins rail business to support internal innovation projects, including the major "Fabric" programmes such as Cab-I-Net.

Similarly, ISS and IPMS have been used for confidential projects at both a central government department and a government innovation agency. In both cases, the frameworks have been configured to align with the specific features of the industry innovation systems concerned, both to analyse and diagnose the health of the system and to enable a quantitative assessment of all relevant aspects of structural integrity and process maturity. The analysis has produced insights about the relationships of subsystems and system actors and has revealed tensions, lacunae, strengths and weaknesses. Similarly, the frameworks enable like-for-like assessment of both innovation projects, innovation investment portfolios and innovation approaches between comparator industries in different countries and sectors.

Conclusion

Innovation systems run beyond organisational boundaries; even a discrete project may stretch beyond to a consortium or even an industry. The IPMS reveals the importance of the platform-building stage of innovation, which is particularly challenging, as platforms cost money to build, but generate no value and save no money until applications that use the capability are actually launched. The five different quadrants of the IPMS demonstrate the challenge of managing innovation across the entire process as well as the need to balance the needs of the present against the needs of the future. The skills, structures, processes, knowledge and decisions at each stage of the spiral are very different and different organisations have different strengths and capabilities which allow them to act most effectively in specific areas around the spiral. This structural perspective can be analysed using the ISS, which in combination with the IPMS proves to be a powerful and effective systematic approach to understanding the management of innovation in complex and ambiguous spaces, making it possible to identify shortfalls, weaknesses and gaps even at industry system level.

The ISS/IPMS approach enables business unit or executive director level sponsors to design and lead innovation programmes and transform innovation capability in complex industry systems. Innovation delivery requires multi-functional integration but enables value to be delivered across technical, financial and market functions. Its headline benefits include:

- Transformed alignment of innovation to business strategy
- Improved effectiveness of innovation programme delivery framework design
- Improved stakeholder effectiveness and portfolio KPIs
- Enhanced decision-making using an holistic "balanced scorecard" approach
- Increased robustness in tactical delivery of innovation-led change
- Improved transparency, logic and focus to business integration
- Improved management of innovation execution risk
- Enhanced competitive differentiation and defensible unique capabilities
- Enhanced relationships across internal and external organisational and industrial boundaries
- Improved insight into customer, channel, supplier and partner needs

The approach is non-sector-specific and suitable for all vertical markets, but has most recently been applied in a variety of transport industry systems from rail to multi-modal freight, as well as previous applications in financial services, banking, fast-moving consumer goods, professional services and information technology. The approach requires a dedicated programme leader with a strategic focus with a mandate to challenge business as usual and, ideally, specialist facilitation skills.

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3D interchange viewer and walk planner

Abstract

Anyone who has ever planned a walking route using one of the online services available, or navigated on foot using a mobile phone's mapping service, may have spotted that it stops working as a building is entered and it fails to provide adequate information about levels and heights. This proof of concept project attempts to address these shortcomings by presenting the walk route as a 3D image with clear separation of levels and an indication of stairs and lifts. The expected outcome is to enhance the user experience, which will make public transport a more attractive alternative and modal change more likely.

Keywords: 3D, Interchange, Walk Planner, Navigation

Introduction

The UK Government, public transport authorities and operators and some Not for Profit accessibility organisations have spent considerable time collecting information about public transport interchanges, which is then delivered to the public in a number of ways (e.g. lines on a flat 2D map, text descriptions and flow chart diagrams). All of these fail to give the traveller a 'feel' for what the terrain will be like once they get to the interchange. Switching the 2D map background to aerial photography or using the street level 3D photography can help in some respects but it falls short once a building is entered (such as a railway station or more importantly an underground station).

This project attempts to address these issues by rendering an interchange as a 3D model with clear separation of levels, that makes it much more obvious where stairs, escalators and lifts are required without the need for a textual description. When a route through the model has been calculated, the user can 'fly' or 'walk' the route by starting an animation which follows the route from origin to destination. In a future version the user's position will be tracked in real time to navigate the route like a sat nav.

A secondary problem is the lack of information about the availability of roadside footpaths and the disappointing lack of a walk planner that uses footpaths rather than road centre lines.

Source data

The choice of source data has a large bearing on the project. Some data sets are country-centric, some carry large licence costs and almost all are 'closed' (i.e. the data cannot be edited by the user). The latter was deemed to be the biggest hurdle because none of the available data sets contains enough detail about interchanges to produce the desired 3D models. A new system would need to be created to store and manage additional data before the real task at hand could be addressed – the 3D drawing.

OpenStreetMap[1] was eventually selected because it provides a world view (although sometimes incomplete). It also provides the ability to add extra data. OpenStreetMap has a further advantage in that some indoor interchanges have already been mapped, such as Newcastle Monument Metro Underground Station shown in **Figure 1**.

3D drawing

Before any 3D drawing could be done a technology needed to be selected. A technology that was portable to various devices was desirable. HTML5 Canvas was chosen, with the ThreeJS[3] open source javascript library. This combination has the benefit that, on some devices, hardware acceleration can be used to render the 3D model (via WebGL[4]) and, as a fallback, the three.js library will perform software rendering in 3D.

There were two key issues relating to the actual drawing; Interpreting the source data and converting road centrelines into 3D roads.

As the tool only draws a 3D model of approximately one square kilometre, the interpretation issue was relatively easy to solve by converting the data into a common format as it is being loaded.

Road centrelines were converted into solid roads by creating a pair of lines parallel to the centreline, each half the road width away. The intersections of these lines with the parallel lines for the next section of road were then calculated to create a seamless join at junctions.

When these lines were used to draw the roads in the 3D world, they worked well except for acute angles which caused spikes in the model, short links which caused gaps to appear, gaps appearing at the top/ bottom of stairs and lifts which could not be rendered because of their vertical nature (they were eventually drawn as blue cylinders).

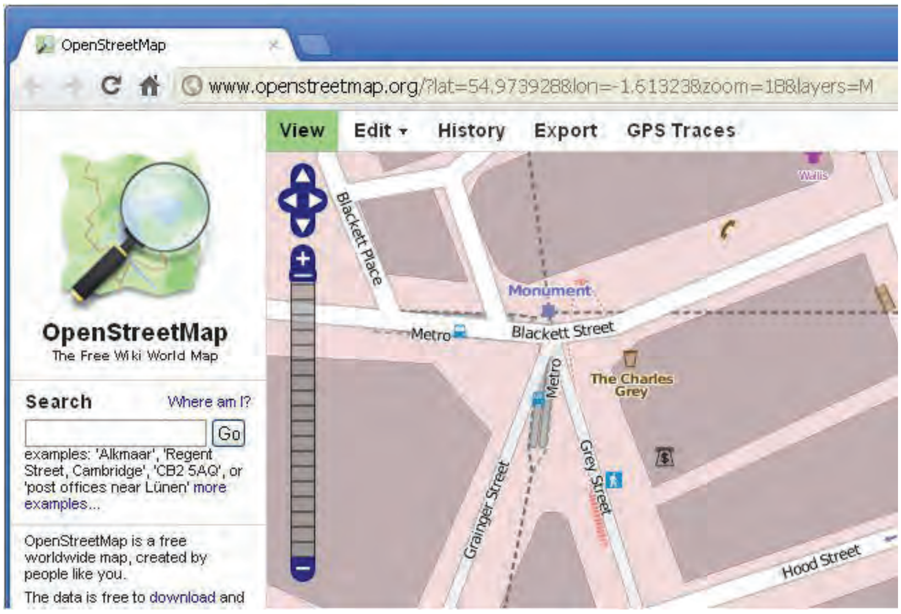

Figure 1. Source data for Newcastle Monument Underground Station[2]

Figure 2. 3D Presentation of Newcastle Monument Metro Underground Station. Entering via stairs on Grey Street.

Key

Black:	Roads	Green dot:	Represents the traveler when animating
Red:	Footpath	Blue S:	Marks the requested start point
Purple:	Stairs	Blue E:	Marks the requested end point
Light blue:	Lifts	Blue bus:	A bus stop
Dark blue:	Rivers and lakes	Grey gate:	A traffic barrier
Green:	Planned route		

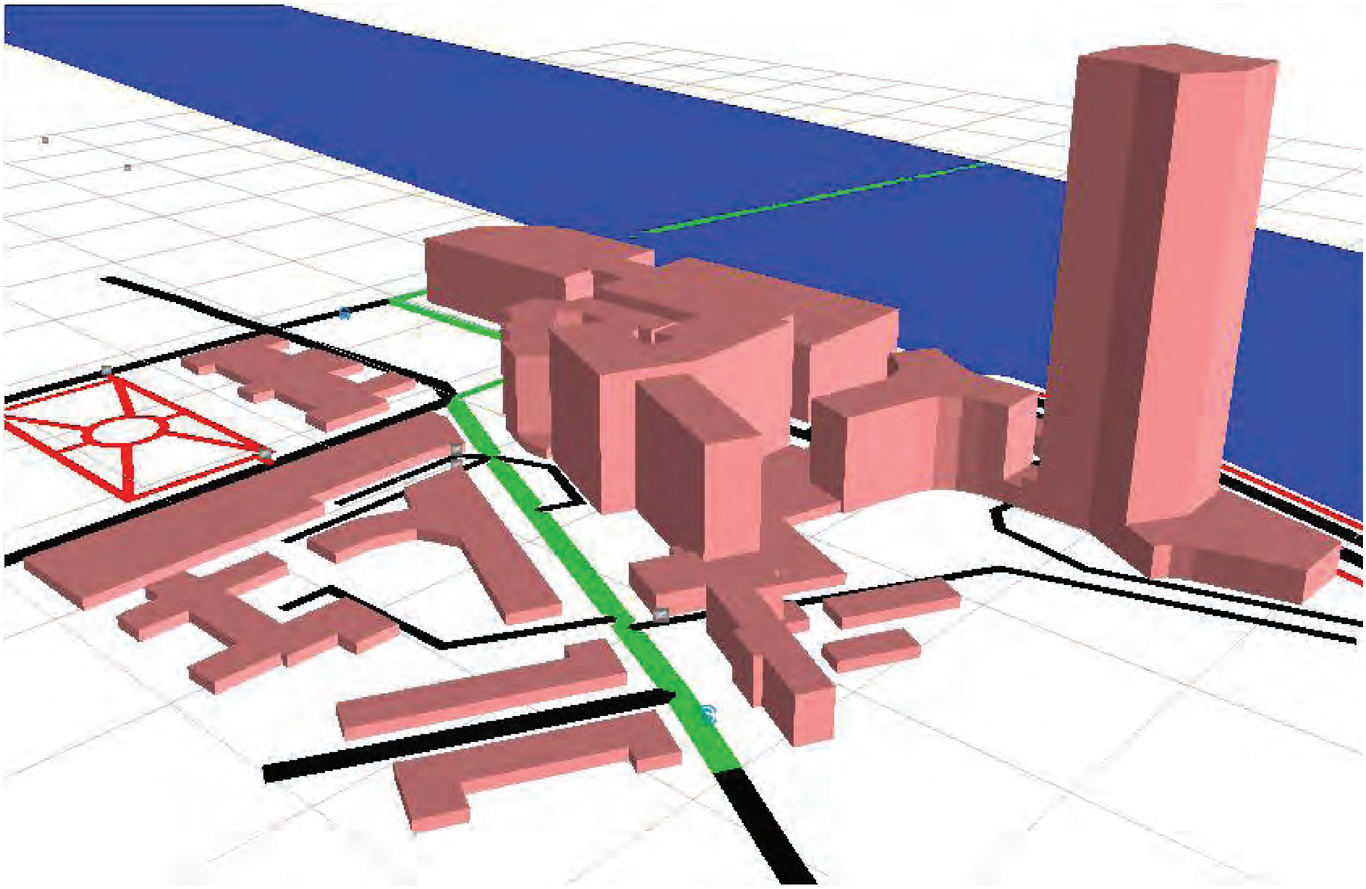

Figure 3. Figure 3 Millbank Tower, London[5]

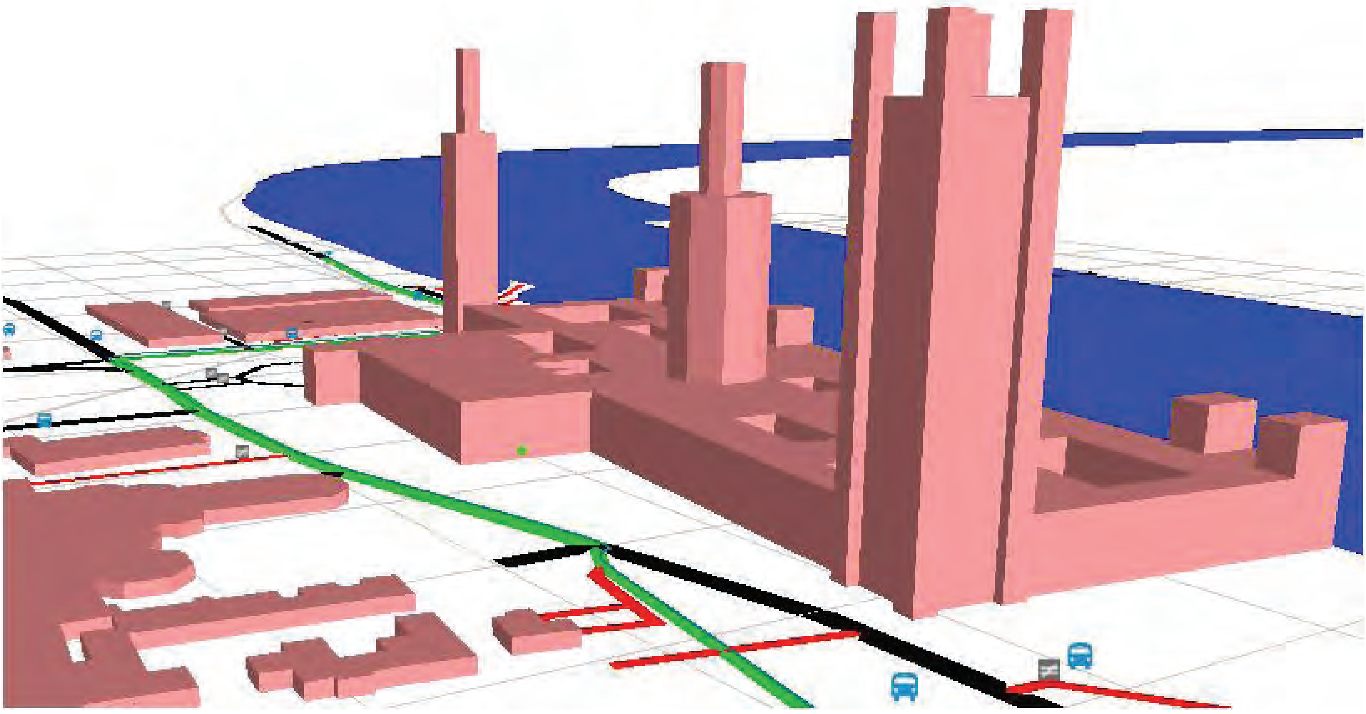

Figure 4. Westminster, London[6] (Height data exists for the palace but not the abbey)

Walk Planning

A simple walk planning algorithm was added for the initial version of the project to calculate a shortest path along road/path centrelines. The only additional feature added at this stage is the ability to avoid stairs, causing the algorithm to use lifts, ramps, or return 'no route found'. The user can select their start and end points by clicking on the map or selecting one of the bus stops rendered as billboards in the 3D model.

Preliminary work has started to plan on the footpaths instead of road centrelines. A new attribute was added to OpenStreetMap to indicate whether a road has a footpath on the left, right or both sides, or has no footpath at all. This is used to render a 2D map of all footpaths to show what is available for routing, shown in **Figure 5**. The key issue to address is where to allow road crossing (in addition to the pedestrian crossing defined in the source data).

Navigation

The ability to 'fly' (3rd person sky view) and 'walk' (1st person street view, see **Figure 6**) has been developed, but tracking the navigation of a route needs to be added. Research suggests two options for indoor navigation; using wifi router triangulation or dead reckoning. However, wifi is unlikely to work in an underground station. A Munich University project[8] has shown some promising results with indoor dead reckoning using a mobile phone's digital compass and accelerometer. A simple approach may be the best solution – show the 'walk' animation in real time and ask the user to press a button each time they come to a corner. If they arrive sooner it will advance the animation and if they arrive later it will continue the animation.

Figure 5. 2D Rendering of footpaths in Epsom[7]

Key

Red:	Railway Lines
White:	Road
Black:	Footpaths
Green:	Pedestrian Crossings

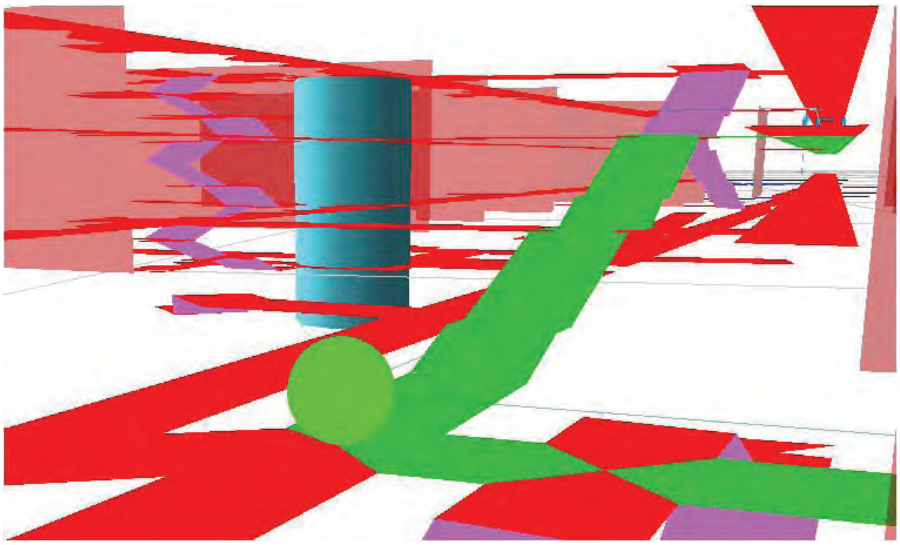

Figure 6. Going up stairs inside Munich University[9]

Conclusion

The work produced so far has generated quite a bit of interest from various sources: Traveline[10], (the UK public transport partnership) have added it to their list of desirable things for the future and the Atkins Pavement Design Team have suggested potential for integrating it with their 2D line of sight product.

The next steps are to find a partner to assist with funding and then to improve the quality of the rendered images (potentially with textures), migrate to mobiles phones, add navigation tracking and develop the pavement walk planner.

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