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Technical Journal

Papers 067 - 080

Plan Design Enable

FOREWORD

Welcome to the fifth edition of the Atkins Technical Journal.

Once again there has been a great response to the call for papers with those published spanning the majority of our business units and covering an excellent mix of accomplished projects and thought leadership. With the 2012 Olympic and Paralympic Games, ever nearing, it is timely to read how we are leading the way in controlling excessive crowd-induced motion of grandstands in general and how we are promoting innovative design on the Olympics, jointly with the Olympic Delivery Authority. Other papers show how we are increasingly providing thought leadership on climate change and sustainability in all sectors of what we do; our Intelligent Transport Systems (ITS) business is leading the way in looking at how ITS can drive down carbon in transport and help us adapt to the effects of climate change, while papers such as that on gas migration risk and the bridges sustainability index show that building sustainability into our designs and advice to clients is business as usual for us.

I hope you enjoy the selection of technical papers included in this edition.

Chris Hendy
Chair of Technology Board

Highways & Transportation

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Footbridge design for the Dubai Metro light rail project

David A Smith

Regional Head of Bridge Engineering

Highways & Transportation

Tony Smith

Senior Engineer

Highways & Transportation

Manuela Chiarello

Engineer

Highways & Transportation

Alan Ho

Engineer

Middle East & India

Abstract

The Dubai Metro Light Rail scheme is a flagship project in the United Arab Emirates which is the longest fully automated rail system in the world. The first line of the rail system was opened in September 2009 with a second line due for completion in 2010. These first two lines include 35 elevated stations along their combined 76 kilometre length. This paper describes the design and construction of the steel truss footbridges developed as part of the station context planning work for all of the elevated stations. The footbridges are all fully enclosed, air-conditioned corridors with the widths of many dictated by the provision of automated walkways. A key aspect to the footbridge design concept was to develop a modular system that could be rapidly designed for scores of differing span arrangements to be suitable for each unique location. Several of the footbridge truss elements comprised structural hollow sections, and the new rules in Eurocode 3 were adopted to design many of the connections between such members. With simply supported spans up to 45 metres long and external cladding creating irregular shaped cross sections, wind tunnel testing was required to demonstrate aerodynamic stability of the footbridges. Several erection methods were also considered to minimise the installation time of completed footbridge spans over the major Dubai highways, and the pre-camber and deflection analysis associated with the methods adopted for lifting were also important aspects considered in the design. Other critical design issues resolved included the design of fillet welded connections in place of full strength full penetration butt welds and the design of several special spans for connecting into non-standard stations and entrance structures.

Introduction

In July 2005, the government of Dubai Road and Transport Authority (RTA) awarded a design and build contract to the Dubai Rapid Link (DURL) consortium for the construction of the first stages of the Dubai Metro Red and Green Lines. The DURL consortium comprised the Japanese companies Mitsubishi Heavy Industries, Mitsubishi Corporation, Obayashi Corporation and Kajima Corporation together with Yapi Merkezi of Turkey. Construction of the infrastructure and stations was the responsibility of a joint venture between Kajima, Obayashi and Yapi Merkezi (Japan-Turkey-Metro joint venture or JTMjv). The JTMjv appointed Atkins as their designer in 2006 and station context planning design work commenced in August 2007.

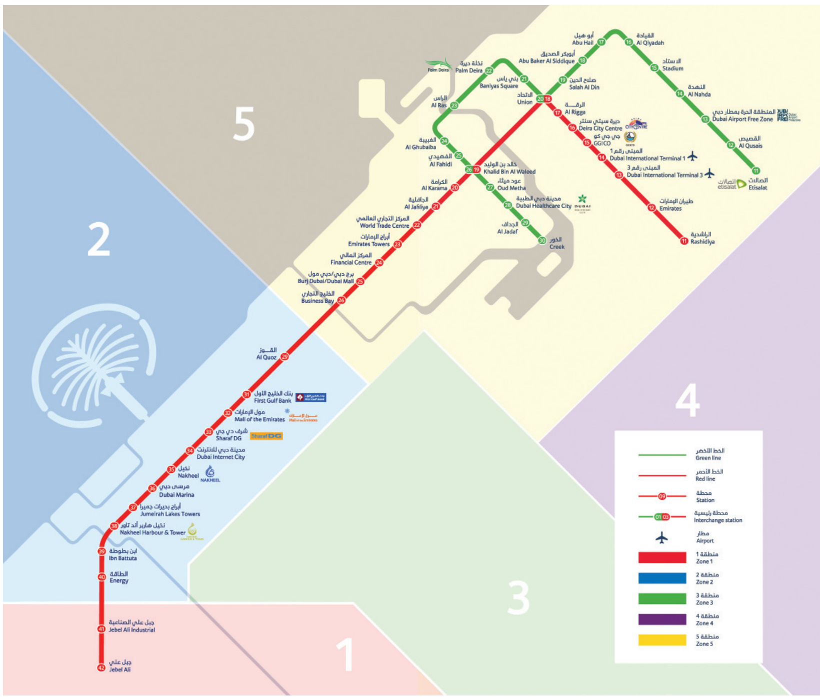


Figure 1 - Red and Green Line Route maps

Figuire 2 - N-truss type footbridge

The Dubai Metro is a driverless, fully automated metro network and is the longest fully automated rail system in the world. The first section of the Red Line was completed in September 2009, and is due to be followed in 2010 with the first section of Green Line.

Detailed metro route maps are illustrated in Figure 1. These first stages of Red and Green Lines included 35 elevated stations. Each station needed to be connected to entrance pod structures or car parks either side of busy, often congested existing infrastructure, with a requirement for unique span arrangements for a family of over 200 footbridges. Each footbridge comprised a fully enclosed, air-conditioned corridor with the widths of many dictated by the provision of automated walkways. Additional structural steelwork was also provided above piers where footbridge direction change occurred in plan. A key aspect to the footbridge design concept was to develop a modular system that could be rapidly designed for scores of differing span arrangements to be suitable for each unique location.

To give an overview of the extent of Red Line and Green Line footbridges there is in excess of:

- 200 spans
- 70 spans with travelators (automated walkways)
- 9000 tonnes of structural steel
- 4.5km total length
- Approximately 25,000 m^2 of deck area.

All spans were based on an N-truss design (Figure 2) with braced floor and roof planes. Three basic footbridge types were designed:

- Narrow Standard - 4.5m width between trusses, maximum 45m span
- Travelator/Wide Standard - 7.7m width between trusses, maximum 45m span
- Low Height Spans where footbridges passed below the metro viaduct - 4.5m width between trusses, maximum 26m span.

Several of the footbridge truss elements comprised structural hollow sections, and the new rules in Eurocode 3[1] were adopted to design many of the connections between such members. With simply supported spans up to 45m long and external cladding creating irregular shaped cross sections, wind tunnel testing was required to demonstrate aerodynamic stability of the footbridges. Several erection methods were also considered to minimise the installation time of completed footbridge spans over the major Dubai highways, and the pre-camber and deflection analysis associated with the methods adopted for lifting were also important aspects considered. Other critical issues resolved included the design of fillet welded connections in place of full strength butt welds and the design of several special spans for connecting into non-standard stations.

This paper describes the design and construction of the steel truss footbridges connecting the elevated stations to their entrances. Separate papers[2,3] cover the design and construction of the viaduct superstructure and substructure.

Footbridge form and development of preliminary design

The proposed form of the footbridges was architecturally-led in appearance. Figure 3 gives an artist's impression of the proposed footbridge at the conceptual design stage and much of this form was retained in the final detailed design. The initial preliminary structural design and architectural perspective only included the 4.5m wide spans however, and was based on the elevations being glazed over their full height. Hollow sections were chosen for the truss chords and verticals with tie bars preferred by the architect for the truss diagonals.

The total distance between entrance pods and station was up to 400m at some stations and a decision was made at an early stage in the design process to include travelators when travel distance exceeded about 100m. Two 1.4m wide travelators were added to the cross section with a normal walking zone between so as to provide the same deck area if one travelator was out of use.

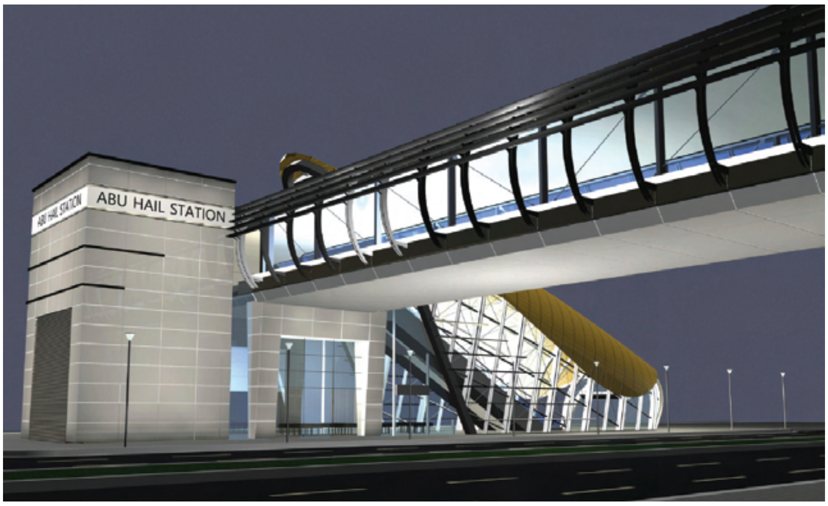
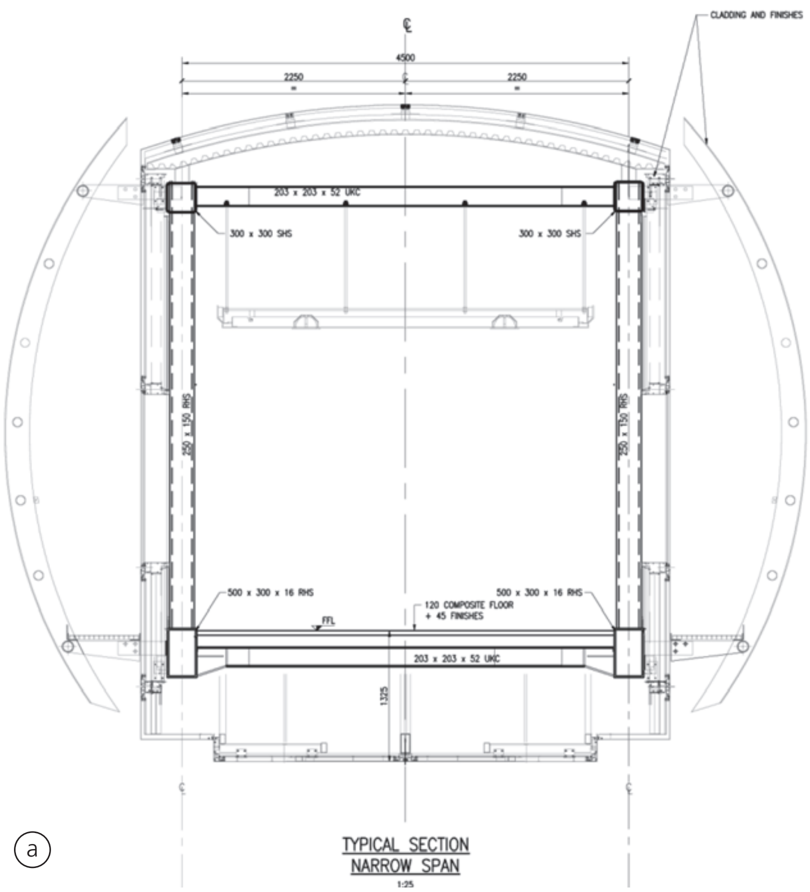


Figure 3 - Initial architectural image

Including the travelator structure and clearances, this increased the overall width between truss centres to 7.7m (as shown in Figure 4).

Deflection of the span under live loading was critical to the successful functioning of the travelator. Preliminary assessment showed that single tie bar diagonals would not have sufficient axial stiffness to limit this deflection. Twin tie bars were considered but discounted because of concerns over load sharing and the space required to accommodate the end fixings. Hollow section diagonals were therefore adopted. Diagonals to the narrow spans were also changed to hollow sections to give a similar appearance to all footbridges throughout the project.

Another change from the initial preliminary design was to move the mechanical and electrical (M&E) services from below the floor to above the roof beams. The total weight of services also increased. The M&E services included pipes supplying chilled water to the stations and the total load could be up to 10kN/m.



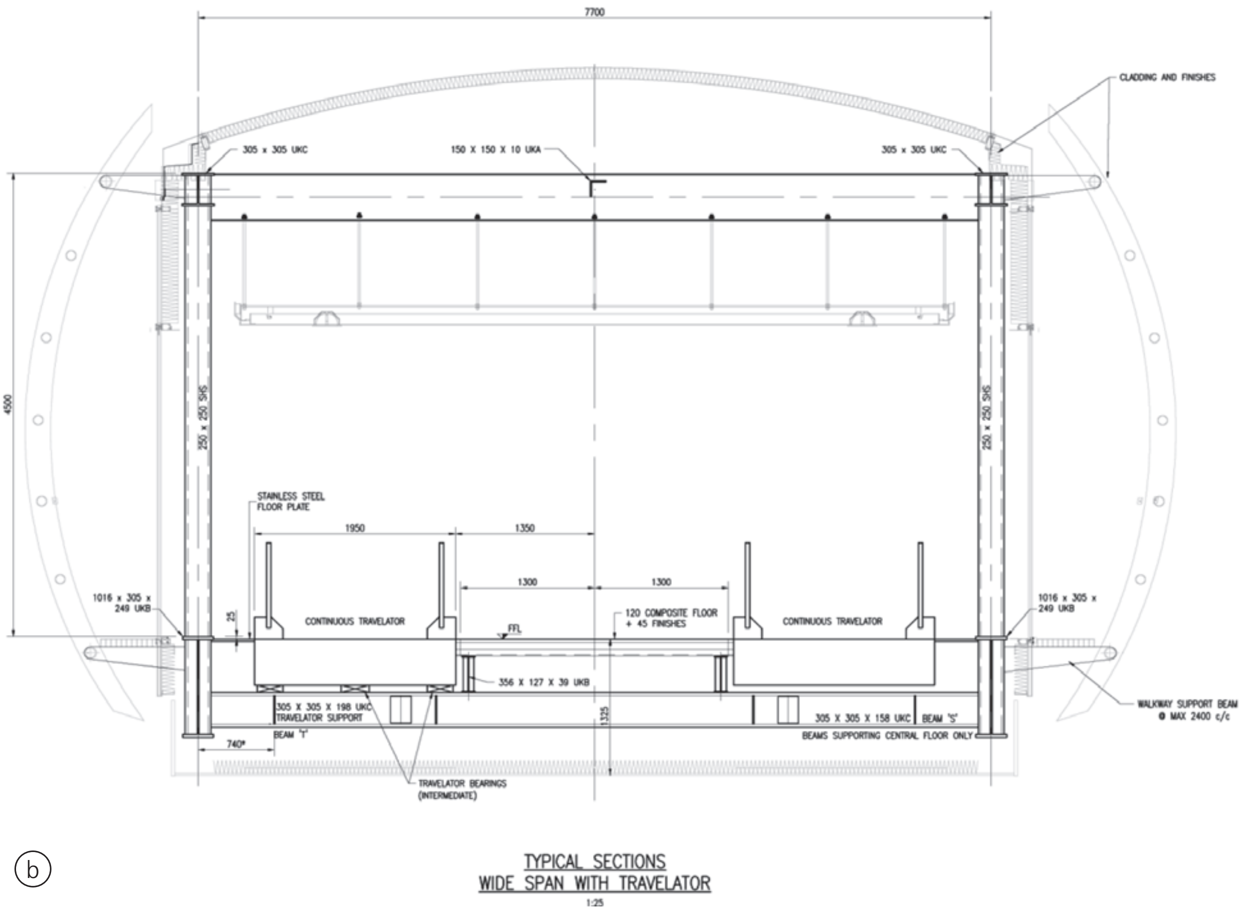


Figure 4 - Typical footbridge cross sections (a) Narrow deck (b) Wide deck

Figure 5 - Typical standard pier arrangement during construction

Placing these services at roof level had significantly adverse effects on the structural dynamics. Also, at the request of the contractor, much of the steelwork not visible in the completed work was changed from hollow sections to rolled sections. Figure 4 illustrates the final cross sections adopted for the wide, travelator spans and the standard narrow spans.

The machinery end pits for the travelators also needed to be located within the footbridges. These pits extended to approximately 1100mm below the floor level and occupied the space vacated by the M&E services. Their presence impacted on the local design of truss elements in this location as well as affecting precamber calculation, dictating travelator setting out and influencing the erection methodology.

Substructure design

To meet the tight footbridge erection programme, it was necessary to complete the design and construction of the footbridge substructures within a 12 month period. To accelerate the design process, the footbridge substructure designs were developed from the viaduct substructure designs already completed and mostly constructed.

Figure 6 - Special pier head

The piers generally comprised 1.75m diameter columns founded on 2.2m diameter monopiles, embedded into the ground between 10m and 25m. A typical standard pier is shown in Figure 5. Using the same design basis as used for the viaducts[3], the pile vertical loading resistance was provided entirely by skin friction without any reliance on end bearing. The use of a monopile solution minimised the excavation requirements for the foundation construction and limited the duration of road closures or carriageway possessions in the urban environment. Design and detailing issues associated with monopile construction are discussed further in Smith and Hendy[3].

Due to the lack of availability of larger drilling rigs in the early stages of construction, 2.2m diameter piles were the largest bored piles available for use. At locations where the 2.2m diameter monopiles could not be designed to give sufficient strength (especially at several junction locations where bending moments at the base of the pier were high due to the large eccentricity of the bearings), twin 2.2m diameter piles were used as a group.

Adopting twin pile groups was not always easy however, due to the many conflicts between the enlarged foundation footprint and the existing utilities or highways. Several bespoke, twin-pile foundation solutions were therefore designed to solve specific conflicts. Such solutions included:

- Designing an opening in the middle of the pile cap (in elevation) to allow water mains pipes to pass through
- Offsetting the pile group from the pier centre to avoid gas mains
- Rotating the orientation of the piles in plan to avoid existing highways.

Later in the construction programme, larger piling rigs became available as the Green Line viaduct substructure work was completed. 2.6m diameter piles were introduced for the footbridge foundations to replace the twin pile groups and minimise the need for bespoke pile group designs at many locations.

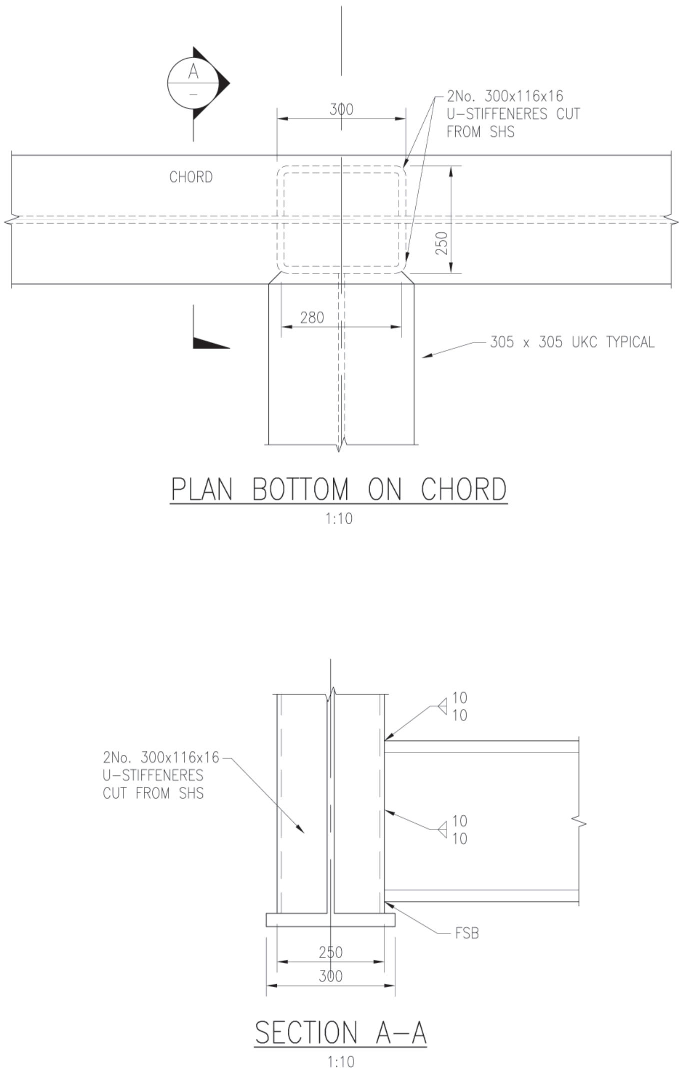


Figure 7 - U-shaped stiffeners

Throughout the project, the contractor encountered considerable difficulties in arranging major utility diversions (permanently or temporarily) in a timely manner to meet the overall construction programme. The exact location of many of these utilities was not discovered until trial pits were completed once access had been provided. Thus the decision on final piling locations was always a difficult compromise between avoiding clashes, minimising utility diversions and finding a suitable position for structural support such that as many standard footbridge designs as possible could be used.

The footbridge pier heads which transfer the vertical load from the footbridge truss to the pier columns were designed with different shapes to suit the orientations of the footbridge spans in plan, according to the design alignment.

The most commonly used pier heads were modified from the form of those used for the viaduct pier heads which comprised prestressed concrete beams formed from precast concrete shells with in-situ concrete infill. Due to the lower overall loads however, the footbridge pier heads did not require presetressing and were constructed from reinforced concrete alone.

To further shorten the construction period, the need for shear links between the outer shells and the inner concrete infill used in the original design was eliminated, by treating the precast shell as a non-structural, permanent formwork component. To save further time in construction, the standard footbridge pier heads were constructed from the same outer curved profiles used for the viaduct pier heads.

At locations where the footbridge trusses required a wider angular change in plan, or at locations where more than two footbridges combine at a junction above the same pier, the bearing locations from the standard trusses could not be accommodated within the extent of the standard pier head. Bespoke in-situ reinforced concrete pier heads were designed for such junctions. The appearance of these bespoke pier heads was developed in conjunction with the architects to ensure their style was consistent with the standard pier heads however. The complex curved profiles of the standard pier heads were replaced by a combination of flat surfaces to reduce the complexity of the formwork on site (Figure 6).

The footbridge trusses are supported on the pier heads along their edges and a steel junction frame was uniquely designed on top of the junction pierheads to fill in the gap between the trusses. Such a structural arrangement significantly simplified the design and construction of the footbridge trusses themselves, which led to swift design output whenever design changes were required.

Standard deck design

The superstructure steelwork was designed to BS 5400:Part 3[4] including the BD 13[5] modifications. Connections involving hollow sections were designed to BS EN 1993-1-8[1] with partial factors adjusted for use with BS 5400 load combinations.

BS EN 1993-1-8 includes rules and formula for design of connections between hollow section components and also between hollow sections and I-beams and plates. These rules are generally those developed in various CIDECT (Comité International pour le Développement et l'Etude de la Construction Tubulaire) publications and given in CORUS/British Steel design guides, but include several arrangements not in the design guides. Even with the additional details covered by BS EN 1993-1-8 some connection details were not fully covered. For example, floor beams were eccentric to the centre line of the chord and some connections had both I-beam and hollow sections framing in to a further hollow section. Where these occurred the formulae had to be used with care.

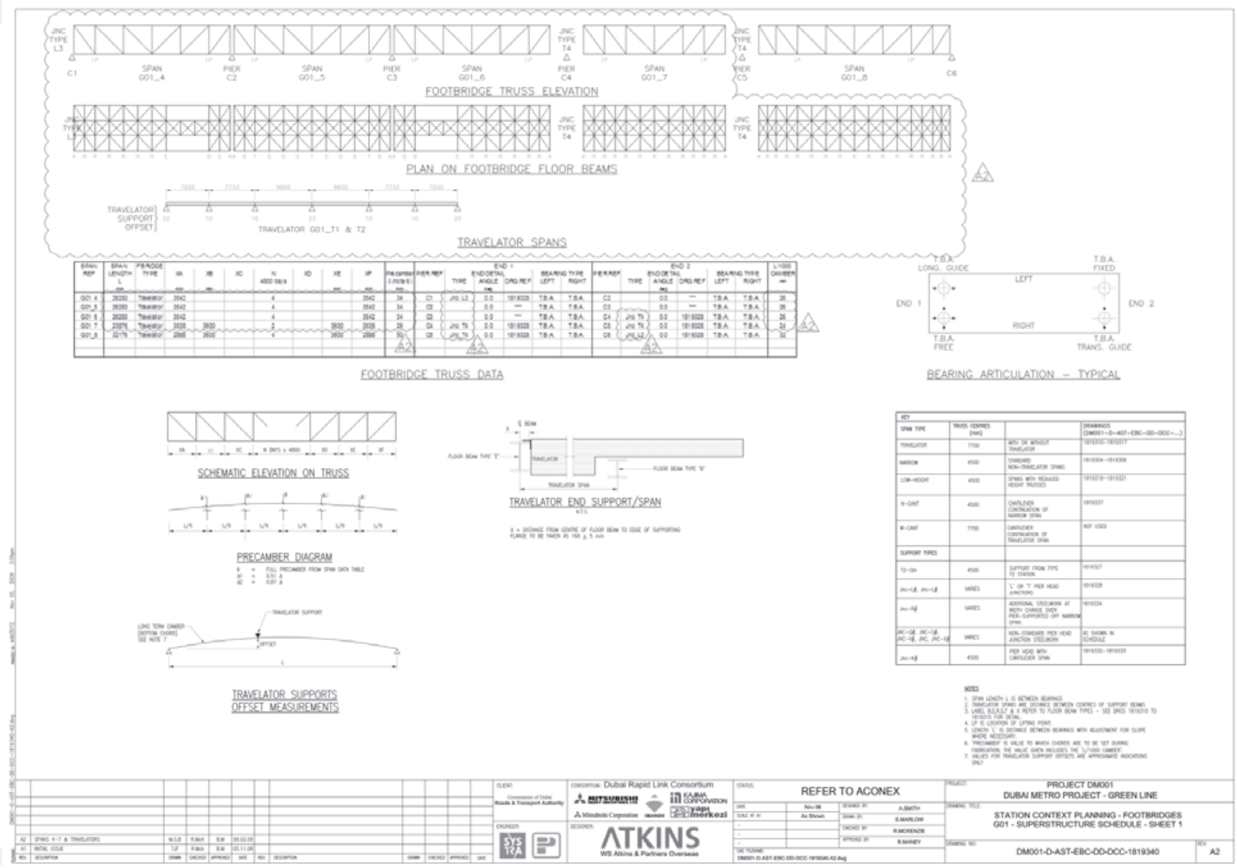


Figure 8 - Typical automated schematic CAD drawing

The wide spans had hollow section verticals and diagonals framing in to I-beam chords; these connections can be inefficient since much of the axial loads from the verticals/diagonals need to pass into the I-beam web. To enhance the capacity, U-shaped web stiffeners cut from hollow sections were provided to continue the verticals and inclined web plates were added to continue the line of the diagonal as shown in Figure 7. The U-shaped stiffeners also provided a connection point for the transverse floor and roof beams.

Several inconsistencies and uncertainties were found in using EN 1993-1-8 and these were raised with the Standard drafting committee which resulted in a (then) unpublished corrigenda being made available. The corrigenda were quite extensive including some significant changes that hadn't been anticipated. Fortunately these changes did not have an adverse effect on the large number of connections already designed.

The structural analysis was based on simple 3D space-frame models without joint eccentricity. For standard spans only the longest span was analysed but most special spans were analysed individually. Lateral bracing diagonals were considered as tension only following a contractor requested change from hollow section to angles. Destabilising forces (FR and FS) were considered separately on a plane frame model of the end ring frames and combined with those from the 3D model for design of sections and connections.

For the travelator spans, two deck configurations were considered – travelators running the full length of the span and no travelators within the (wider) span. The latter gave the worst design load for most elements since live load was carried over a much greater area. Most real spans had some lengths with full-width deck.

Design automation for specific locations

Span lengths were dependent on where piers could be located which in turn depended on ground level and sub-surface obstructions. As a result nearly every span was a different length. The basic truss bay length between verticals was set at 4.8m to align with glazing/cladding joints at 1.2m centres and external feature ribs at 2.4m centres. Transverse roof beams were aligned with the vertical truss members; floor beams also aligned with the verticals with additional beams at half spacing (2.4m nominal) but floor beams had to be excluded at travelator pit locations.

Clearly most spans would not be an exact multiple of 4.8m so several options were considered for dealing with this length variation. The final solution adopted was based on the following logic:

- Span symmetric – even number of bays
- No bays greater than 4.8m long
- End bay not shorter than 2.4m (if possible)

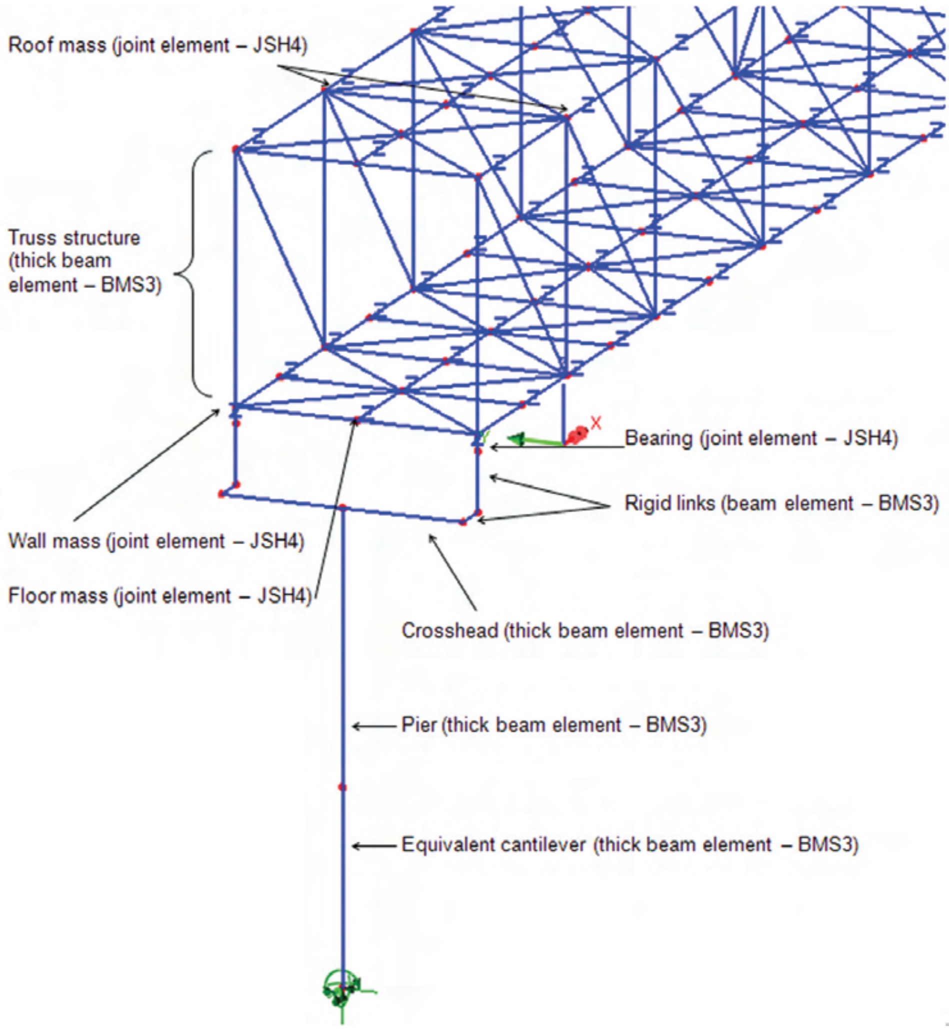


Figure 9 - Footbridge idealisation for dynamic analysis

- Allow 2nd, and in extreme cases 3rd, bay from end to be 3.6m
- End bay should be shorter than 2nd bay.

Whilst the contractor/fabricator was responsible for producing the fabrication drawings for individual spans there was a requirement for the designers to provide data on bay length and travelator positions. Travelator spans also contained 5 different floor beam designs whose position depended on exact travelator location. Initially it was planned to provide this information in tabular form only on drawings but it soon became clear that some drawn information was required to support the tables. Faced with having to draw, and in many cases revise several times, the layouts for over 200 spans, it became obvious that this required automated drawing production. Span data was already supplied by the substructure team in spreadsheet form and further spreadsheets were used by the superstructure design team to determine bay lengths and position travelator supports. These spreadsheets were used with minimal amendment to provide data for an AutoCAD macro.

Typically a layout drawing covered a run of up to 5 spans at one station and provided elevation and floor level plans for each span together with a diagrammatic elevation of the travelator. Plans and elevations were aligned in terms of chainage giving a visual appreciation of their relative positioning. Travelator span lengths were included on the travelator elevation but all other dimensions were covered by a table linked to the spreadsheet. The floor plan also included the floor beam type references. Figure 8 shows extracts of a typical automated drawing. With the spreadsheet data available, production of the drawing itself only took a few minutes - it took longer to add revision clouds.

Gradients and changes in direction in plan were not shown, but stations and junction types were indicated. As the resulting drawings were standard AutoCAD drawings, final ‘finishing’ adjustments such as adding special end details could easily be incorporated. The drawings also included information on pre-camber, lifting point position and cross references to other drawings. Bearing type details were also included initially but this information was later transferred to a separate drawing.

As noted previously, travelators were deployed for walk distances greater than approximately 100m. During the design, consideration was given to using travelators up to 100m long and also to containing travelators wholly within a single span. The former was rejected as it would still have left long walking distances in the event of breakdowns; the latter whilst removing travelators crossing movement joints resulted in lots of

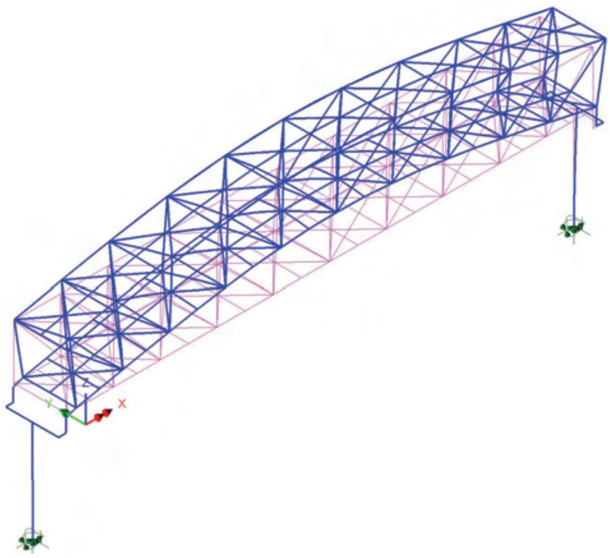


Figure 10 (a) - Natural frequencies - Torsional mode (1.41 Hz)

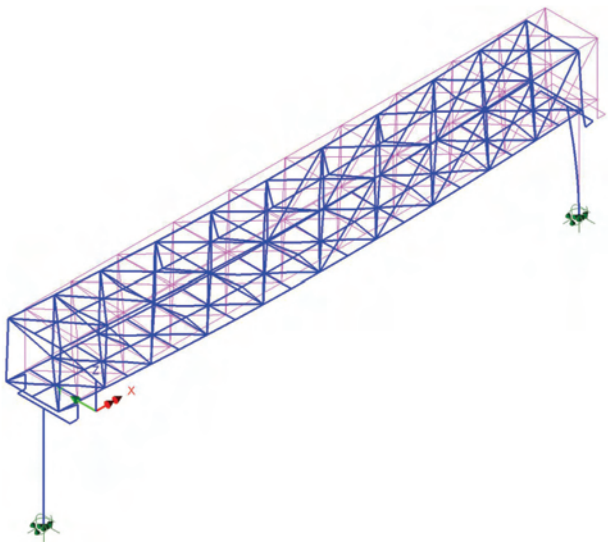


Figure 10 (b) - Natural frequencies - Lateral mode (1.59 Hz)

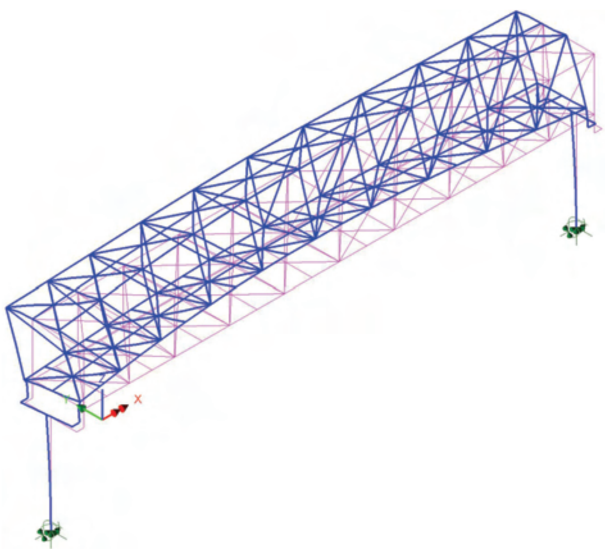


Figure 10 (c) - Natural frequencies - Vertical mode (1.97 Hz)

short travelator lengths and was also rejected. The final solution involved travelators up to 50m long, continuing from one span to the next if needed.

The travelators have their own side trusses which are supported of the main footbridge floor beams at a maximum of 9.6m centres using simple bearings. End pit spans were limited to 7.2m but had to be at least 5.2m long. Where travelators passed over joints between spans it was necessary to keep the travelator support point as far from the joint as possible in each span to minimise the change in grade of the travelator. The travelator was never supported at the footbridge truss end ring frame.

Aerodynamic stability

In-keeping with the basic standard designs, simply-supported spans up to 45 metres in length were checked for susceptibility to aerodynamic excitation against the simplified rules in BD 49[6].

Natural frequency analyses were undertaken to obtain the primary vertical, lateral and torsional modes of vibration. Finite element models were developed using LUSAS[7] with 3D thick beam elements used to model the primary structural members of the steel truss, concrete pier head, pier column and foundation as illustrated in Figure 9. Joint elements of appropriate stiffness and freedoms were used to represent the articulation of the bearings; joint elements with an appropriate eccentricity were also used as lumped masses at specific locations to represent the weight of cladding, services, walls and floor and roof finishes. Exact locations of the lumped masses were refined in steps from an initially crude, global application to a more refined distribution in order to avoid local buckling modes which had the potential to modify the overall response of the structure. Eccentricities between the pier head and the footbridge floor levels were also taken into account using rigid link elements. The soil-structure interaction of the piled foundations was modelled using equivalent cantilevers.

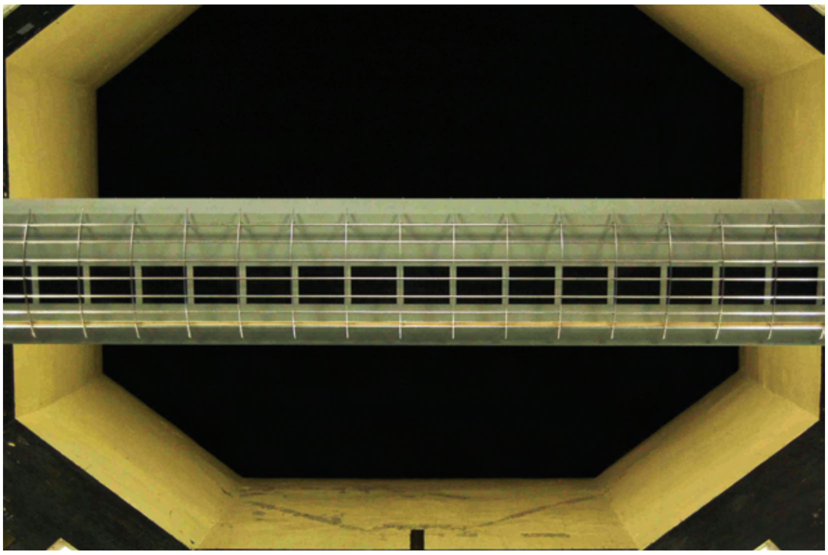


Figure 11 - Footbridge model in wind tunnel

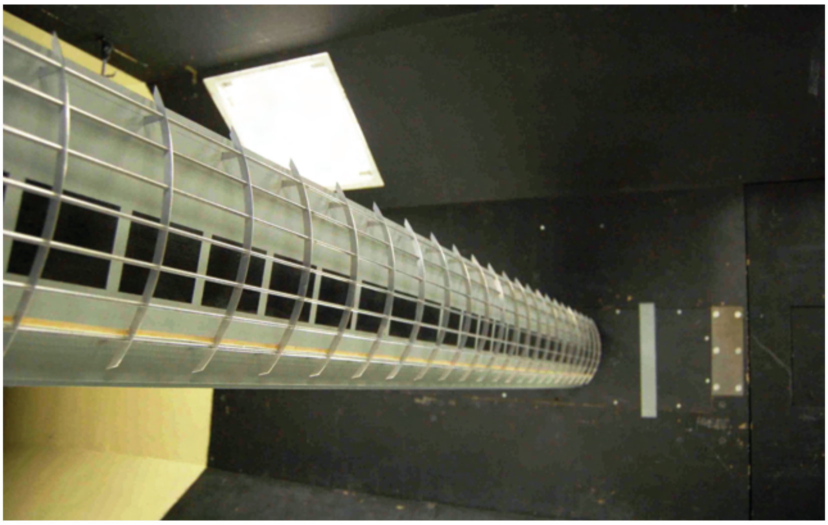


Figure 12 - Footbridge model in wind tunnel

The fundamental torsional (combined with transverse), lateral (longitudinal) and vertical modes obtained for the footbridge are illustrated in Figure 10, with frequencies of approximately 1.41Hz, 1.59Hz and 1.97Hz respectively. These dominant modes were identified on the basis of an analysis of total mass participation over a range of frequencies for each excitation vector.

The fully-enclosed nature of the footbridges together with the irregular-shaped cross sections due to the geometry of the external cladding meant that the footbridges fell outside the scope of BD 49/01. Nevertheless, using the rules for an initial calculation suggested that, whilst the footbridges satisfied the limits for vortex excitation, they didn't comply with the divergent amplitude response checks for galloping and stall flutter.

It was therefore necessary to demonstrate by other means that the wind speed required to induce the onset of galloping was in excess of the critical wind speed limit.

A wind tunnel test was subsequently commissioned to determine this and also to calculate an equivalent drag coefficient for the specific section geometry. The wind tunnel tests were carried out in BMT Fluid Mechanics Limited's aeronautical wind tunnel using a 2 dimensional, 1:25 scale section model of the bridge deck, illustrated in Figures 11 and 12.

Static load coefficients were measured in smooth flow for wind angles in the range of ±5°. Divergent responses were investigated in smooth flow for a series of configurations (with and without a ground plane) for a range of wind angle and structural damping.

The footbridge was found to be stable up to wind speeds of approximately 70m/s (sufficiently in excess of the maximum design wind speed of 54m/s) based on the first bending frequency of the bridge at approximately 2.0Hz.

For a logarithmic decrement due to structural damping of 0.03, considered appropriate for structures like the Dubai Metro footbridge, the vortex shedding responses from the testing were negligible.

The footbridge was also checked for vibration serviceability to BD 37[8], to ensure that the structure was not excessively excited by pedestrian use.

Footbridge erection

Most primary fabrication took place outside Dubai. Originally it was considered that large sections of side truss could be fabricated and delivered to site laid on their side with a minimum of site welding. In practice, delivery was in much smaller components with increased site welding. Many of the metro stations are situated along Sheikh Zayed Road (SZR), a major dual carriageway road with up to 6 lanes in each direction (as indicated in Figure 13).

Carriageway possessions for footbridge erection and construction were clearly very limited and so these spans were constructed including cladding and finishes to the side of the road on temporary trestles and then positioned on the piers during overnight possessions using self-propelled modular trailers (SPMTs). Figure 14 shows a typical footbridge span being erected using a SPMT.

With most of the permanent load in place when lifting the spans with the SPMT, lifting points needed to be under a truss vertical and end clearances meant these were often 2 bays in from each end. The need to brace the compression chord above the lifting point meant that lifting points could not be within the length of the travelator end pits - where transverse beams and bracing had to be omitted. Positioning of travelators was therefore inter-dependent on method of erection and span geometry.

Figure 13 - Completed footbridges over Sheikh Zayed Road

Mobile cranes were used to erect the footbridges at other locations where existing traffic flow and infrastructure was less congested (Figure 15). Generally, the external cladding of the footbridges erected by mobile crane was constructed progressively after the steel trusses were rested onto their permanent bearing positions.

Design of non-standard footbridge spans and details

It was not surprising that just three basic, standard design arrangements were not able to cope with all 200 plus spans and several special spans and details were required. A few spans required small end skews and these were easily accommodated by varying the length of the end bay on each side. Short cantilever spans off the end of main spans were also used mainly to take the footbridges into the end of entrance pods where no suitable support member was available. Similarly, a short cantilever off junction pier-head steelwork was also needed.

As discussed above, the footbridges were generally supported on their own piers independent of station or viaduct substructure, but spans into Type T2 stations were supported on

Figure 14 - Footbridge supported on SPMT

the station structure itself. At the interface the station roof is curved in both plan and section. Since plan curvature was small, the bearings were offset slightly from the station beam web to give a square end to the footbridge in plan. The curve in section however, meant that the top chords and end ring frame needed to be set back from the bearing location with short bottom chord stubs.

Two spans, one at each of Marina Station and Jameira Lake Towers Station, were to be built over stations for the Al Sufouh tram and needed pedestrian interface with the tram access. The exact tram alignment had not been established however, and so a footbridge pier could not be positioned within the station. An N-truss arrangement does not allow for side access within the span and it was necessary to modify these spans to include a Vierendeel truss section. This required larger chords and verticals; the bottom chord also had to be lowered to permit the footbridge deck to pass over it.

At Nakheel Station the adjacent footbridge spans had to be higher than the station concourse level to give clearance to the adjacent highways. A flight of stairs within the footbridge was introduced at the station end together with a lift for disabled access. Further complications for these spans were that the overall depth had to be reduced above the stairs and lift to fit below the metro viaduct and a station roof brace member had to pass through the bottom chord.

Also, for one span, the width had to reduce from being wide at the remote end to narrow at the station end with the width change taking place over a short distance within the span. Clearance between the roof brace and footbridge chord had to be sufficient to accommodate the independent flexing of both structures under transient loads. The exact location of the opening in the footbridge chord could not be finally established until the roof was erected.

A span with width change but without the additional complications of stairs and station roof brace was also required at Al Qiyadah Station on the Green Line.

This also included an end detail for direct connection to a Type T2 station.

At Al Karama station a span with 35° end skew was required meaning it needed one more truss bay on one side. Although the longest side of this span was close to the maximum standard span design length, accommodating the skew was mainly a detailing problem.

About one-third of the stations involved footbridge runs with change of direction or bifurcations; many of these were of simple 90° L or T form with most combinations of wide to narrow spans covered. These were designed as structural steel extensions mounted directly on the pier-head concrete and independent of the footbridge span steelwork. Figures 16 and 17 illustrate a typical example of this. Small angular changes were accommodated by tapered cantilever extensions to the span steelwork with larger angular changes using independent steelwork as for T and L junctions.

Design and construction problems

Primary section design was straightforward but connection design details were a critical issue, particularly the connection between the floor beam and vertical of the end frames. In normal working conditions the moment on this connection was not too problematic but future bearing replacement requires jacking under the floor beam remote from the main truss planes, generating very large moments in the connection.

Figure 15 - Footbridge erection using mobile crane

For the narrow spans there was sufficient space between floor level and top of pier to deepen the beam but the floor beams on travelator spans had to be lower for the travelator to pass over and could not be deepened sufficiently.

Hollow section verticals had strengthening plates added but it was still necessary to haunch the beam ends within the short distance between the travelator and end vertical.

Trial pits were required at most pier positions before they could be finalised. Inevitably some services were not where expected and piers had to be re-located or deleted and this led to continual needs to update the span layouts. The use of the automated drawing production process was essential to ensure timely delivery of these designs ahead of the tight construction programme.

Three separate steelwork fabricators were employed on the footbridges. Each had their own supply line and sought to use alternative sections more readily available to them. This was mainly for angle sections and hollow sections, angles in particular were often offered in unusual sizes or in steel other than BS EN 10025[9]. Alternative sizes could be easily checked for adequacy but steel grades needed further investigation to ensure adequate yield strength and ductility. Based on feedback from the outline design a plate girder alternative was included for the bottom chord (1016×305 UKB), with plate thicknesses proposed by one fabricator; a second fabricator was unable to source the plate thickness proposed by the first so a further alternative needed to be developed.

End welds to hollow section verticals, diagonals and chords were designed to be full penetration full strength butt welds but several spans were fabricated and erected with only fillet welds of inadequate capacity. The major problem this created was how to enhance the capacity given that many of the affected spans were over SZR with at least external cladding in place. Cutting out the welds and preparing for and providing butt welds was not an option with spans over SZR, since too much further strength would be lost and temporary propping could not be installed.

Figure 16 - Steel extensions above pier heads at footbridge junctions during construction

Figure 17 - Steel extensions above pier heads at footbridge junctions during construction

Discussions on site between designers' local engineers, welding specialists and welding staff determined that sufficient access space existed to the external cladding to allow fillet welds to be augmented. To limit temporary local loss of weld strength during over-welding, heat input needed to be controlled meaning small weld beads and multiple passes were required. All spans had been designed on a generic basis giving, in many cases, conservative values of member forces. In an effort to minimise the amount of re-welding several spans were analysed in detail considering their actual bay lengths and travelator positions.

Data from these spans was then used to generate algorithms from which welds for further spans could be rapidly assessed. Butt welds were used on all remaining spans not fabricated at the time the error was discovered.

The floor construction consisted of a reinforced concrete slab with permanent formwork topped by tiles with mortar bedding and the structural design specified this to be of constant thickness over the whole span, following the deck pre-camber.

Figure 18 - Completed footbridge (interior)

It was recognised early on that travelators needed to be linear between their ends but this should have been able to be accommodated by varying the show on the travelator parapet/hand-rail and adjusting packing below the travelator bearings. This requirement was not adhered to on a number of spans leading to excess deck slab thickness. Fortunately most local floor beam components had been designed for a thicker deck (as the deck thickness was later reduced in an effort to reduce dynamic mass) and actual span configurations gave a margin for global increase allowing most increases to be accepted. Two spans where the increase had been excessive did however need to be re-profiled.

Conclusions

The design of over 200 footbridge spans as part of the Dubai Metro station context planning was challenging. Great use of automation and a modular construction approach allowed for the frequently changing span requirements to be rapidly designed to a tight design and construction programme. All Red Line footbridges were completed in time for the opening of their respective stations in 2009 (Figures 18 and 19), with the Green Line footbridges due to follow in 2010.

Acknowledgements

This paper is published with the permission of the Dubai RTA and the JTMjv. The authors acknowledge Genta Tabe and his JTMjv team, particularly Naohito Nishikawa. They would also like to acknowledge Atkins' Project Director, John Newby, and his management team led by Ross McKenzie for their excellent co-ordination of the numerous design interfaces. Regular, close liaison with Atkins' team of architects, led by Adrian Lindon, was also vital to the successful delivery of the footbridge designs.

Figure 19 - Completed footbridges (exterior)

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Performance based seismic design of tall building structures: case study

Farshad Berahman

Senior Structural Engineer

Middle East & India

Abstract

The direct application of the traditional seismic design procedures in traditional codes to tall building structures can lead to poor structural forms, uneconomical structural designs, and, in some cases, to buildings that will not perform well in moderate and severe earthquake shaking. Recently efforts have been made to implement PBD (Performance Based Design) philosophy for seismic evaluation of tall building structures. In this regards, this paper presents the structural engineering design approach used to evaluate the seismic behaviour of Icon Hotel by implementing performance based design methodology. The analysis results identified the building weak points and also the building seismic behaviour during future anticipated earthquakes.

Seismic design of buildings in Dubai is traditionally based on the Uniform Building Code, UBC (ICBO, 97) and the seismic loads have been based on zone 2A. Several studies have been carried out to verify the proper seismic zonation of Dubai and the results vary from zone 0[3,14,2], zone 1[16,1] and Zone 3[22,10]. Most of the current structural engineers use the linear procedure, specifically the response spectrum analysis, for structural analysis and follow the seismic detailing requirement of the code to assure the ductility requirement under inelastic behaviour of the structure. However, buildings will undergo plastic deformations during major seismic events and therefore more accurate analysis is required to verify the basic seismic design assumptions and to get more useful information for design[18]. The direct application of the traditional design procedures in traditional codes can lead to poor structural forms, uneconomical structural designs and in some cases, to buildings that will not perform well in moderate and severe earthquake shaking[6].

More recently, guidelines for performance based seismic design of high rise buildings have been published by several organizations in Los Angeles[13], San Francisco[21], and CTBUH[6]. Research is also under way at the Pacific Earthquake Engineering Research Centre (PEER) for performance based design of tall buildings (the Tall Building Initiative). Dubai as the home of many tall buildings and the tallest tower in the world, has introduced a new code[7] to respond to the need to develop design criteria that will ensure safe and usable tall buildings following future earthquakes. In this code which is not yet official, building seismic design follows performance based design philosophy and for buildings taller than 60m, design shall be verified by nonlinear dynamic analysis for different performance levels. With this regards, seismic performance of this building was evaluated and results are presented in the following sections. The Icon Hotel's hybrid structural system is also presented.

The 42 storey wheel shaped tower (Figure 1) is 160m high with an external diameter of 165m, an internal diameter of 78m and a depth of 35m. The building is structurally formed by two concrete core walls which are placed 96m apart at either legs of the wheel. The top section of the wheel is formed by a steel bridging structure spanning between the two cores. The cores are 13m by 15.5m on plan and are enhanced in terms of overall stiffeness by adjacent columns, Figure 2. The concrete core walls together with the bridging structure, create three 2D wheel shape mega frames which carry the gravity loads and act as portal frame against the lateral loads in longitudinal direction. Figure 3 shows the 3D ETABS[5] model of the tower.

The bridging structure consists of four main elemements:

- A steel moment resisting frame in longitudinal direction
- A steel bracing in transverse direction
- Three steel mega trusses which are located in the mechanical floor levels
- Three steel arches that form the outer diameter of the wheels (Figure 4).

Figure 1 - The Icon Hotel-Architectural rendering

The trusses alone did not have sufficent stiffness and were therefore augemented by the arch to keep the veritical deflections to managable levels.

The building design is dominated by the gravity loading, however, the lateral loads from wind and seismic loads presented engineering challenges. In longitudinal direction, the linked structure combines the benefits of shear walls which deform predominently with bending configuration and moment frames which deform predominently with shear configuration and finally create a portal frame to resist the lateral load. In cross direction, two concrete core walls, which are fairly spread the whole width of building, carry the horizontal loads. Above the core walls, lateral loads are resisted by steel arches plus moment frames in longitudinal direction and steel braces in cross direction. The building's first three predominent mode shapes are shown in Figure 5, the first two modes are translational and the 3rd mode is torsional.

Lateral loading on the structure

Both wind and seismic loading were evaluated in the analysis and design of the structure. Wind tunnel testing was performed by RWDI laboratory[19] to determine more accurately the actual wind pressures applied to the building as well as the translational and torsional accelerations experienced at the top level. The inter-story drifts under 50 years wind return period was kept below 1/500 as per the local authority requirement.

A site specific seismic hazard study was performed by Fugro West Inc[9] and the resulting response spectrum curve was input to the model. It was found that UBC 97-Zone 2A design spectrum, governed the seismic design, which is considered as the minimum requirement by local authority.

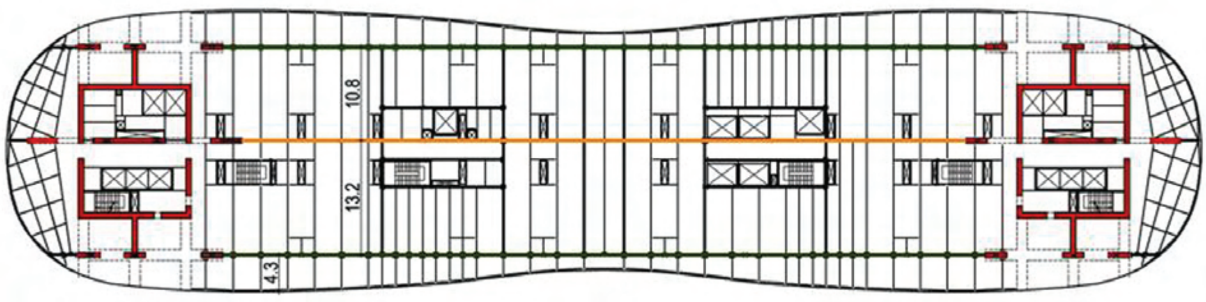

Figure 2 - Structural floor plan at level 34-Red: Main core walls and adjacent columns, Green: Outer wheel shape mega frame, Orange: Central wheel shape mega frame

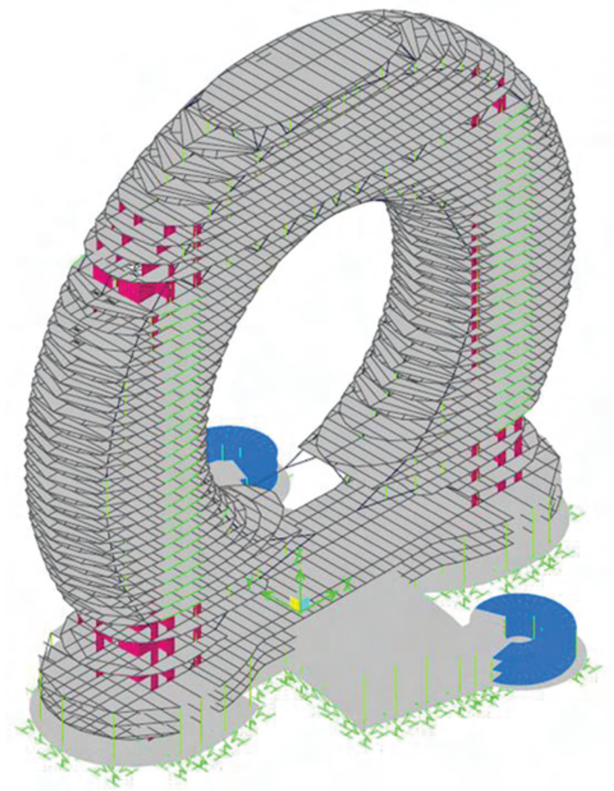

Figure 3 - 3D ETABS Model

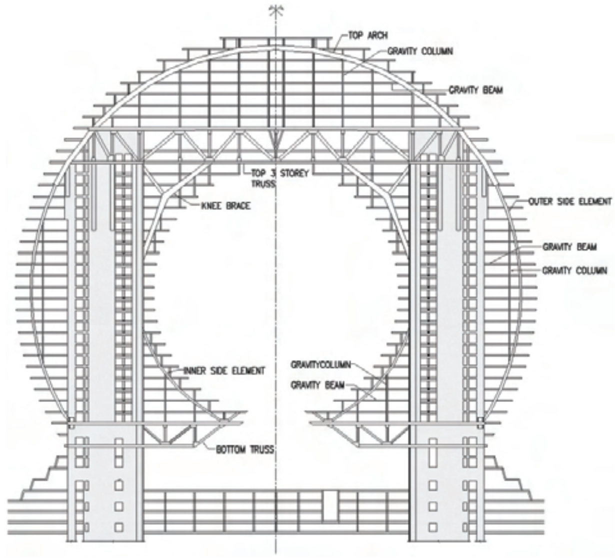

Figure 4 - Central wheel shape mega frame

Comparing the wind and seismic overturning moment over the height of the structure, it was found that wind load controlled the cross direction of the tower while seismic controlled the longitudinal direction.

Performance based seismic evaluation

Performance based seismic engineering is the modern approach to earthquake resistant design. Rather than being based on prescriptive, mostly empirical code formulation, performance based design is an attempt to produce buildings with predictable seismic performance[15]. Therefore, performance objectives such as life safety, collapse prevention, or immediate occupancy are used to define the state of the building following a design earthquake. In one sense, performance based seismic design is limit-state design extended to cover the complex range of the issues faced by earthquake engineers. Performance based seismic design can be used as a tool to evaluate the building behaviour during future anticipated earthquake.

In this regards, building can be designed and detailed to existing codes such as UBC 97 and the building seismic behaviour can be audited by implementing Performance Based Seismic Design approach. This section will outline the steps taken for Performance Based Seismic evaluation of Icon Hotel.

Performance objectives

A seismic performance objective shall be selected for the building, consisting of one or more performance goals. Each goal shall consist of a target building performance level and an earthquake hazard level. DM 2009[7] building code defines 3 levels of earthquake hazard: 1- Frequent earthquake (E1) with a return period of 72 years. 2- Infrequent and higher intensity earthquake ground motions (E2) with a return period of 475 years. 3- The highest intensity, very infrequent earthquake ground motions (E3), with a return period of 2475 years. The multiple performance objectives of tall buildings in Normal Occupancy Class (residence, hotel, office building, etc.) are also identified as Immediate Occupancy (IO) / Minimum Damage (MD) Performance Objective under (E1) level earthquake, Life Safety (LS) / Controlled Damage (CD) Performance Objective under (E2) level earthquake, and Collapse Prevention (CP) /Extensive Damage (ED) Performance Objective under (E3) level earthquake, and Upon the requirement of the Owner or the relevant State Authority, higher performance objectives may be identified for tall buildings in Normal Occupancy Class.

DM 2009[7] has defined 4 design stages for the performance design of Normal Occupancy Class tall buildings as follow:

(1) Design Stage (I – A): Preliminary Design (dimensioning) with Linear Analysis for Controlled Damage/ Life Safety Performance Objective under (E2) Level, this design stage is as same as the design requirement of UBC 97 including the minimum design base shear requirement of the code.

(2) Design Stage (I – B): Design with nonlinear Analysis for Life Safety / Controlled Damage Performance Objective under (E2) Level Earthquake.

(3) Design Stage (II): Design Verification with Linear Analysis for Minimum Damage/ Immediate Occupancy Performance Objective under (E1) Level Earthquake.

(4) Design Stage (III): Design Verification with nonlinear Analysis for Extensive Damage/ Collapse Prevention Performance Objective under (E3) Level Earthquake.

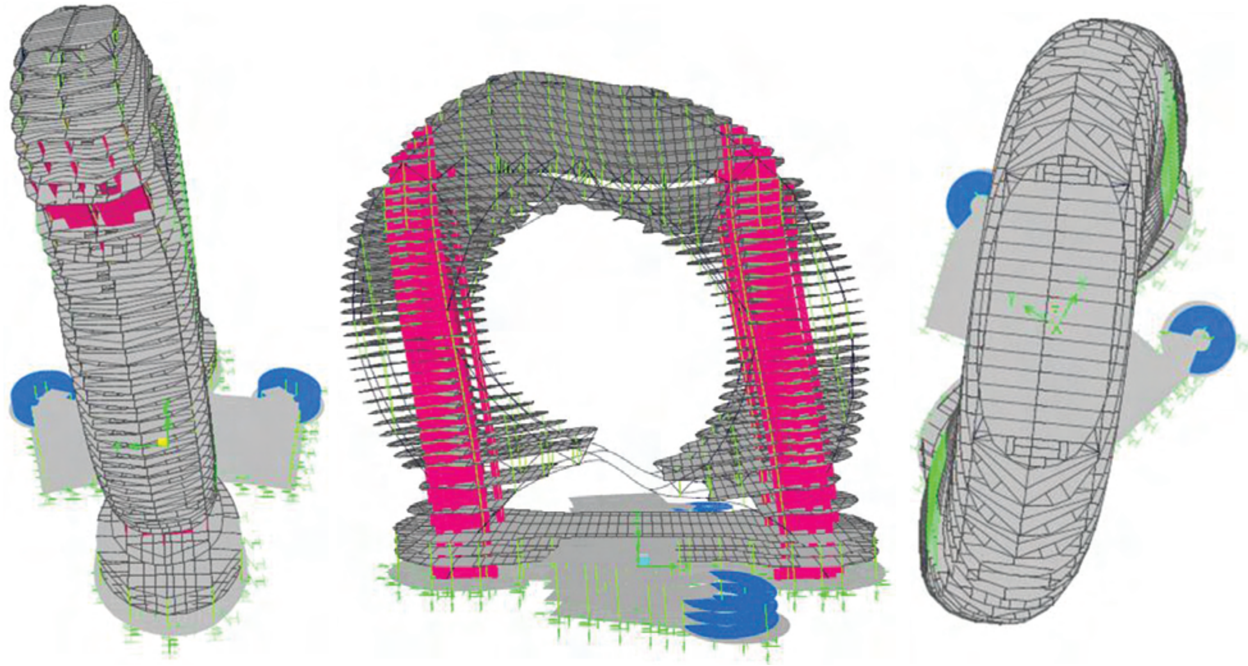

Figure 5 - First 3 modes of Icon Hotel - From left: 1st mode (4.3 sec), 2nd mode (3.3 sec), 3rd mode (3 sec)

Table 1 - Performance based design stages for Icon Hotel

Design Stage	Design Stage I-A	Design Stage I-B	Design Stage III
Earthquake Level	E2	E2	E3
Performance objective	LS/CD	LS/CD	CP/ED
Analysis Type	3-D Linear Response Spectrum Analysis	2-D Nonlinear Time History Analysis	2-D Nonlinear Time History Analysis
Earthquake load/ Time History	Seismic Zone 2A UBC 97-Soil type Sc	Time history obtained from Seismic Hazard Study- 475 years return period	Time history obtained from Seismic Hazard Study- 2475 years return period
Ductility Factor	UBC 97-R=4.5-Bearing Wall System	N/A	N/A
Story Drift Ratio Limit	%2 for inelastic deformation=0.7*R*elastic deformation	2.5% (DM 2009)	3.5% (DM 2009)
Member Strength Design	Member to be designed according to UBC 97	Design to be verified	Design to be verified
Load Factors	Factored load combinations	Service load combinations	Service load combinations
Material strength	Design Strength	Expected strength	Expected strength
Acceptance Criteria	Strength & Story drift ratio	Strain & Story drift ratio	Strain & Story drift ratio

It is worth mentioning that DM 2009 has its own requirement for strength based design, some of the requirements such as ductility factor (behaviour factor) are borrowed from Euro Code (BS EN 1998-1:2004, 2004)[4] and it is totally different from UBC 97. For the purpose of Icon Hotel design, only performance objectives and design stages are followed from DM 2009[7] and strength based design remained based on UBC 97[11], Table 1 summarises the performance design stages and objectives selected for this project.

Seismic hazard study

Site specific studies are often required to characterise the seismic demand for the longer period range of interest for many tall buildings[6], moreover, for conducting nonlinear time history analysis, DM 2009[7] requires a minimum of three or seven sets of earthquake ground motions (acceleration records in two perpendicular horizontal directions) matched with the design spectra. Earlier in this paper, it was shown that codified response spectra based on zone 2A UBC 97[11] is not a perfect representative of different site condition especially for long period range. With this in mind, Fugro West, Inc.[9], was selected to provide geotechnical earthquake engineering services for the Icon Hotel project. Following steps have been taken by FWI for this study:

(1) Refining the seismotectonic model that FWI has developed for the region in order to characterise the various sources of seismicity that impact the UAE region in the Gulf.

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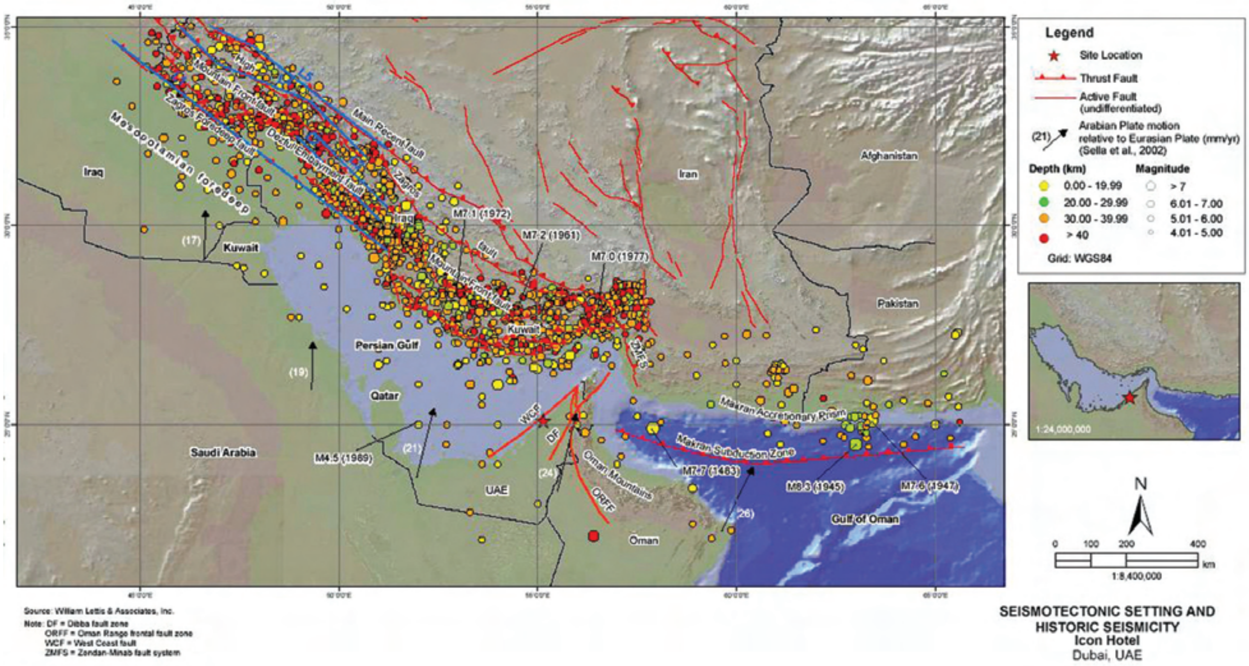

Figure 6 - The seismotectonic setting and historic seismicity around the project site (FWI, 2008)

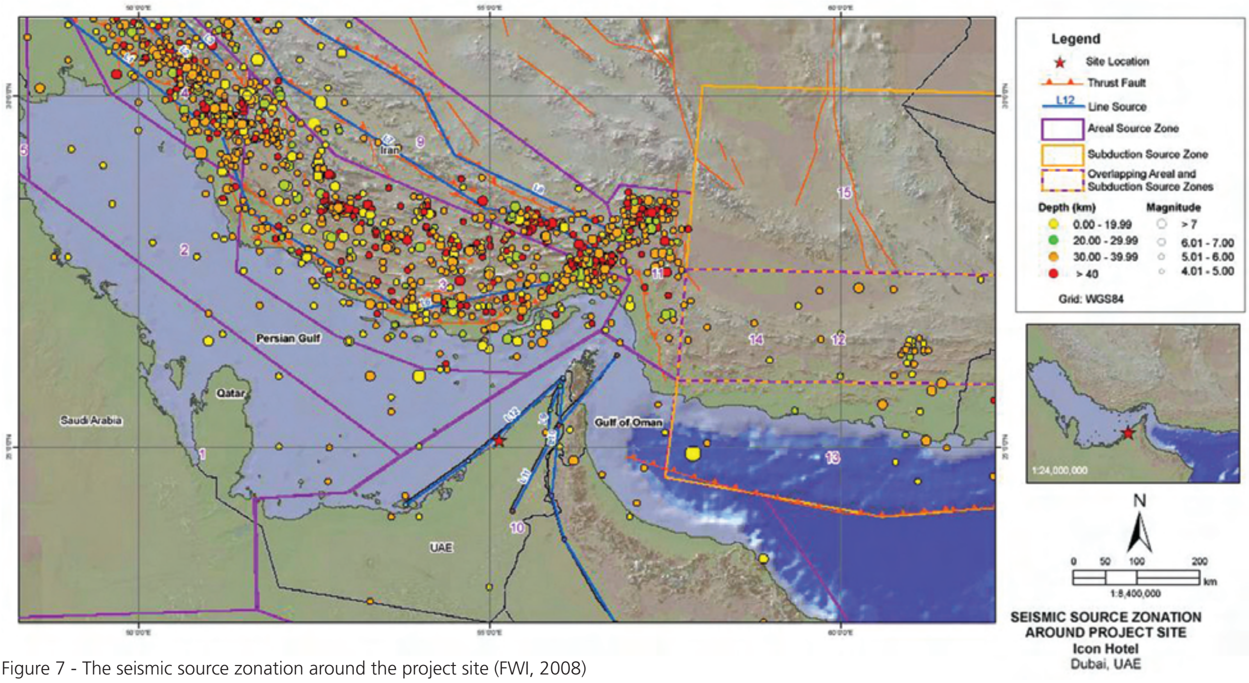

Figure 7 - The seismic source zonation around the project site (FWI, 2008)

This process involved delineating the geometry and seismicity characteristics of potential seismogenic sources within about 300 to 1000 kilometres of the project area. Based on this review, FWI modelled twelve areal shallow crustal sources of seismogenic shaking, one interplate subduction source, two intraplate subduction sources, and in addition, twelve planar shallow crustal sources were considered as independent faults.

Figure 6 shows the seismotectonic setting and historic seismicity around the project site, and Figure 7 shows the seismic source zonation around the project site.

(2) Conducting probabilistic seismic hazard analysis (PSHA) to compute acceleration response spectra compatible with design spectra presented in IBC 2006 (ICC, 2006)[12]. In order to do PSHA analysis, the relative distribution of magnitudes for each seismic source was modelled using one of three magnitude probability density functions consist of: a) Truncated Exponential b) Youngs and Coppersmith (1985)[23] c) Pure Characteristic. Decision tree was used to address the epistemic uncertainty associated with a) The empirical attenuation relationship b) The maximum magnitude on the areal and the planar fault sources c) The slip rate on the planar fault sources.

Table 2 - Estimated peak ground acceleration at rock boundary (design level)

Design Level	Horizontal PGA (g)	Vertical PGA (g)
IBC 2006	0.15	0.1

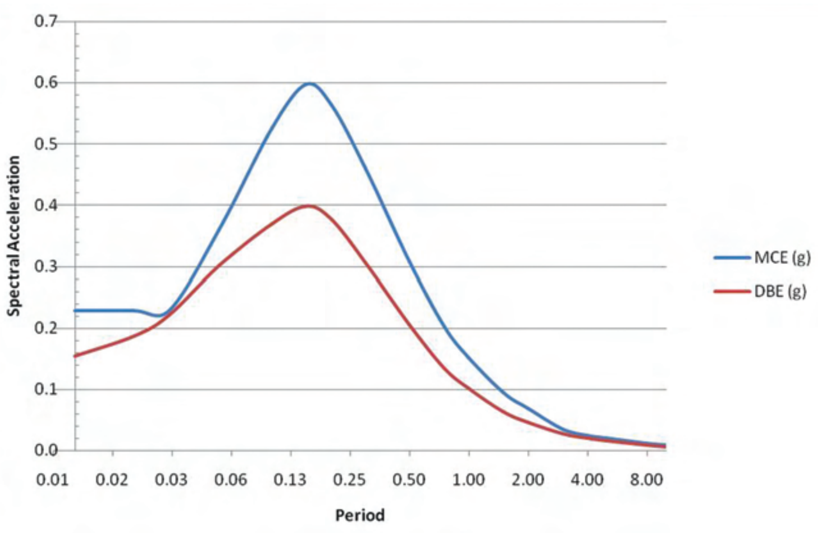

Figure 8 - The estimated equal hazard horizontal response spectrum for the MCE and DBE at rock boundary (FWI, 2008)

The estimated peak ground acceleration is presented in Table 2 at the “rock” boundary, with a shear wave velocity of approximately 880 meters per second or more applicable at a depth of around 16 meters.

The estimated equal hazard horizontal response spectrum for the MCE (E3 in this paper) and DBE (E2 in this paper) at the rock boundary is shown in Figure 8.

(3) De-aggregating the seismic hazard results to identify the key contributors to the hazard in each zone in terms of earthquake magnitude, distances to the seismogenic sources, and types of seismogenic sources. The de-aggregation of the hazard revealed three main sources of contribution to the hazard the site. The West Coast fault (approximately 6.5km from the project location) and the Oman Peninsula areal source zone are the largest contributors to the hazard at the shorter structural periods. At longer structural periods, the contribution is primarily from the West Coast and Zendan-Minab faults. The majority of the hazard for the MCE (2,475-year return period) comes from small to intermediate earthquakes (i.e., 4.5 – 6.0 magnitude earthquakes) for short structural periods with distances from 10 to 30km. This coincides with the maximum contribution from the West Coast fault and Oman Peninsula zone for the shorter structural periods. At longer structural periods, a second hazard mode is observed in the de-aggregation with large magnitude earthquakes (i.e., 7.0 to 7.5) at distances greater than 50km. This likely corresponds to the increased contribution of the Zendan-Minab fault, which is roughly 70km from the project site.

(4) Selection of acceleration time histories to match with Spectrum at rock level. Three sets of accelerograms were selected and matched to the MCE and DBE spectra at bedrock level using a time-domain spectral matching procedure. The ground motions were selected from high-quality recordings with emphasis given to the overall shape of the response spectra of the recorded motion relative to the target spectrum, as well as the magnitude, distance, and PGA of the recorded time histories, Table 3 shows the selected time histories, Figure 9 shows the spectrally matched time histories at DBE level and Figure 10 shows the spectrally matched time histories at MCE level.

(5) Site Response Analyses to assist with the development of design response spectra and time histories at the depth of maximum soil-pile interaction. The non-linear site response analyses were performed for both DBE and MCE time histories to capture the non-linear response of the soft soil. The subsurface conditions at the project site consist of 12 to 16 meters of loose to medium dense sand fill overlying bedrock (reclaimed land). In some areas, the sand fill is underlain by about two to three meters of dense sand. The fill deposits will be densified through ground improvement. To analyse the site response, assumption was made that the post improvement subsurface profile is consist of 12 to 19 meters of medium dense to dense sand overlying bedrock with an average shear wave velocity on the order of 200 m/s is expected in the improved sand fill.

Table 3. Selected time histories

Set	Earthquake	Magnitude	Distance (Km)	Recording station	Designation
1	1994 Northridge, USA	6.7	18.2	90053 Canoga Parl-Topanga Can	CNP 106 CNP 196
2	1976 Gazli, USSR	6.8	22.3	9201 Karakyr	GAZ000 GAZ090
3	1992 Landers, USA	7.3	42.5	12025 Palm Springs Airport	PSA 000 PSA090

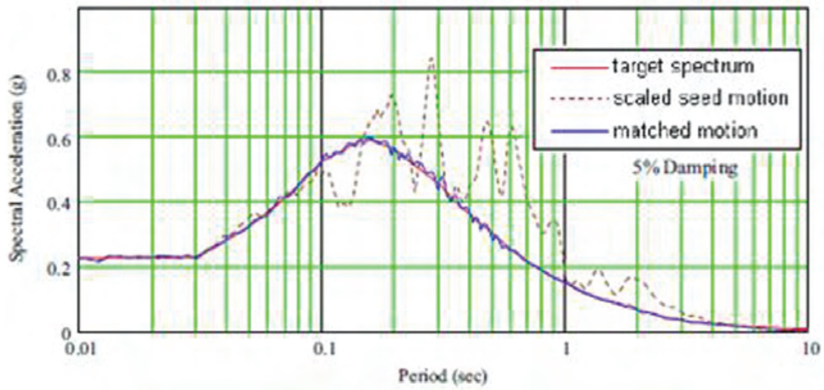

Figure 9 - The spectrally matched time histories at DBE level at rock boundary. CNP 196 motion, 1994-Northridge earthquake (FWI, 2008).

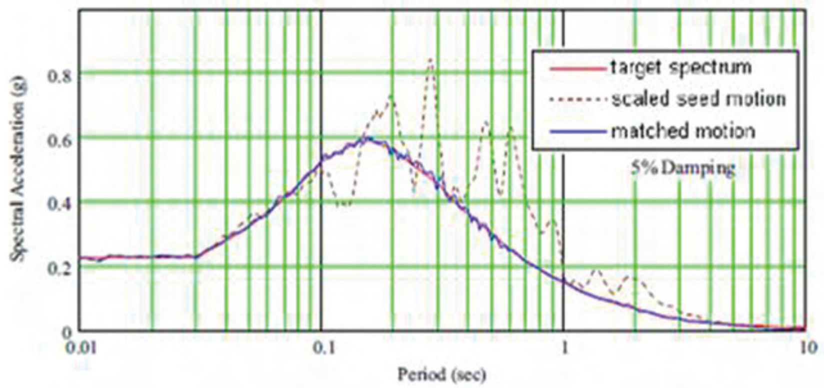

Figure 10 - The spectrally matched time histories at MCE level at rock boundary.GAZ 090 motion-1976 GAZLI earthquake Russia (FWI, 2008).

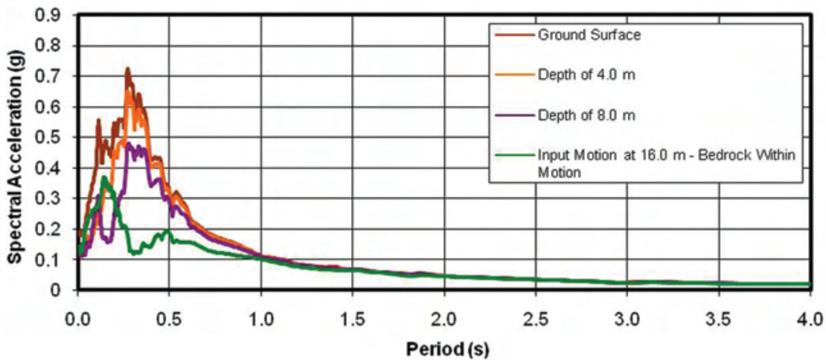

Figure 11 - The response spectra at rock, 8m from surface, 4m from surface, and ground surface for E2 (DBE) CNP 106 earthquake record (FWI, 2008).

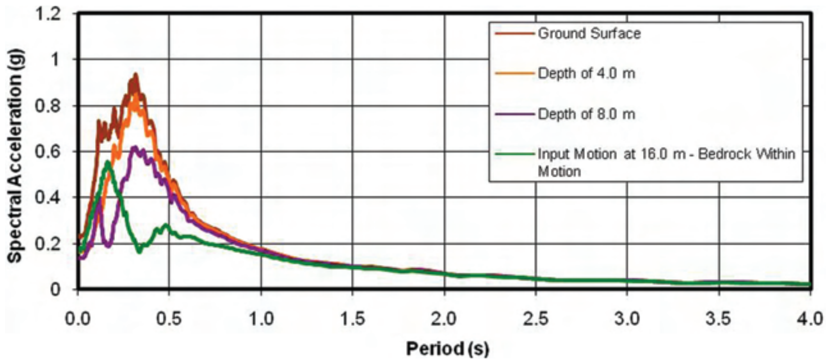

Figure 12 - The response spectra at rock, 8m from surface, 4m from surface, and ground surface for E3 (MCE) CNP 106 earthquake record (FWI, 2008).

Since the tower is supported on pile foundations, the response of the structure is dependent on interaction between the pile foundations and the surrounding soils.

Detailed soil-structure interaction analyses were not included in this work. Transfer of the loads between the soil and the piles occurs at some depth below the ground surface and depends on several factors such as the stiffness of the piles and the soil, fixity of piles at the head, etc. In the absence of detailed soil-structure interaction analyses, it is recommended that the performance of the proposed structures be based on ground motions at the level of maximum soil-pile interaction rather than the motions at the ground surface. Based on this, maximum soil-pile interaction depth approximately assigned 8meters for the stiffer piles (1.0m to 2m diameter piles), and a depth of 4 meters for the smaller piles (600mm to 900mm diameters).

Figures 11 and 12 show the response spectra at rock, 8m from surface, 4m from surface, and ground surface for E2 (DBE) CNP 106 (Northridge, 1994) earthquake scaled record and E3 (MCE) CNP 106 earthquake scaled record respectively. As shown on the figures, site response analyses show significant amplification of periods between 0.3 to 1.0 second. That amplification is largely associated with the presence of medium dense to dense sand above the bedrock.

It is worth mentioning that for the sake of nonlinear analysis for this work, time histories at 4m from the surface have been used and for the design stage I-A, the design spectrum from UBC 97[11] is used due to the minimum requirement by local authority, Figure 13 shows the comparison of UBC and Icon Hotel estimated design spectra.

Nonlinear time history analysis

A two dimensional transient nonlinear dynamic analysis with material and geometric nonlinearity was performed to determine the E2 and E3 level earthquake demand on the building's structural system. The nonlinear time history analysis was carried out in order to evaluate the maximum drift and nonlinear behaviour of building and verify that whether they were within acceptable limit.

The acceptance limits for nonlinear behaviour are defined in this section. CSI PERFORM 3D[17], a finite element software product of Computer and Structures Inc., was used to run nonlinear analysis. This software uses the implicit Newmark =1/4 for step by step integration through time.

The linear results from PERFORM 3D[17] were verified against the results from ETABS[5]. The modelling assumptions are addressed in this section and the results are presented only for level E3 earthquake.

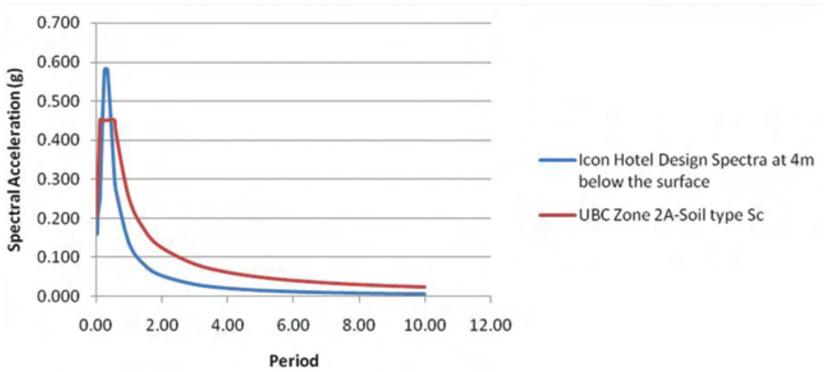

Figure 13 - Comparison between UBC (zone 2A, Sc soil type) and estimated Icon Hotel design spectra at 4m below the surface.

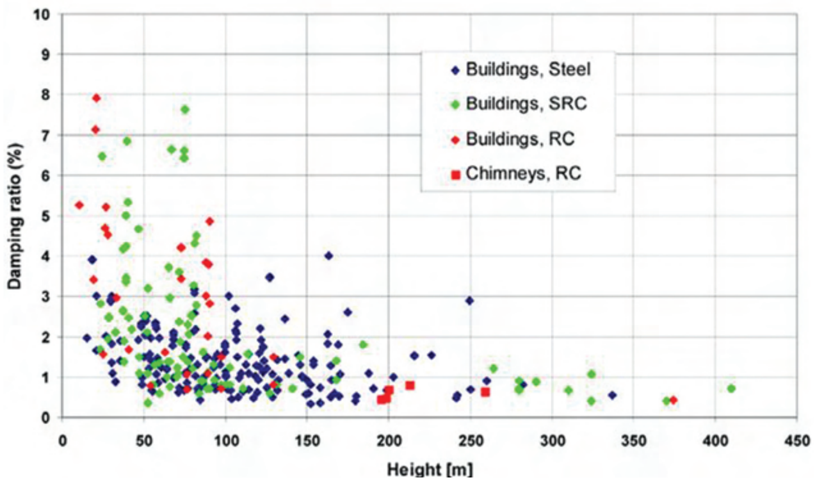

Figure 14 - Measured damping ratio vs. Building height for first translational modes (CTBUH 2008)

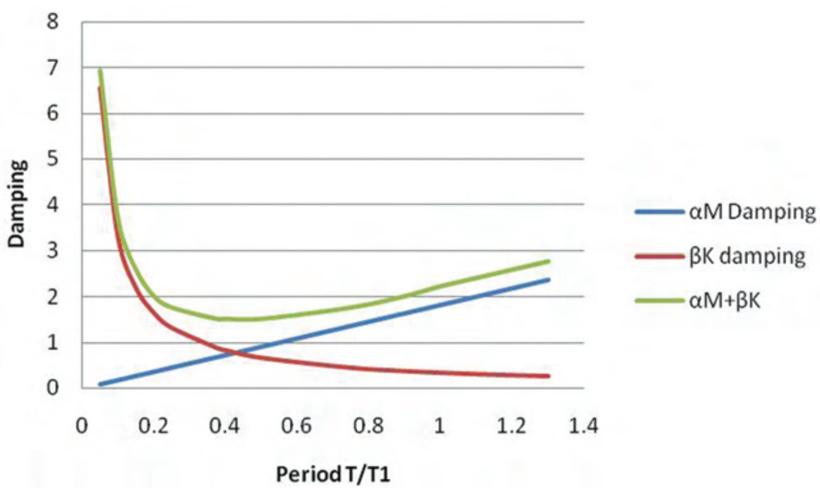

Figure 15 - The selected Rayleigh damping-First mode=3.573 sec

Damping

Damping in buildings varies depending on the selection of materials, structural system geometry, foundation and types of architectural finishes. The level of damping also varies as a function of the building response. Importantly, hysteretic energy dissipation (damping) associated with yielding and damage in structural components is automatically accounted for in nonlinear response-history analysis[6]. These factors should be considered when selecting a damping ratio for modal or nonlinear response-history analysis. Whereas 5% of critical damping has been traditionally assumed for conventional buildings designed by code procedures, there is indisputable evidence that this is higher than the actual damping of modern tall buildings. With this regards, CTBUH (2008)[6] has presented a graph (Figure 14) based on the Japanese database[20] which shows the damping measured for different type of buildings with respect to height. Based on this graph (Figure 14), although DM (2009) allow considering maximum %5 damping, %2 damping was selected to address the intrinsic (viscous) damping in Icon Hotel building.

PERFORM 3D[17] allows two types of viscous damping, namely Modal and Rayleigh damping, in which, Rayleigh damping, was selected for this work and based on the recommendation in PERFORM 3D[17] user guide manual, and were chosen so that the damping is 2% at TB = 0.9T1 , where T1 is the first mode period, and so that the damping is also 2% at TA = 0.2T1, then the damping is close to 2% over a range of periods from 0.2T1 to T1 . This will cover the most important modes. Higher modes are more heavily damped. Figure 15 represents the selected Rayleigh damping.

Structural modelling

Three design stages are defined for the performance based design of Icon Hotel. Therefore, a 3D model was built in ETABS[5] for the design stage I-A, and building was designed as per UBC 97 requirement and linear spectral analysis for zone 2A, Sc soil type. A 2D model was set up in PERFORM 3D[17] for the longitudinal direction of the building (Figure 16) for design stage I-B and III. The results from both software (both 2D model) were compared by comparing the mode shapes and the different node deflection under gravity load. Table 4 shows the comparison of first 3 modes of the building in both the softwares. In general, results from PERFORM 3D[17] shows very good agreement with ETABS[5] with some difference due to the selection of further reduced E value for design stage III to address crack properties at CP stage. Figure 17 shows the second mode of the 2D model in ETABS[5] and PERFORM 3D[17].

Table 4 - The comparison of first 3 modes of the building (2D Model)

	ETABS	PERFORM 3D	Difference %
First Mode Period (Sec)	3.448	3.573	4
Second Mode Period (Sec)	0.7289	0.8244	13
Third Mode Period (Sec)	0.6663	0.7236	9

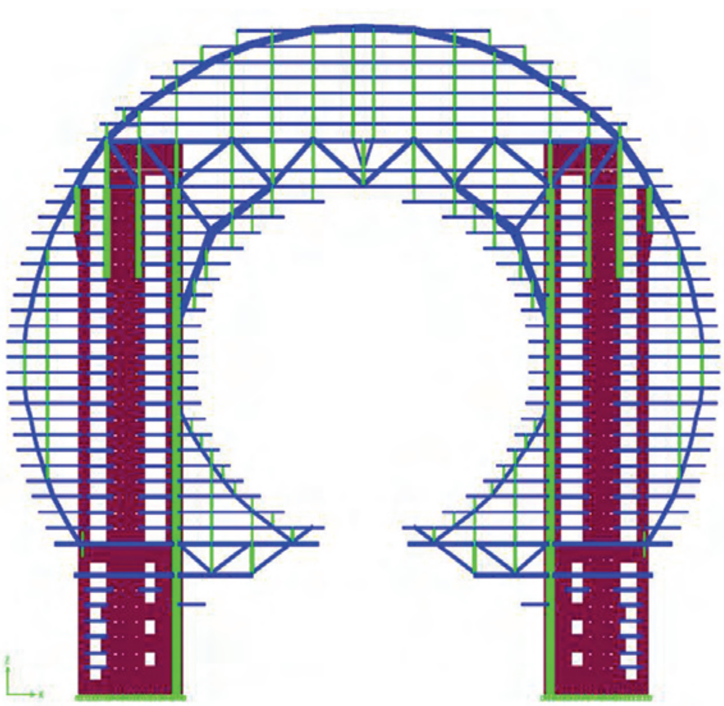

Figure 16. 2D ETABS model for longitudinal direction of building

Material properties for concrete

To accurately capture the nonlinear behaviour of the elements, realistic material model was used for the concrete strength. The concrete stress-strain relationship is related to the reinforcement and the confinement of the section. C70 (70 MPa cubic strength) was used for the concrete core walls all the way up to the top of the shear walls (in design stage I-A) and it was considered unconfined since no special design was done for the shear wall boundary confinement. The stress-strain relationship of concrete is shown in Figure 18, and also summarized in Table 5.

It is usual to use the expected material strength, which can be substantially larger than the nominal strength, this requires some sensitivity analysis, but this effect was ignored in this work.

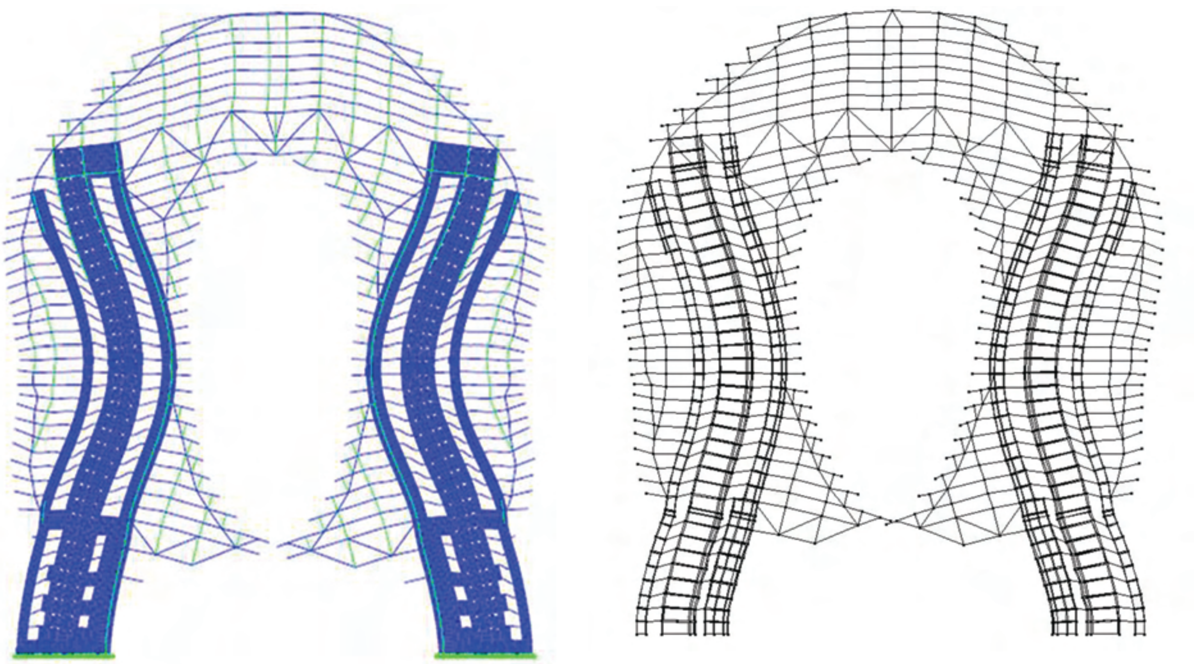

Figure 17 - 2D ETABS model –Second mode shape-left ETABS-Right PERFORM 3D

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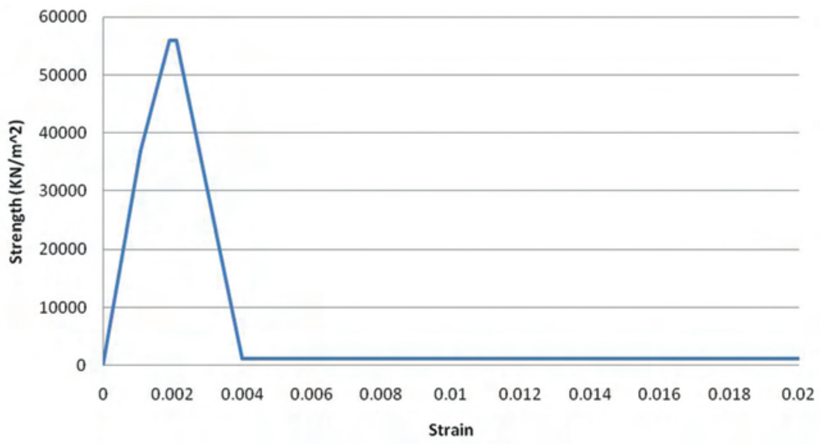

Figure 18 - Stress-Strain relationship for unconfined C70 concrete

Table 5 - C70 concrete specifications

Compression Strength at 28 days (KN/m^2)	Tension Strength	Modulus of Elasticity (KN/m^2)	Crushing Strain	Spalling Strain	Failure Strain
56000	0	3.4 E7	0.0021	0.004	0.02

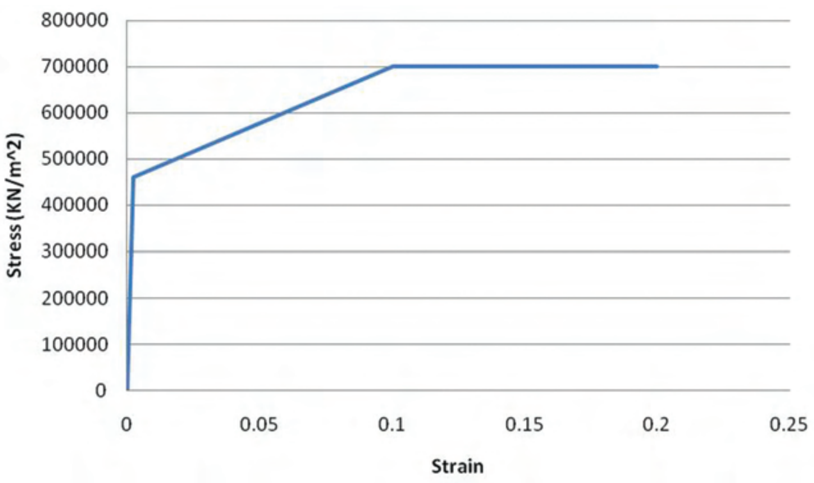

Figure 19 - The compression stress-strain relationship for grade 60 steel reinforcement

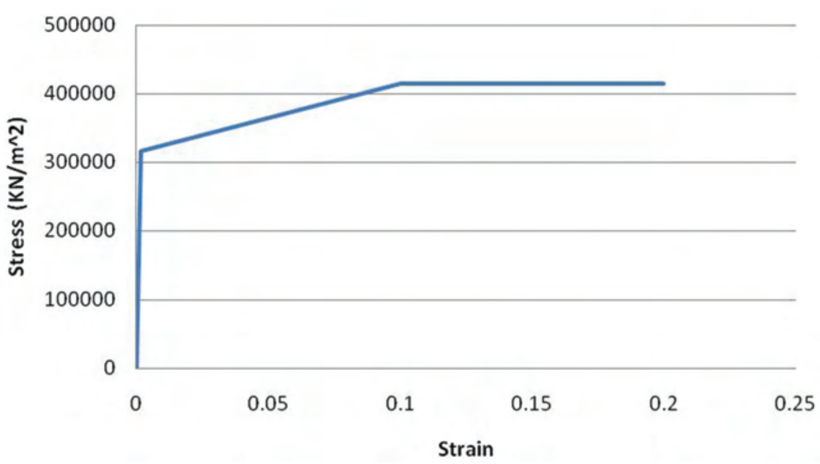

Figure 20 - The compression stress-strain relationship for S355 grade steel used for structural steel members

Material properties for steel

A strength hardening steel model was used as a basis for the structural steel and reinforcing bar steel. The steel material model assumes symmetrical behaviour for both compression and tension. Figure 19 shows the compression stress-strain relationship for grade 60 steel reinforcement and Figure 20 shows the compression stress-strain relationship for S355 (355 MPa yield strength) grade steel used for structural steel members with FY reduction for the thickness more than 8mm. As same as concrete, the expected strength was not used for this work.

Elements description

PERFORM 3D[17] has a comprehensive library of elements for modelling the nonlinear behaviour. Among the available elements, the following were selected for the Icon Hotel:

Shear walls bending behaviour: It is not a simple task to model inelastic behaviour of shear walls and most structural design packages, unlike PERFORM[17], still cannot provide nonlinear shell elements. Shear walls were modelled using fibre section, in this regard, outer concrete columns and central walls were modelled with auto size fibre section divided into 8 fibres with 1% reinforcement in each fibre (Figure 21). Inner concrete columns, which are in fact composite sections with heavy steel member embedded inside concrete sections, modelled with fixed size fibre section as shown in Figure 22.

Axial strain gages were added to the corner of concrete walls and columns to measure concrete compression strain and steel tension strain. To calculate the bending demand on the shear walls, wall rotation gages were added separately for outer columns, central walls, and inner columns. Figure 23 shows the rotation gage for the central walls at first level.

Shear walls shear strength: In tall shear wall structures it is common to allow inelastic behaviour in bending, but it requires that the wall remain essentially elastic in shear. To check the shear force, structural section which is basically a cut through a wall cross section over several elements, was provided for outer columns, central walls, and inner columns separately, the

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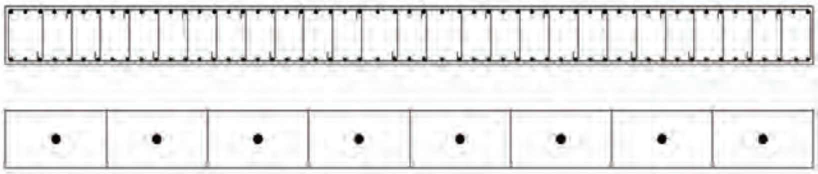

Figure 21 - Auto size fibre section for outer column and central wall

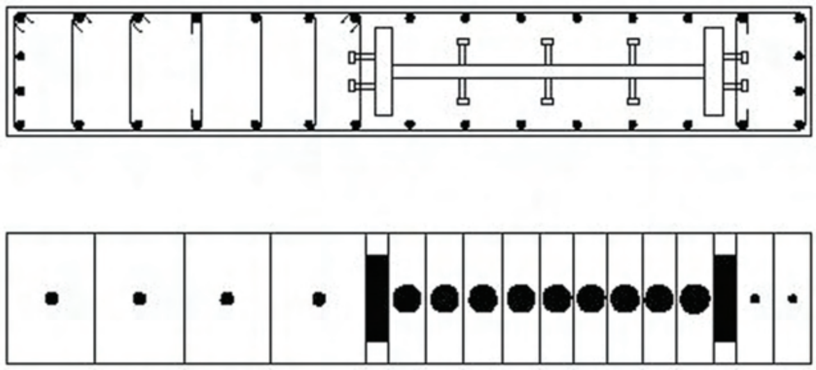

Figure 22 - Fixed size fibre section for inner composite columns

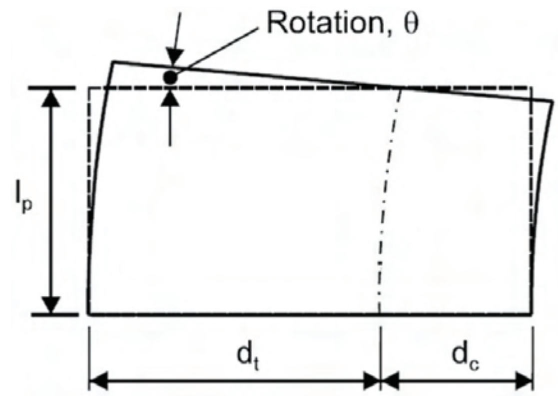

Figure 23 - Rotation gage for central wall.

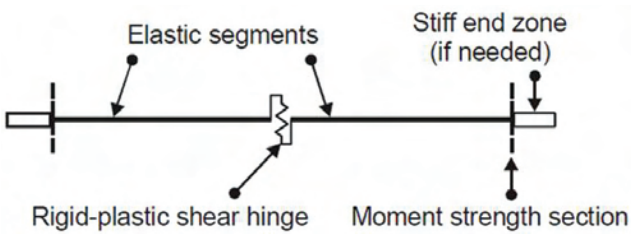

Figure 24 - Model for deep coupling beams.

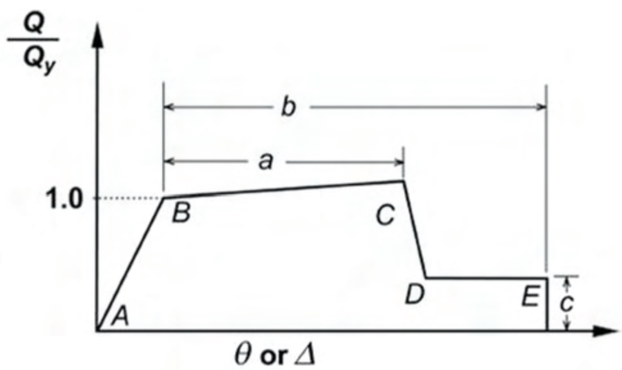

Figure 25 - Generalised force-deformation model is shown for concrete and steel beams

shear strength checked against

$$0.8\left(0.83\sqrt{f_c'}bd\right)$$

for concrete. For composite walls and columns, additional strength was considered due to embedded steel members.

Deep coupling beams: for the coupling beams with span to depth ratio less than two, it was assumed that shear is the controlling behaviour and it was modelled with two elastic segments with the rigid-plastic displacement type shear hinge in the middle as shown in Figure 24. To account for the depth of coupling beams for the connection of beam to the wall, vertical imbedded member added to the corner of the walls at connection between wall and deep coupling beams. The imbedded members are very stiff in bending, but have negligible axial stiffness.

Slender coupling beams: for the coupling beams with span to depth greater than four, chord rotation model[8] was used for steel and concrete coupling beams. The generalised force-deformation model is shown for concrete and steel beams in Figure 25. To connect the slender beam to the wall, horizontal imbedded member was added. The imbedded members are very stiff in bending, but have negligible axial stiffness.

Slender concrete coupling beams were checked for the maximum shear exerted by earthquake to make sure that it remains elastic.

Top Arch, knee brace, side elements, top and bottom chord of the main and bottom truss, vertical member of the main and bottom truss: These members were treated as column member and chord rotation model[8] was used to model the nonlinear behaviour.

Diagonal members in main and bottom truss: these members were modelled as simple nonlinear bar which can only resist the axial force. Simple bar can buckle in compression and also yields in tension.

Other columns and beams: these members modelled as elastic column and beam respectively just to carry gravity load and transfer it to main elements.

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Table 6 - Collapse prevention E3 level earthquake acceptance criteria

Element	Action type	Classification of Action	Expected Behaviour	Stiffness Modifiers	Acceptance Limit for Nonlinear Behaviour at E3 level Earthquake (Criteria From FEMA 356 Unless Noted)
Shear Walls	Axial-Flexure Interaction (Shear Wall Rotation-From Rotation Gage)	Ductile	Nonlinear	Out of Plane Flexural- 0.25 EI In-Plane-Automatically accounted by Fibre Section	0.006 radians
	Concrete Compression Strain	Ductile But No Crushing allowed	Nonlinear	_	0.0021
	Steel Tension Strain (From Strain Gage)	Ductile	Nonlinear	_	Inside The Hinge Region 0.06 (DM 2009) Outside The Hinge Region 0.0033 (1.5 yield strain)
	Shear	Brittle	Linear	Shear-0.25GA	Code Maximum Allowed Shear Strength
Deep Coupling Beams-Reinforced Concrete	Shear	Ductile	Nonlinear	Shear-0.25GA	Shear Strain < 0.025 radians (assumed)
Slender Coupling Beams-Reinforced Concrete	Flexure	Ductile	Nonlinear	0.5EI	0.025 radians
	Shear	Brittle	Linear	Shear-0.25GA	Code Maximum Allowed Shear Strength
Slender Coupling Beams-Steel	Flexure	Ductile	Nonlinear	EI	9 y
Top Arch	Axial-Flexure Interaction (Hinge Rotation)	Ductile	Nonlinear But Preferred To Remain Elastic	EI	11 (1-1.7 P/PCL) y.
Main and Bottom Truss-Top Chord, Bottom Chord, and Vertical Members	Axial-Flexure Interaction (Hinge Rotation)	Ductile	Nonlinear But Preferred To Remain Elastic	EI	11 (1-1.7 P/PCL) y.
Side Elements	Axial-Flexure Interaction (Hinge Rotation)	Ductile	Nonlinear But Preferred To Remain Elastic	EI	11 (1-1.7 P/PCL) y.
Knee Brace	Axial-Flexure Interaction (Hinge Rotation)	Ductile	Nonlinear But Preferred To Remain Elastic	EI	11 (1-1.7 P/PCL) y.
Diagonal Member of Main and Bottom Truss	Axial Tension and Compression	Ductile	Nonlinear But Preferred To Remain Elastic	EI	Compression 7Δc (Δc is the axial deformation at expected buckling load) Tension 9ΔT (ΔT is the axial deformation at expected tensile yielding load)
Gravity Beams and Columns	Remain Linear				
Drift	%3.5 (DM 2009)				

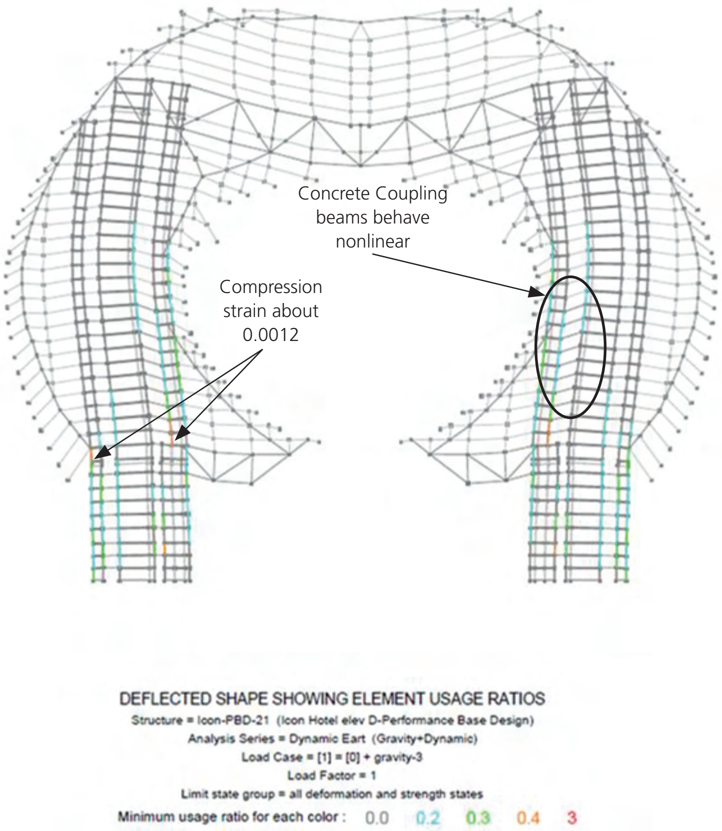

Figure 26 - State of concrete compression strain and the area where coupling beams show nonlinear behaviour under gravity load.

Analysis methodology

Each analysis consists of two separate nonlinear runs that are appended to give one set of results. The first nonlinear constitutes a load pattern representing the self weight, super imposed dead load and 25% of live load on the structure. The second nonlinear analysis is the integrated time history analyses which apply the time history on the structure. A total six time histories applied to the longitudinal direction of the building (three sets of two perpendicular directions of earthquake time histories applied along one direction to account for maximum earthquake component) and maximum response from these six time history analysis considered as the final result as per DM 2009[7].

Acceptance criteria for nonlinear E3 level analysis

Acceptance criteria for E3 level to meet the performance objective of CP/ED (Table 1) were taken from FEMA 356 (FEMA 2000)[8] and DM (2009)[7]. The acceptance criteria are presented in Table 6.

The desired behaviour is as follows:

(1) The RC and composite walls can hinge in bending at the base. The steel reinforcement can yield, but there should be little or no concrete crushing.

(2) The wall can crack in bending in the higher stories, but otherwise should remain essentially elastic (i.e., there should be little yielding of the reinforcement).

(3) The wall should remain essentially elastic in shear, including in the hinge region at the base.

(4) The deep coupling beams can yield. These beams are assumed to be controlled by shear.

(5) The slender coupling beams can yield. These beams are assumed to be controlled by flexure.

(6) Other steel members (other than gravity columns and beams) preferably remain elastic.

(7) All the gravity columns and beams remain elastic.

Nonlinear time history analysis evaluation

At first step, gravity load was applied to the structure. When gravity loads are applied to structures with fibre sections, it is possible for concrete cracking to occur (steel yield or concrete crushing should not occur). Concrete cracking is a nonlinear event, so it is often necessary to specify that the gravity load analysis is nonlinear. In this structure the behaviour is nonlinear, so nonlinear analysis was used. The results show that along with concrete cracking, some of the slender concrete coupling beams behave inelastic under gravity load so these coupling beam need to be strengthened or changed to steel coupling beam. Shear wall are highly loaded under gravity loads which shows that the system is mostly behave like bearing wall system rather than pure shear wall. Figure 26 shows the state of concrete compression strain and the area where coupling beams show nonlinear behaviour.

At second step, time histories applied to the building and acceptance criteria (Table 6) were evaluated at each time step. Results generally show that most of the nonlinear and earthquake energy dissipation happened through the nonlinear behaviour of slender and deep concrete coupling beams and yielding of steel and crushing of concrete did not happen in the shear walls and all the steel members remained elastic, this was the desired behaviour and our building essentially met the assumed performance objectives.

DM (2009)[7] requires maximum 3.5% inter-story drift under level E3 earthquake. Maximum inter-story drift under all time histories were evaluated and it was seen that the maximum inter-story drift is around 0.006 which is well below the limit. Figure 27 shows the envelope of inter-story drifts.

Shear walls need to remain essentially elastic in shear under earthquake load. The envelope of analysis results show that the maximum demand over capacity ratio (D/C) is 0.45 in the central wall which meet the acceptance criteria. Figure 28 shows the envelope of shear force and the capacity of central shear wall.

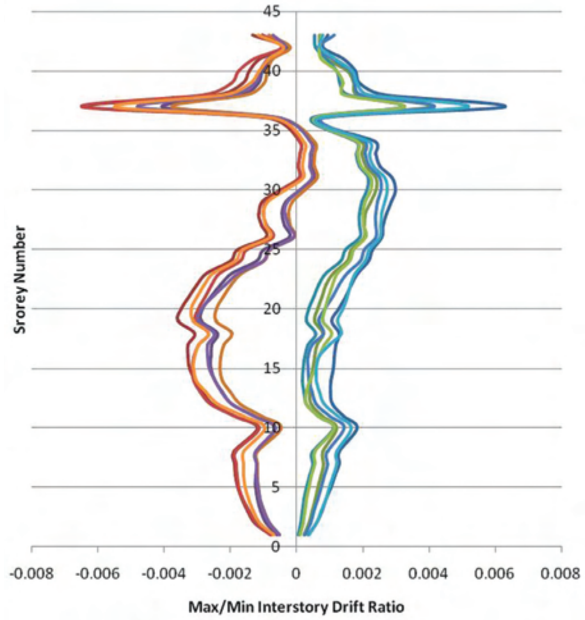

Figure 27 - The envelope inter-story drift ratios for MCE earthquakes (Acceptance limit 3.5%)

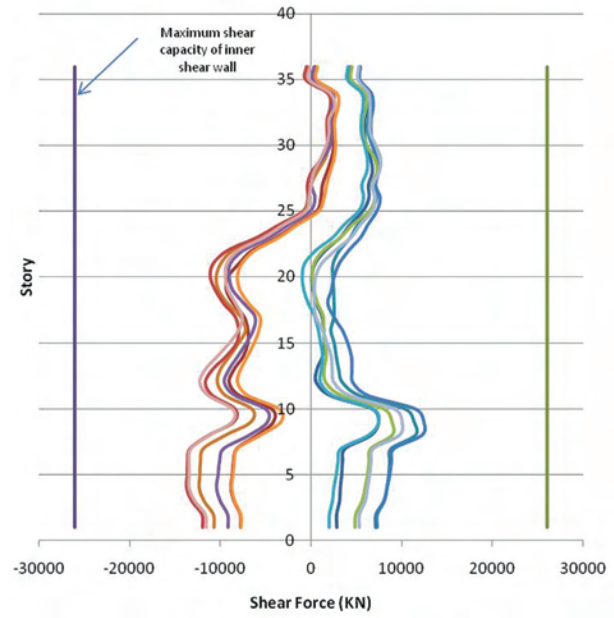

Figure 28 - The envelope of shear force and the capacity of central shear wall

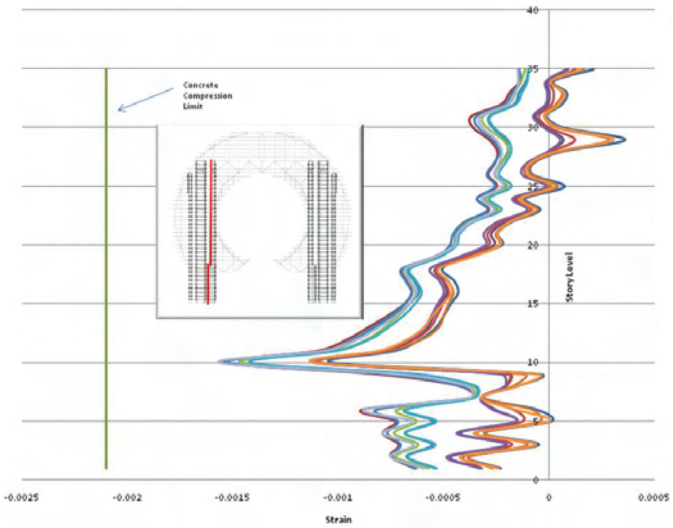

Figure 29 - Envelope of the concrete compression strain at critical location

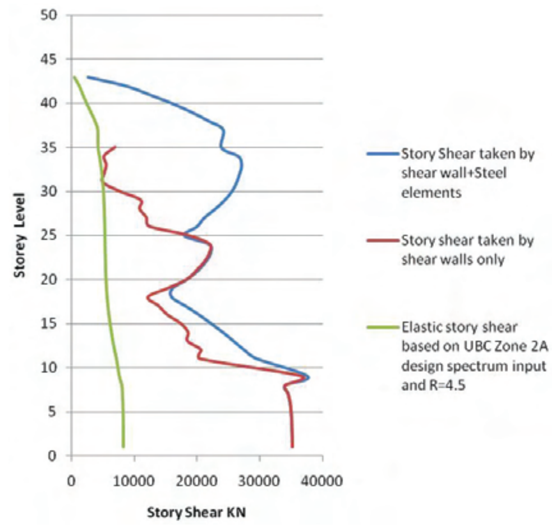

Figure 30 - Story shear comparison-Lander 000 MCE event

Concrete compression strain remained under the assumed concrete crushing strain as shown in Figure 29.

Story shears are compared in Figure 30, in this figure reduced elastic story shear due to the UBC design spectrum input with R (ductility factor) equal to 4.5 is compared to maximum story shear carried by shear walls only and shear walls plus steel elements under Lander 000 MCE event.

Review of the dissipated energy by different mechanism provides valuable information for assessing the performance of the structure. Figure 31 shows the energy dissipated during Lander 000 event.

It can be seen that most of the energy is dissipated by elastic strain energy and also viscous damping; small amount of energy is dissipated by in-elastic energy through inelasticity of concrete coupling beams.

To understand the exact locations of hinge formations and nonlinear behaviour of the structure, Lander000 time history which creates the maximum drift was magnified four times and applied to the structure. Analysis stopped at 13.44 seconds due to concrete crushing at storey level 1 to 12 and excessive shear wall rotation at lower levels (Figure 32).

Review of the dissipated energy showed that in-elastic energy is still small as compare to elastic and viscous energy (Figure 33) which indicate that building does not have ductile behaviour due to some weak points and heavy gravity loads applied to the shear walls (Bearing Wall System) but it has good over-strength factor due to primary design of elements for full gravity load and limited inter-story wind drift. Although the applied load is 4 times more than the anticipated real load but concentration of crushing in specific areas, lead us to identify the weak points.

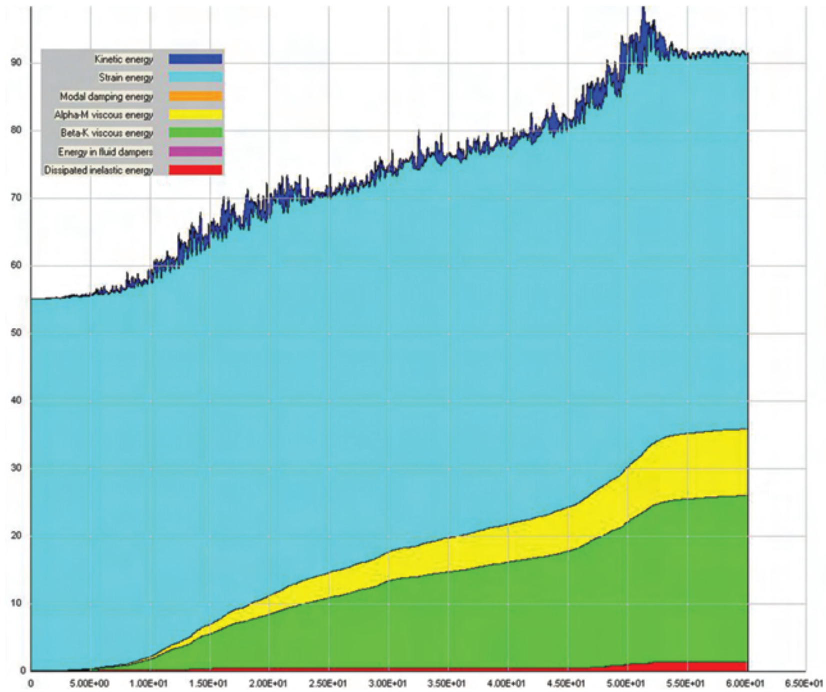

Figure 31 - Dissipated energy by different mechanisms-Lander 000 MCE event

The weak points can be avoided or strengthen. Based on this, it was decided to add confined boundaries for the first 15 levels of shear wall by adding confined boundary elements wherever possible. At the location where side element meet the concrete shear wall, steel member imbedded inside the concrete to provide smooth load transfer to the shear wall.

The analysis of the animations provided us the exact behaviour of the building through the entire time duration and specifically it showed that above the shear walls where the lateral load resisting system is just provided by steel arch, is moving very much out of phase as compare to the shear walls movement, in other word, the arch is much more flexible than the shear wall. It was decided to add steel bracing above the main truss all the way up to the steel arch to avoid this out of phase movement. Figure 34 shows the proposed new braces.

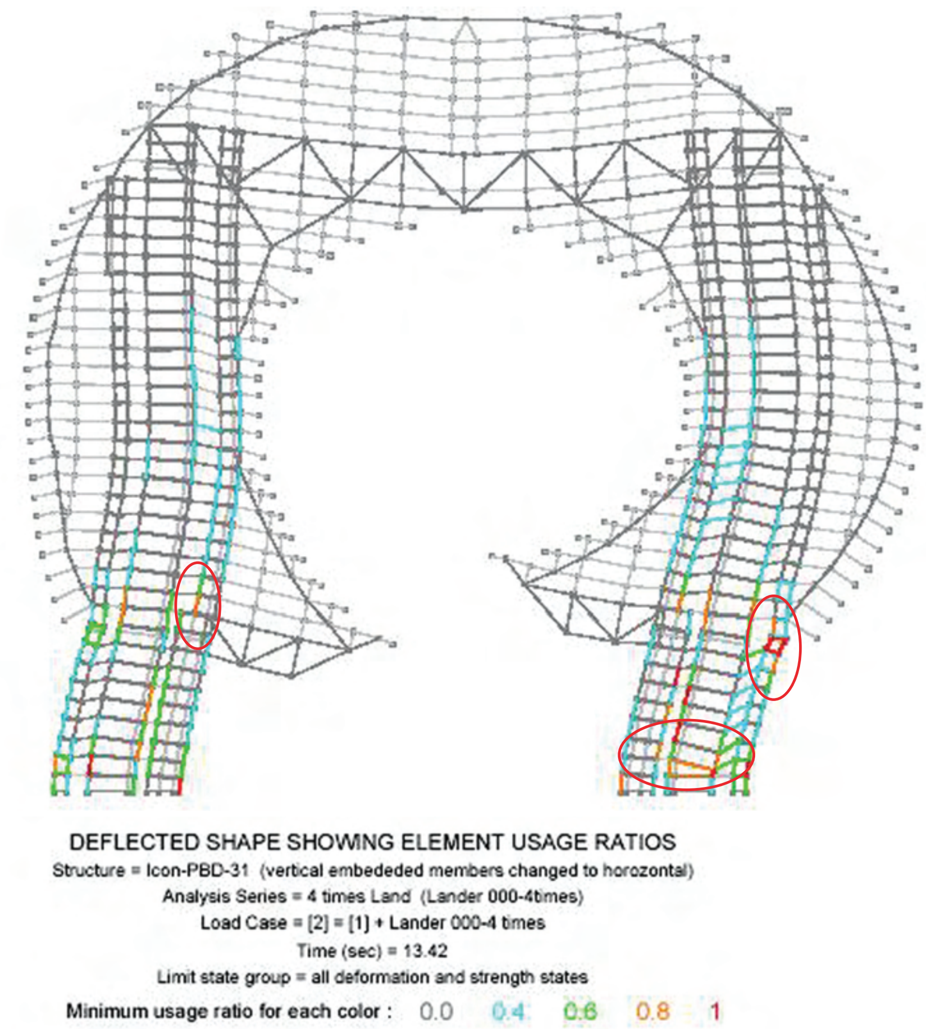

Figure 32 - Concrete crushing under four times stronger earthquake

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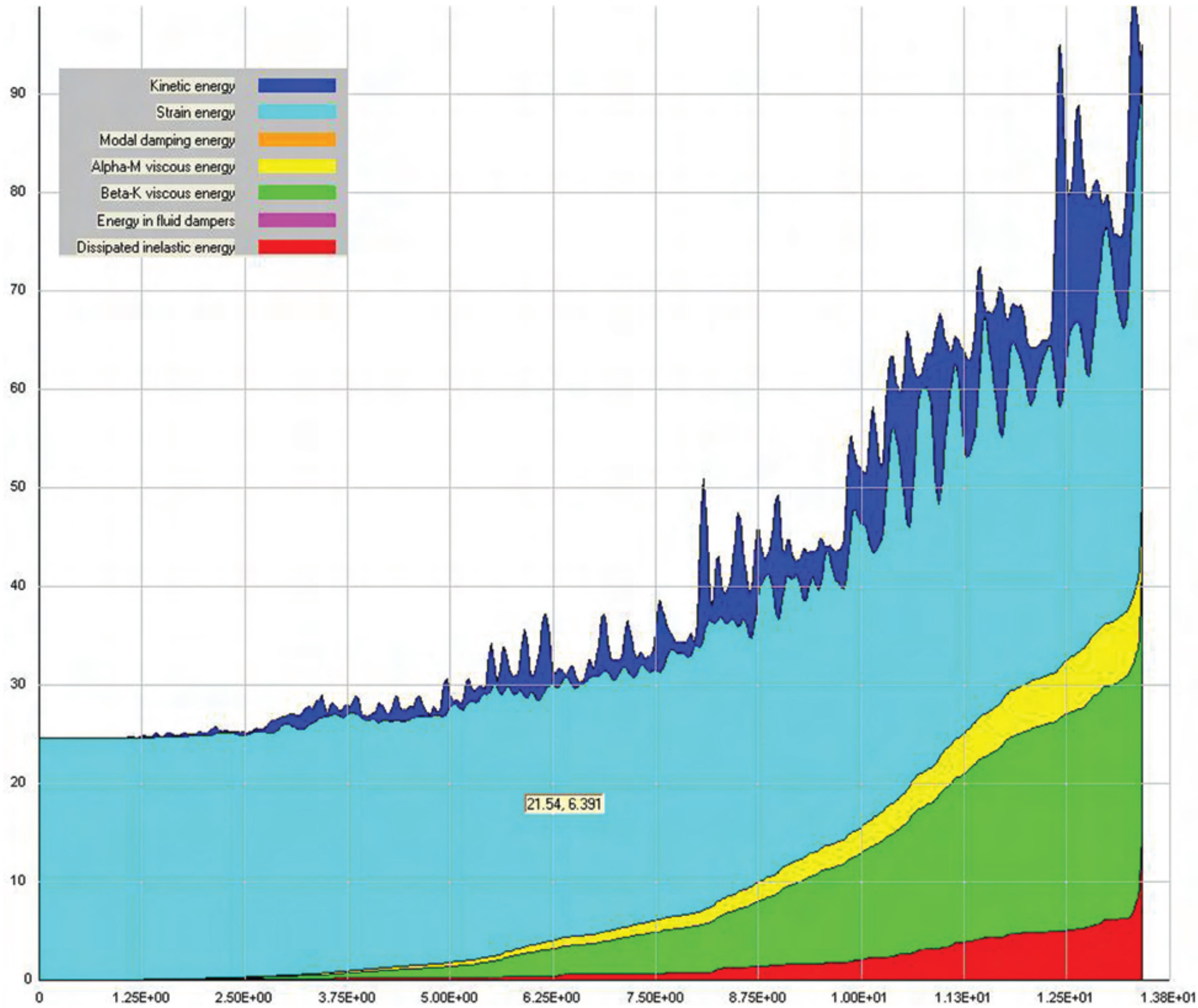

Figure 33 - Dissipated energy by different mechanisms-scaled up four times Lander 000

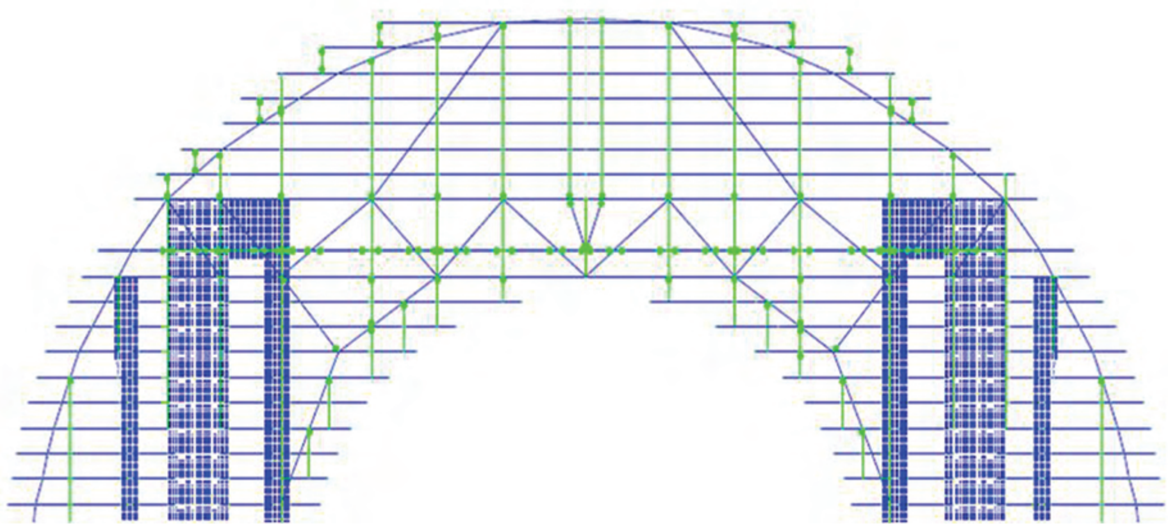

Figure 34 - The proposed new braces

Conclusion

In this study, structural system of Icon Hotel for gravity and lateral loads were presented. Performance based seismic design approach was selected to evaluate the seismic behaviour of the building for anticipated future earthquake. In this regards nonlinear time history analysis was used and modelling assumptions were presented. Following conclusions were made:

(1) Some of the slender coupling beams behave inelastic under gravity loads. These coupling beams need to be strengthened or changed to steel beams.

(2) Scaled time histories applied to the structure and acceptance criteria were evaluated at each time step. Results show that steel member generally remain elastic and building essentially meet the desired performance objectives.

(3) Analysis of animation through each time steps revealed that steel arch is more flexible than shear walls. To avoid the out of phase movement of top portion, additional steel bracing added to the structure above the shear walls.

(4) Review of the dissipated energy by different mechanism showed that most of the earthquake energy is dissipated by elastic strain energy and some part by viscous damping. This behaviour indicates the low ductility of system and comparably high over-strength of bearing wall system.

(5) Scaled Lander 000, which creates the maximum inter-story drift, was magnified 4 times and applied to the structure. Although the applied load was 4 times more than the anticipated load but concentration of concrete crushing in specific areas lead us to identify the weak points. Additional confined boundary zone added to first 15 levels of shear walls to avoid sudden concrete crushing and more ductile behaviour.

(6) Performance Based Seismic Design approach can provide very useful information for the design of tall building structures. Due to the improvement of computers speed and availability of commercial nonlinear software systems, this methodology can be used in design offices as normal day to day practice.

Acknowledgments

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Design charts for the assessment of grandstands subject to dynamic crowd action

Graham Parkhouse

Consultant

Design & Engineering Solutions

Ian Ward

Associate Director

Design & Engineering Solutions

Abstract

The dynamic response of stadium and grandstand structures during lively sporting events or pop concerts should not be a cause of discomfort and never of alarm. Such adverse structural behaviour may occur when the crowd is responding to a musical beat on a structure that is sensitive to frequencies below 6Hz. A natural frequency limit of 6Hz has been used for some years as a design criterion. New UK recommendations for the dynamic assessment of grandstands now offer engineers an alternative set of criteria based on a new assessment method that predicts the vibration that may actually occur on a particular structure during a particular type of event.

The new method takes advantage of recent UK research and is novel in that it takes account of the mechanics underlying human structure interaction. The new method is more lenient than the 6Hz frequency criterion, showing some structures with a lower natural frequency to be acceptable. This paper describes the new assessment method and provides design charts based upon it.

Introduction

An important aspect of grandstand design is ensuring that crowd loading cannot cause excessive dynamic motion of the structure, resulting in discomfort or panic in the crowd. This is potentially a problem when crowds move rhythmically, e.g. to music, and excite the structure at one of its natural frequencies.

In 2000, a Working Group was set up by the Institution of Structural Engineers and two government departments to look at this issue. The group has recently published Recommendations[1] in which two methods for assessing the ability of a structure to withstand dynamic crowd loads are presented. A grandstand may be considered to meet these Recommendations if the requirements of either one of the two methods are met. The first method, termed Route 1, is based on the previous guidance document[2] and judges acceptance using only the lowest relevant natural frequency of the structure. The second method, Route 2, is a new approach which enables engineers to predict the likely vibration levels and to compare them against acceptable limits based on human comfort and safety.

This more rigorous approach provides a way of demonstrating compliance of a structure which may have otherwise failed the Route 1 test. In addition, it enables engineers to investigate the effect of management actions aimed at improving the dynamic performance of the structure e.g. by keeping rows of seating unoccupied.

The new assessment method is a significant development for engineers engaged in the assessment of grandstand dynamic behaviour. This paper discusses the basis of the new method and provides design charts to assist in its practical implementation. The design charts offer a rapid means of assessment requiring only the results from a natural frequency analysis of the empty structure. The charts also clearly illustrate how factors other than natural frequency affect the dynamic performance of grandstands subject to crowd loading.

The new assessment method

Previous assessment methods[3,4,5] have considered dynamic crowd action as an oscillating external load applied to an empty structure, as shown in Figure 1(a). This can be modelled by the single degree of freedom (SDOF) system, shown on the right of the figure.

The new (Route 2) assessment method is explained in Appendices 1 and 2 of the Recommendations. It requires the dynamic characteristics of the crowd to be modelled as well as those of the structure, to take account of human-structure interaction effects. This is a novel departure from established practice, and since it takes better account of the underlying mechanics, it results in a more realistic prediction of structural response[6]. A reason why modelling the crowd has a significant effect on structural response is their relatively high damping, 25% of critical for an active crowd standing with legs bent, compared with about 2% of critical for the empty structure.

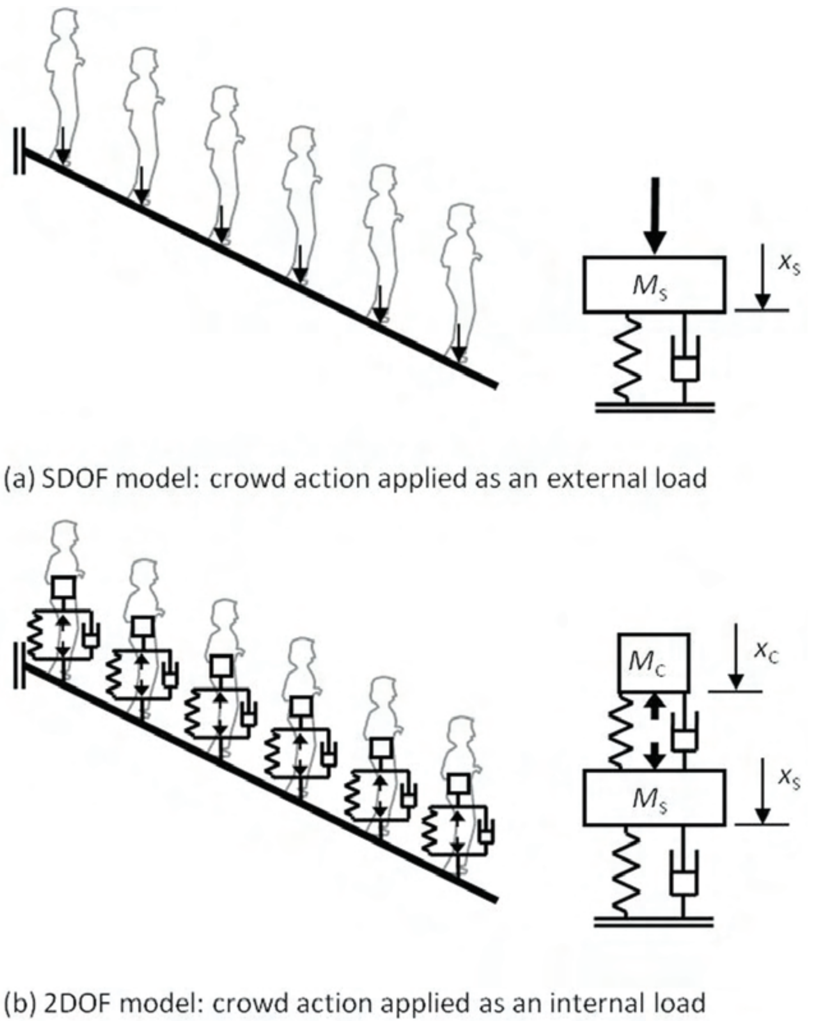


Figure 1 - Equivalent SDOF and 2DOF models of the crowd-structure system

In the new assessment method the most accurate way to model the human-structure interaction effects is to add 'crowd units' to a finite element model of the empty structure and

to analyse the combined system, providing a comprehensive analysis of the effect of a particular crowd on a structure. Each crowd unit would have a mass equal to that of the crowd it represents, connected to the structure by spring and damper elements (Fig 1b).

The exciting force generated by a crowd unit is represented by a vertical force pair simulating muscle action inside the crowd unit. Appendix 2 of the Recommendations defines the properties of the crowd units and the force amplitudes for three harmonics having frequencies of 1, 2 and 3 times the activity frequency. The magnitudes of these forces depend on the activity frequency, being greatest when the activity frequency is around 2Hz.

A sweep through a range of activity frequencies allows a maximum acceleration or displacement to be calculated for all significant modes of the empty structure (a significant mode being one by which people can excite the seating deck through vertical movement and feel its motion).

The Recommendations consider four different types of crowd loading, associated with Design Event Scenarios, ranging from Scenario 1, a low profile sporting event with a relaxed viewing public, to Scenario 4 which covers high energy events such as pop/rock concerts with vigorous participation of the crowd. Scenario 1, the least lively, is not a candidate for a Route 2 assessment. Acceleration and displacement, not frequency, are the criteria for a Route 2 assessment and the Recommendations give acceptable values of each for the different scenarios. The fact that the Route 2 assessment method is applied to all significant modes avoids the difficulty associated with a Route 1 assessment, that of distinguishing just one significant mode amongst other modes (potentially with lower natural frequencies).

An approximate solution can be obtained by treating the grandstand and crowd as a 2 degree of freedom (2DOF) system, using an assumed mode shape for the displacement of the crowd (discussed in Section A.1.3.2 of the Recommendations). This transforms the crowd-structure system (Figure 1b) to the 2DOF system on the right of the figure, with x_{c} and x_s denoting the displacement of the crowd and structure respectively. This 2DOF system is the basis for the simplified Route 2 assessment method presented in this paper and described in the flow chart in Figure 2.

Underlying the simplified method is the assumption that the crowd units adopt mode shapes

that are identical to those of the empty structure in their vicinity. The response of the structure to vertical excitation can then be characterised by just three parameters:

- fs, the natural frequency of the empty structure
- μ, the modal mass ratio, of the modal mass of the crowd to the modal mass of the empty structure
- λ, the crowd location factor.

The way these three parameters are calculated for a particular structure and crowd arrangement is given in Figure 2.

The distribution of the crowd in relation to the mode shape affects the modal mass ratio and determines the crowd location factor. This factor has two terms: the term ϕ_{iv}/M_c, which relates to the loading, and the term ϕ_0, which relates the result to the location where the vibration will be judged. The modal force that should be applied to the 2DOF system is $\phi_{iv}\, gG$ where g is the acceleration due to gravity and G is the Generated Load Factor defined in the Recommendations. So the term ϕ_{v}/M_c is the factor that when multiplied by $M_c gG$ gives the correct loading. Both the modal mass ratio, μ, and the crowd location factor, λ, are unaffected by arbitrary scaling of the set of ϕ values.

Simplified Route 2 Assessment Procedure

Step 1

Perform a modal analysis of the empty structure. For each mode this will provide a natural frequency (f_S), modal mass (M_S) and mode shape ($\phi_1, \phi_2 \ldots \phi_n$ for each of the points 1, 2 … n on the model at the foot of a crowd unit). Each of the mode shape values will have a vertical and horizontal component, denoted by subscripts v and h, as shown in the figure.

Step 2

For each significant mode:

1. Calculate the modal mass of the crowd:

$$M_C = \sum m_i \phi_{iv}^{\ 2} \qquad (1)$$

where m_i is the mass of the i^{th} crowd unit and each crowd member represented by the crowd unit contributes 80kg

2. Calculate the modal mass ratio:

$$\mu = M_C / M_S \qquad (2)$$

3. Calculate the crowd location factor:

$$\lambda = \phi_O \left| \sum m_i \phi_{iv} \right| / M_C \qquad (3)$$

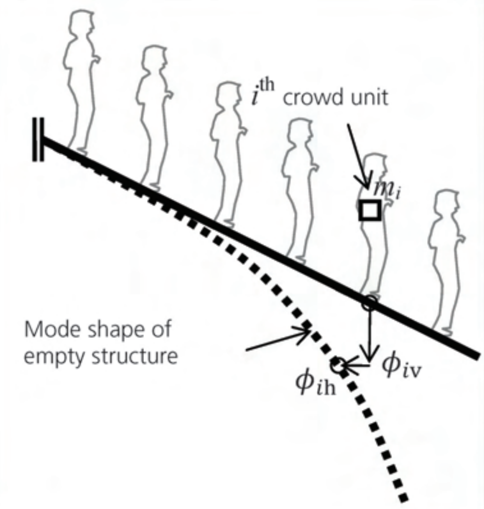
where ϕ_O is the resultant value of the mode shape at the point on the model where the vibration is to be judged. For acceleration this is the point in the crowd where the maximum acceleration is felt, e.g. in the front row of a cantilever tier, and for displacement it is the point where the maximum displacement is seen.

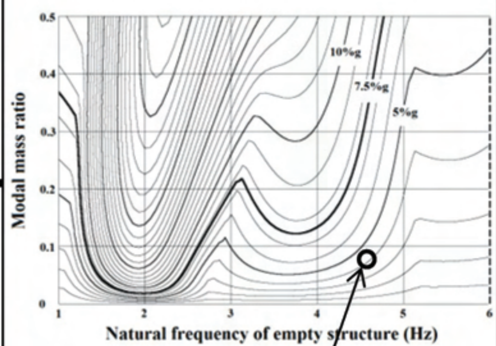
The resultant value of ϕ_O is $\sqrt{\phi_{Ov}^{\ 2} + \phi_{Oh}^{\ 2}}$.

4. Read an RMS acceleration or displacement from the appropriate chart (Figures 3, 4, 5 or 6), at the point (f_S, μ).

5. Multiply this value by $\lambda/1.5$.

The RMS acceleration or displacement so obtained is the predicted value for this particular mode.





From fig. 4, acceleration = 4.1%g ×1.54/1.5
= 4.2%g

Step 3

If there is more than one significant mode, repeat step 2 for each mode. A resultant may then be obtained by taking the square root of the sum of the squares (SRSS) of the values for each mode.

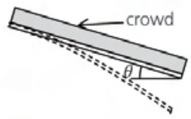
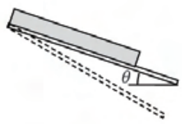
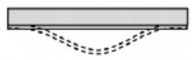
Step 4

Compare the RMS value so obtained with the allowable values[1], i.e.

Scenario 2: 3%g
3: 7.5%g
4: 20%g and 7 mm displacement

Figure 2 - Route 2 assessment method using design charts

Table 1 - Examples of the crowd location factor, λ, calculated using equation 3 of Figure 2, for maximum crowd vibration.

Cantilever, fixed ended Crowd is uniformly distributed		$\lambda = 1.566/\cos\theta$
Rigid cantilever with rotational flexibility at the support Crowd is uniformly distributed over the whole or just the rear portion. The mode shape is linear and zero at the rear end.		$\lambda = 1.5/\cos\theta$
Fixed ended beam Crowd is uniformly distributed		$\lambda = 1.320$
Simply supported beam Crowd is uniformly distributed		$\lambda = 1.273$
Platform For any crowd distribution		$\lambda = 1$

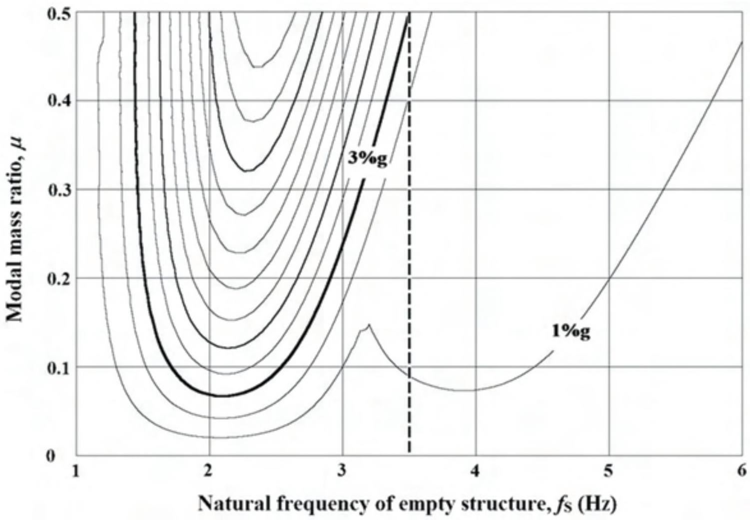


Figure 3 - Predicted RMS accelerations, Scenario 2 loading, with λ = 1.5

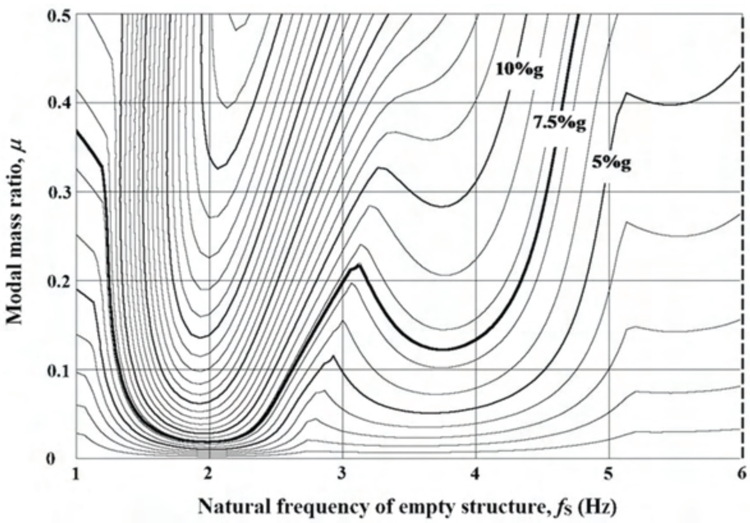


Figure 4 - Predicted RMS accelerations, Scenario 3 loading, with λ = 1.5

The crowd location factor generally varies very little from structure to structure. Its values for some generic structures and crowd configurations are shown in Table 1. The fundamental modes of all cantilevers have values close to 1.5 and fully loaded single span beams have values close to 1.3.

The design charts

Design charts have been obtained by analysing 2DOF systems for a range of structure natural frequencies and modal mass ratios. Charts for Scenarios 2, 3 and 4 are shown in Figures 3-6. These charts apply to structures with a value of λ = 1.5; for those structures which have a value of λ other than 1.5, the responses read from the charts each need to be factored by λ/1.5.

The design charts show that, when the assessment of a structure is by Route 2, its response to dynamic crowd action depends primarily on two parameters: the modal mass ratio and the empty natural frequency. The heaviest contour on each chart defines the acceptance criterion: structures characterised by points below the heaviest contour pass. It is apparent that acceptability is more likely to be achieved with higher natural frequencies and with lower modal mass ratios.

Only the former criterion is acknowledged by Route 1; the Route 1 acceptance criteria are bounded by the vertical dashed lines on each chart. The design charts show that Route 2 assessments are much less restrictive than Route 1 assessments.

Scenario 2 loading

Scenario 2 events are typified in the Recommendations as a ‘classical concert and typical well attended sporting event’, with the ‘audience seated with only few exceptions – minor excitation’. The criteria for Route 1 is for a relevant natural frequency greater than 3.5Hz, and, for Route 2, an RMS acceleration of less than 3%g. Figure 3 shows that structures that have a natural frequency less than 3.5Hz, which therefore fail the Route 1 test, may have low enough modal mass ratios to pass the Route 2 criterion of 3%g.

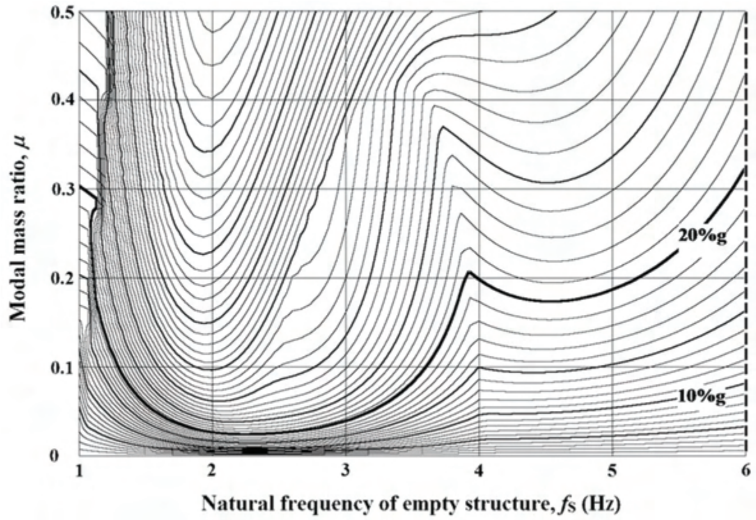


Figure 5 - Predicted RMS accelerations, Scenario 4 loading, with λ = 1.5

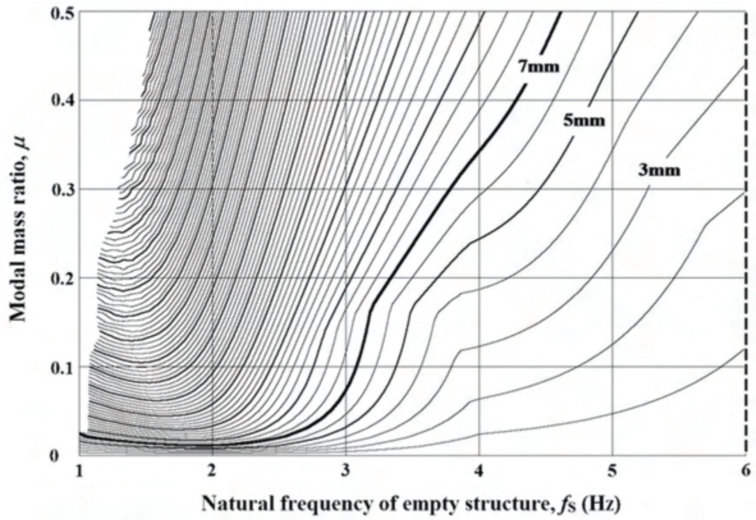


Figure 6 - Predicted RMS displacements, Scenario 4 loading, with λ = 1.5

Scenario 3 and 4 loadings

Scenario 3 events are 'commonly occurring events including, inter alia, high profile sporting events and concerts with medium tempo music and revival pop-concerts with cross generation appeal', with a 'potentially excitable crowd' who could all be 'standing and participating during some part of the programme'.

Scenario 4 events are 'more extreme events including high energy concerts with periods of high intensity music'. The crowd is 'excited, mostly standing and bobbing with some jumping', composed of people who are 'mainly young and active with vigorous participation'.

Scenario 4 is livelier than Scenario 3, and the expectation of comfort is different: Scenario 3 crowds expect a degree of comfort while Scenario 4 crowds are prepared to accept some discomfort, but vibration must not be allowed to reach levels that could cause significant distress or risk of panic. The criterion for Route 1 for both scenarios is for a natural frequency greater than 6Hz, and, for Route 2, an RMS acceleration of less than 7.5%g and 20%g- respectively. Figures 4 and 5 only show characteristics of structures that would fail the Route 1 criterion, yet, there is a zone defining a set of structures that are acceptable according to a Route 2 assessment.

Comparing Figures 4 and 5, Scenario 4 is generally a harsher criterion than Scenario 3, but structures with a natural frequency around 4Hz are more likely to fail the Scenario 3 criterion. This is because Scenario 3 loading is frequency weighted according to the probability of occurrence of songs in the overall pop repertoire, and 2Hz is a very popular tempo with a high chance that the second harmonic will significantly excite the structure at 4Hz.

Displacements

The Recommendations place a separate limit on displacement, of 7mm RMS, in consideration of visible movement that might cause alarm. The displacement response for Scenario 4 is shown in Figure 6. Comparing the thickest contour of this figure with that of Figure 5, it is apparent that only for structures with a natural frequency of less than 2.5Hz is displacement rather than acceleration the critical criterion. No similar chart has been provided for Scenario 3 loading, but it shows that only for structures with a natural frequency of less than 2Hz is displacement rather than acceleration the critical criterion. Acceleration is always the governing criterion for Scenario 2 loading.

A possible exception to this rule may occur when a global mode, excitable by the crowd, contributes to large displacements away from the crowd, in the roof, for example. Then, a check on displacements will be appropriate.

Sensitivity to natural frequency discrepancies.

The Recommendations (Section 4.1) highlight the fact that there can be significant differences between predicted natural frequencies and those subsequently obtained from testing. If predicted values of natural frequency are being used as the basis for the acceleration checks, it would be prudent to allow for natural frequency variations of, say, ±15% from the analytical results. The design charts readily show the effect of such variations on the resulting accelerations/displacements.

Sensitivity to structural damping

For a SDOF system, such as that of Figure 1a, the structural response at resonance is inversely proportional to the damping, so that if the structural damping decreases slightly by   % then, to a first approximation, the response increases by η- %.

STRUCTURES

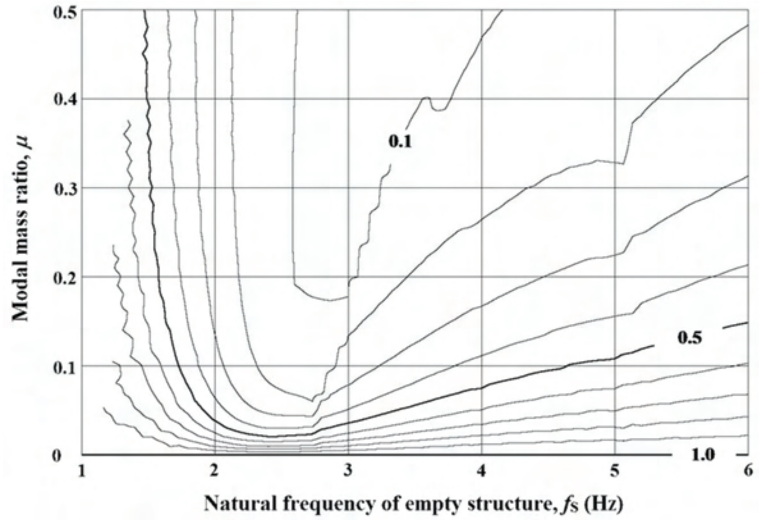


Figure 7 - Sensitivity, η, of Scenario 3 and 4 accelerations to changes in structural damping

However, for the 2DOF system representing the crowd and the structure (Figure 1b), the change in response is also governed by the damping in the crowd. Because the damping of the crowd is generally greater than that of the structure, a structure's response to dynamic crowd action can be relatively insensitive to changes in its own damping, at least for structures with commonly occurring natural frequencies and modal mass ratios. The sensitivity to the damping in this case is expressed through the parameter , so that if the structural damping decreases slightly by % then the response increases by %.

The design charts are based on a structure having a damping ratio of 2% for each of its modes, as proposed in the Recommendations. Figure 7 shows the sensitivity, , of Scenario 3 and Scenario 4 accelerations to small changes in structural damping away from 2%. It shows how the sensitivity decreases from a value of unity as the modal mass ratio increases.

To illustrate the use of η, consider the example shown in Figure 2. Figure 7 shows that, for f_s = 4.61Hz and μ = 0.078, the sensitivity value, η, equals 0.6. Therefore, if the structure used in this example had a damping value of 1.8% rather than 2% (i.e. 10% lower) then the acceleration would be a factor of 0.6×10% higher, i.e. 6% higher than the value read off from the design chart (4.45%g rather than the 4.2%g quoted in Figure 2).

Worked examples

Three different grandstand structures are assessed using the simplified method outlined in Figure 2 and they are presented at the end of this paper with the appropriate modal information as worked examples. In each case 'accurate' answers, obtained by analysing the structure with all the crowd units represented as shown on the left of Figure 1b, are also provided to give an indication of errors likely to be associated with the simplified method. The examples chosen are typical of structures that fail the Route 1 natural frequency criteria but which pass the more realistic Route 2 vibration response criteria.

2D models have been used for the examples because they demonstrate the method and the issues most clearly; the method and the principles apply equally to 3D models. Guidance on important factors to be included in analytical models is provided in the Recommendations (Appendix 3). The worked examples presented in this paper have been analysed using the DynamAssist structural dynamics software[7].

Example 1

The grandstand of Example 1 has two natural frequencies between 4 and 5Hz, but both of these modes combine significant sway of the main structure with vertical motion of the cantilever, which results in low modal mass ratios. It would fail the Route 1 criterion of 6Hz for Scenario 3, but passes the Route 2 criterion of 7.5%g.

Example 2

This grandstand is assessed for Scenario 2. A Route 1 assessment requires relevant natural frequencies to be above 3.5Hz, so its 2.23Hz mode would cause it to fail. However, it might be argued that the 2.23Hz mode is really a global sway mode and will not be significantly excited by dynamic crowd action, so is not a relevant mode. On such grounds the grandstand might be allowed to pass a Route 1 assessment. The benefit of a Route 2 assessment is that such judgements are unnecessary; the calculated RMS acceleration of 1.5%g demonstrates that the grandstand easily passes.

Examples 3 and 4

The initial assessment to Scenario 4 with the tier fully occupied fails, as shown by Example 3, because the calculated RMS acceleration on the front row of the cantilever is 24.5%g, well above the 20%g-limit. Example 4 illustrates how a Route 2 assessment can be used to determine what management actions can be taken for the grandstand to become compliant. The same structure now has two rows of seats, represented by node 9, unoccupied. This reduces one modal mass ratio from 0.300 to 0.143, and reduces the maximum RMS acceleration experienced by the crowd, at node 8, to just under the 20%g limit.

Conclusions

Recently published recommendations[1] have introduced a new analysis method for predicting vibration levels that can be generated by crowds of people in a grandstand, taking account of the effects of human-structure interaction.

This paper has presented a simplified procedure for applying the new method, which only requires a modal analysis of the empty structure and knowledge of the crowd's location. The procedure has been used to derive design charts, showing the sensitivity of grandstand response to natural frequency and modal mass ratio.

A number of worked examples have been provided to demonstrate the application of the procedure. Results calculated more accurately have been presented alongside those from the simplified method. These show that the simplified method gives good agreement with the more precise method.

Previous guidance on grandstand dynamics used only the natural frequency of the structure to judge the acceptance of the structure. The design charts clearly demonstrate that the modal mass ratio also has a very significant effect on the dynamic response. Indeed, grandstands that may have failed the previous check based solely on natural frequency may be found to be acceptable when the modal mass ratio is taken into account.

The design charts can readily be used to consider the effect of changes in natural frequency and modal mass ratio on the likely acceleration levels. In particular, the effect of modal mass ratio can be used to confirm the suitability of crowd management decisions. This has been demonstrated in the Worked Examples, where a decision to keep the front two rows of a grandstand's cantilever unoccupied made the structure suitable for a Scenario 4 event.

The design charts are based on a structural damping ratio of 2%, as specified in the Recommendations. The effect of different structural damping ratios has been considered. It has been found that, because the damping of the crowd in a full stadium is generally greater than that of the structure, a grandstand's response to dynamic crowd action is relatively insensitive to changes in its own damping, except when there is a very low modal mass ratio. This is particularly significant if considering the use of additional damping as a remedial measure; the improvement in performance will be less than anticipated if human-structure interaction is ignored.

Acknowledgements

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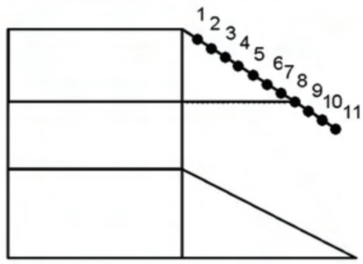
Worked examples

Grandstand Example 1

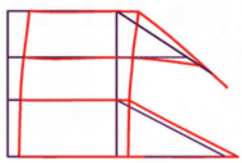
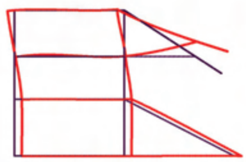
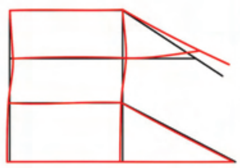
Assessment for Event Scenario 3

Damping ratio of structure = 2%

Mass of each crowd unit = 960kg



2D model of structure
showing crowd location

	Mode 1		Mode 2		Mode 3	
Natural frequency	4.09Hz		4.61Hz		6.09Hz	
Modal mass	247 tonne		45.9 tonne		29.1 tonne	
Mode shape						
Mode shape values	node	value	node	value	node	value
Vertical component	1	-0.050	1	0.095	1	0.115
	2	-0.081	2	0.152	2	0.170
	3	-0.119	3	0.221	3	0.235
	4	-0.162	4	0.293	4	0.303
	5	-0.218	5	0.385	5	0.388
	6	-0.275	6	0.478	6	0.474
	7	-0.335	7	0.576	7	0.567
	8	-0.397	8	0.678	8	0.666
	9	-0.458	9	0.784	9	0.774
	10	-0.520	10	0.892	10	0.886
	11	-0.581	11	1.000	11	1.000
Horizontal component	11	0.645	11	0.288	11	0.580
Modal mass ratio	0.005		0.078		0.123	
Crowd location factor	2.20		1.54		1.74	
Scenario 3 acceleration	1.0%g		4.2%g		3.0%g	

RMS displacement at node 11 due to all modes	0.7mm	(0.8mm*)
RMS acceleration at node 11 due to all modes	5.2%g	(4.5%g*)
Allowable rms acceleration for Scenario 3	7.5%g	∴ Pass

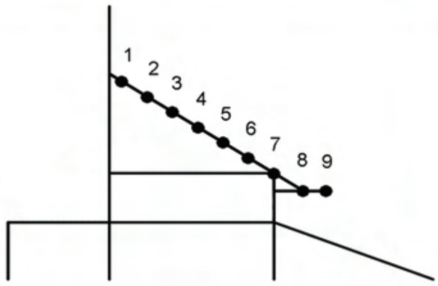
* Calculated using the direct method with crowd units.

Grandstand Example 2

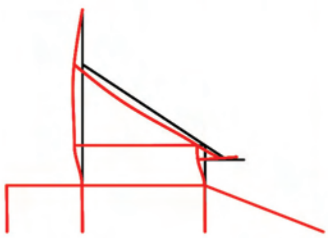
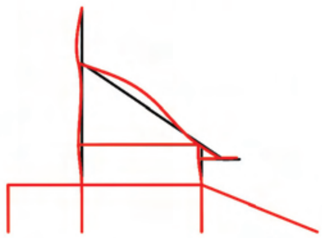
Assessment for Event Scenario 2

Damping ratio of structure = 2%

Mass of each crowd unit = 1740kg



2D model of structure showing crowd location

	Mode 1		Mode 2	
Natural frequency	**2.23Hz**		**4.15Hz**	
Modal mass	**70.8 tonne**		**30.9 tonne**	
Mode shape				
Mode shape values	**node**	**value**	**node**	**value**
Vertical component	1	-0.101	1	0.239
	2	-0.262	2	0.695
	3	-0.341	3	0.974
	4	-0.315	4	0.944
	5	-0.202	5	0.637
	6	-0.061	6	0.239
	7	0.015	7	0.037
	8	0.136	8	0.086
	9	0.257	9	0.138
Horizontal component	3	-1.000	3	0.302
Modal mass ratio	**0.010**		**0.162**	
Crowd location factor	**2.18**		**1.42**	
Scenario 2 acceleration	**0.7%g**		**1.3%g**	

RMS displacement at node 3 due to both modes	**0.4mm**	**(0.4mm*)**
RMS acceleration at node 3 due to both modes	**1.5%g**	**(1.4%g*)**
Allowable rms acceleration for Scenario 2	**3%g**	**∴ Pass**

*** Calculated using the direct method with crowd units.**

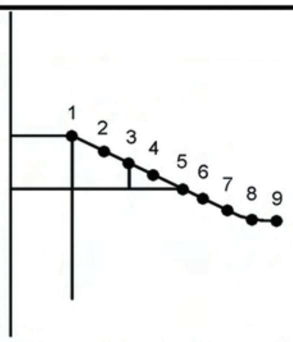
STRUCTURES

Grandstand Example 3

Assessment for Event Scenario 4

Damping ratio of structure = 2%

Mass of each crowd unit = 1800kg



2D model of structure showing crowd location

	Mode 1	Mode 2
Natural frequency	**1.33 Hz**	**3.90 Hz**
Modal mass	**262.9 tonne**	**11.5 tonne**
Mode shape		

Mode shape values	node	value	node	value
Vertical component	1	-0.017	1	0.010
	2	-0.094	2	0.018
	3	-0.167	3	0.035
	4	-0.252	4	0.078
	5	-0.361	5	0.185
	6	-0.441	6	0.306
	7	-0.542	7	0.502
	8	-0.646	8	0.725
	9	-0.754	9	1.000
Horizontal component	9	0.156	9	0.320

	Mode 1	Mode 2
Modal mass ratio	**0.012**	**0.300**
Crowd location factor	**1.48**	**1.57**
Scenario 4 acceleration	**3.9%g**	**24.2%g**

RMS displacement at node 9 due to both modes	**8.8mm**	**(6.4mm*)**
RMS acceleration at node 9 due to both modes	**24.5%g**	**(22.9%g*)**
Allowable rms acceleration for Scenario 4	**20%g**	**∴ Fail**

*** Calculated using the direct method with crowd units.**

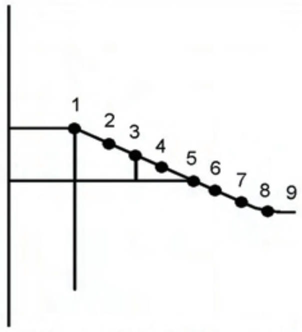
Grandstand Example 4

(as Example 3 but with front 2 rows empty)

Assessment for Event Scenario 4

Damping ratio of structure = 2%

Mass of each crowd unit = 1800kg



2D model of structure showing crowd location

	Mode 1	Mode 2
Natural frequency	**1.33Hz**	**3.90Hz**
Modal mass	**262.9 tonne**	**11.5 tonne**
Mode shape		

Mode shape values	Mode 1 node	Mode 1 value	Mode 2 node	Mode 2 value
Vertical component	1	-0.017	1	0.010
	2	-0.094	2	0.018
	3	-0.167	3	0.035
	4	-0.252	4	0.078
	5	-0.361	5	0.185
	6	-0.441	6	0.306
	7	-0.542	7	0.502
	8	-0.646	8	0.725
	9	-0.754	9	1.000
Horizontal component	8	0.163	8	0.305
	9	0.156	9	0.320

	Mode 1	Mode 2
Modal mass ratio	**0.008**	**0.143**
Crowd location factor	**1.48**	**1.60**
Scenario 4 acceleration	**2.7%g**	**19.6%g**

RMS displacement at node 9 due to both modes	**6.5mm**	**(4.8mm*)**
RMS acceleration at node 8 due to both modes	**19.7%g**	**(19.2%g*)**
Allowable rms acceleration for Scenario 4	**20%g**	**∴ Pass**

*** Calculated using the direct method with crowd units.**

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www.DynamAssist.com

Design of the Olympic Park Bridges H01 and L01

David Baird

Olympic Delivery Authority

Chris Hendy

Head of Bridge Design and Technology

Highways & Transportation

Patrick Wong

Senior Engineer

Highways & Transportation

Rachel Jones

Senior Engineer

Highways & Transportation

Tony Sollis

Senior Engineer

Highways & Transportation

Hayden Nuttall

Structural Design Director

Design & Engineering Solutions

Abstract

In preparation for the 2012 Olympic and Paralympic Games, many bridges were required crossing over waterways, highways and railways. This paper describes the technical challenges on two of the Atkins designed bridges - H01 and L01.

Bridge H01 is a single-span integral highway bridge of composite steel box-girder construction that carries a local distributor road over the River Lea in the north of the Olympic Park. It was the first box girder bridge to be erected in the Olympic Park and was opened to construction traffic in October 2009. It is planned to be opened for public use in December 2010 for access to Stratford City. Box girders are torsionally stiff in their completed state but are less so during construction if they are "open top", as here, and require restraint to the compression flange during deck slab construction to prevent lateral buckling. The design of Bridge H01 utilised vertical cross-bracing at regular centres for this purpose, but detailed analysis indicated that this alone was insufficient to prevent significant second order twisting effects under torsional loading. The paper discusses how the designers addressed these issues and how updated national box girder design guidance was produced as a result.

Footbridge L01 crosses Ruckholt Road and provides the vital northern link to the Olympic Park for pedestrians to and from the Northern Spectator Transport Mall. Its construction is expected to be completed in June 2011. The single span structure is a unique form of construction whose behaviour is a hybrid between that of a tied arch and Vierendeel girder. The steel bridge deck itself is 42m long and 5.5m wide, while the arch rib is made exceptionally slender by exploiting the overall Vierendeel girder behaviour. The closely spaced solid plate hangers provide both lateral and in-plane stability while creating a striking effect in elevation. The complex geometry required elastic critical buckling analysis to determine the resistance of the arch and non-linear analysis to assess the effects of arch deformation on the rigid hangers and deck. The main learning points from the analysis and design of this unique structure are presented in this paper.

Introduction

The Olympic Park bridges in Stratford include 11 highway bridges, 13 pedestrian bridges, 6 underpasses and many other temporary bridges with spans up to 56.5 m. Many of them carry major utilities over waterways or railways. The bridge and associated wingwall designs, including finishes, are co-ordinated to fit seamlessly with the Park's topography and landscape during the Games period and in Legacy. Minor modifications to some bridges will be required after the Games for Legacy transformation. This paper describes the particular technical challenges encountered on two of the bridges, H01 and L01.

Bridge H01

Bridge H01 is a 52 m single-span integral highway bridge of steel-concrete composite "open top" box-girder construction. It is situated in the north of the Olympic Park and carries a local distributor road over the River Lea. It was the first box girder bridge to be erected in the Olympic Park and was opened to construction traffic in October 2009. It is planned to be opened for public use in December 2010 for access to the Stratford City development. The general arrangement is shown in Figure 1.

An open top box girder solution was adopted for Bridge H01 for a number of reasons:

- The absence of outstand flanges give a sleek appearance and reduce opportunity for bird roosting.

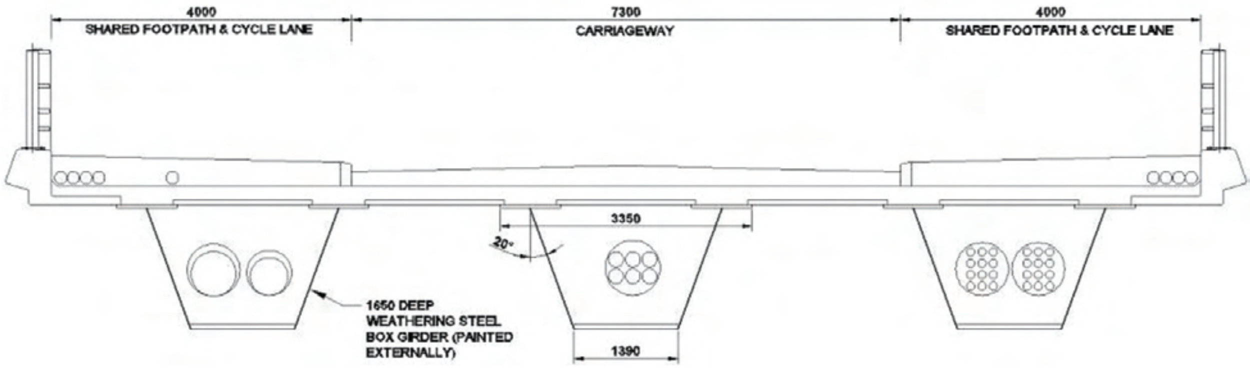

Figure 1 - Cross section of bridge H01

- The inclined webs increase the amount of light under the bridge and reduce the apparent depth of the structure when viewed in elevation
- The services running over the bridge can be concealed within the box (Figure 2), thus improving the aesthetics
- The absence of a steel top flange give material savings compared to closed box solutions and reduced the health and safety risks associated with fabricating closed boxes.

These advantages had to be balanced against a number of disadvantages, however, including:

- Higher fabrication costs and heavier lift weights than for a plate girder
- Greater difficulty in providing the corrosion protection, inspection and maintenance compared to a plate girder due to the internal surfaces. This led to the use of weathering steel to prevent the need for painting the inside of the box
- The need to restrain the outstand top flanges against buckling under wet concrete loading, which would not be required with closed top boxes. This proved to be quite a significant consideration for the particular geometry of bridge H01.

Structural analysis

The structural analysis for the in-service case of bridge H01 was straightforward. The bridge was analysed by grillage analysis following the recommendations of the SCI

Figure 2 - Bridge H01 prior to deck slab construction

STRUCTURES

Design Guide for Composite Box Girder Bridges[1]. A single spine member with outriggers to the positions of the webs was initially used and the distortional flexibility of the boxes was modelled by artificially softening the torsional stiffness of the box; this was an approximation since the distortional flexibility depends on the particular loading applied but the relatively short span, thick plates and small distance between cross braces on Bridge H01 meant that distortional flexibility was very small.

The design for the wet concrete construction case was more challenging and its treatment is much less well documented. During construction, when the concrete slab is not in place, the St. Venant torsional inertia is that of an open section and is therefore several orders of magnitude lower than that of the closed box in the final situation above. However the single spine beam approach with a reduced torsion constant is inappropriate because the open box behaves more like a pair of plate girders in this situation and there is an increase in effective torsional stiffness provided by in-plane bending of the webs and flanges as the cross section twists (the warping stiffness).

Suitable vertical (torsional) bracings or diaphragms are obviously required in this erection condition to give stiffness to the cross-section. Consideration was given to providing top flange plan bracing at this stage to create a pseudo box and eliminate the torsional flexibility. This was ruled out since the plan bracing would have needed to be attached to the underside of the flanges to avoid interfering with deck slab construction and connection to the underside of the flange would have been awkward to construct.

It was clear that the top flanges in compression would need to be checked for buckling. It was initially considered that the open top box could be modelled in the same way as a pair of I-girders with a common flange and that the common flange would effectively provide lateral restraint to overall lateral buckling in the same as does a deck slab in the completed condition of a steel-concrete composite I-girder deck. This is effectively the approach in reference 1 where it is assumed that buckling of the top flange in compression will occur between cross braces and there is no overall instability of the box. There was however concern as to whether the bottom flange was wide enough to provide this lateral stability and whether a global mode might occur.

Boxes with sloping webs, like Bridge H01, are particularly likely to produce this situation. The reason for this is that the elastic critical buckling moment for lateral torsional buckling has the generic form:

$$M_{cr} = \left[\frac{\pi^2 EI_z}{L^2}\left(GI_T + \frac{\pi^2 EI_w}{L^2}\right)\right]^{0.5} \quad (1)$$

I_{w}- is the warping constant;

I_{z}- is the minor axis second moment of area;

I_{T}- is the St. Venant torsional inertia;

L- is the length of the beam between points of torsional restraint.

Expression (1) only applies to bisymmetric sections (not open top boxes) with uniform end moments but it illustrates the resisting components which mainly comprise St Venant torsional stiffness, GIT, and warping stiffness EI_w/L^2. Open top boxes have very low St Venant torsional stiffness so the first term cannot be relied upon to provide any significant resistance (For closed boxes, this term is very large and effectively eliminates lateral torsional buckling for all but the most extreme aspect ratios of tall and narrow boxes). The second term therefore provides the majority of the resistance to lateral torsional buckling.

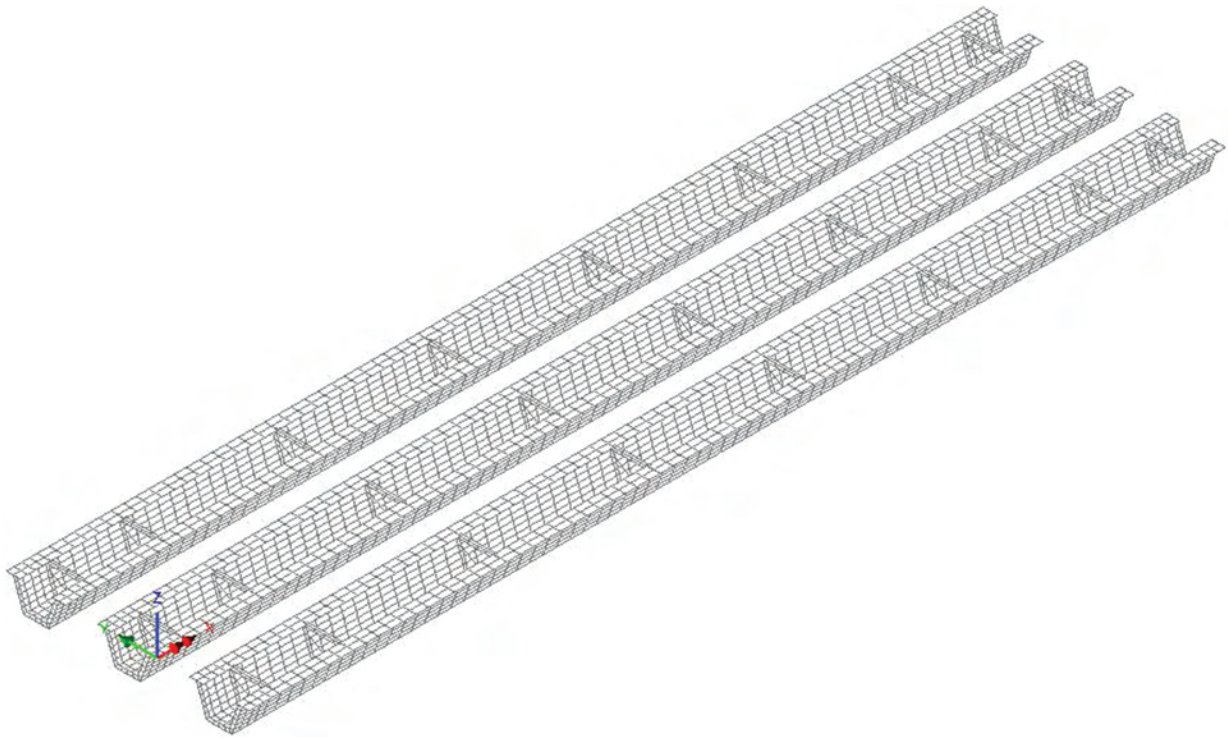

Figure 3 – Shell finite element model for the open top case

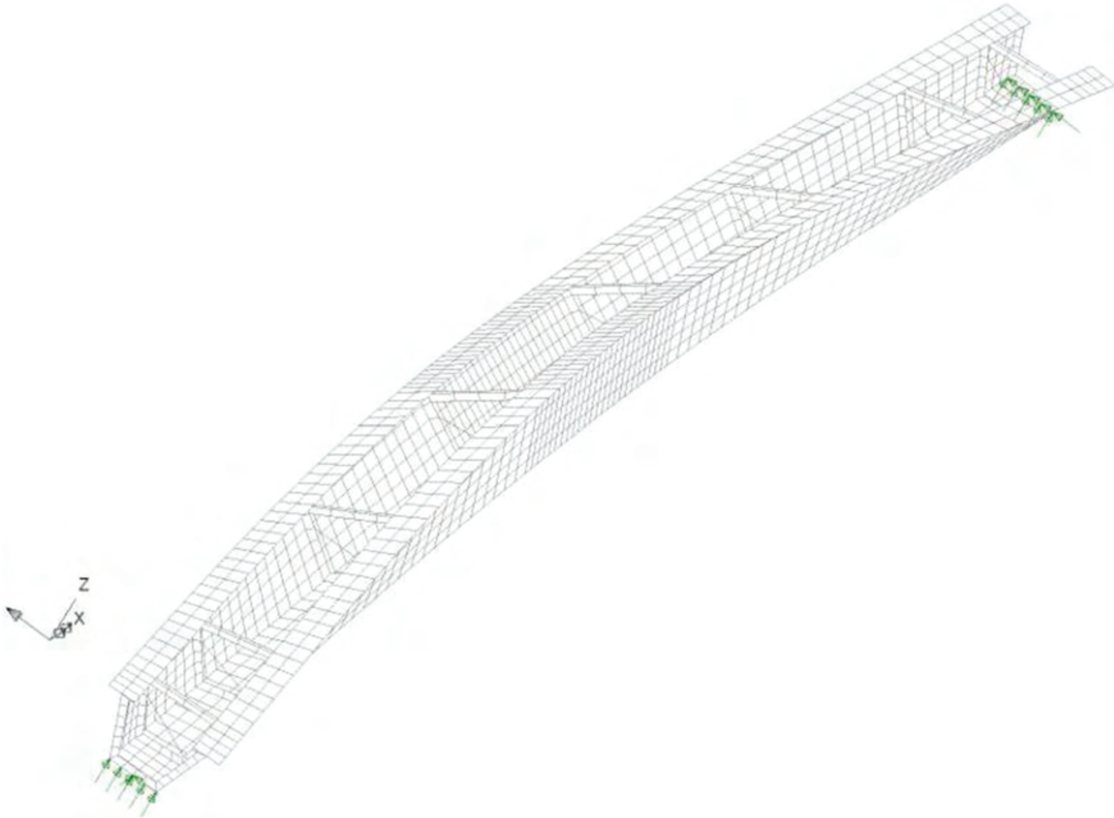

Figure 4 - Lowest buckling mode under application of wet concrete and self weight

The stiffness here depends on how much stiffness the individual plates can provide by in-plane bending when twisted about the cross section's shear centre. Narrow boxes with sloping webs provide less stiffness because of the reduced flange width (and therefore reduced transverse bending stiffness) and the reduced lever arm between the webs. In the limit, a V shaped section with no bottom flange will have a shear centre at the junction between the webs and therefore essentially no warping stiffness. Torsional stiffness is then provided only by the GIT term and the lateral torsional buckling resistance of the overall section will be very low.

There was again consideration given to the provision of plan bracing to the top flange to ensure that the buckling length was restricted to the length between cross braces but it was finally decided to analyse the open top box with a 3-D shell finite element model (Figure 3) to determine Mcr directly and hence obtain the slenderness for buckling following the approach in Eurocode 3.

This analysis was completed and a global mode was indeed obtained as seen in Figure 4. The slenderness for this mode was very much lower than that considering only buckling of the top flange between the cross braces and, although adequacy of the boxes under concreting was demonstrated, there was little reserve of strength available in this condition.

Construction and lessons learnt

The erection of the steelwork went ahead as planned in October 2008. It soon became clear that the girders were more torsionally flexible than had been imagined (Figure 5a), with significant rotation of the girders having occurred under the relatively light loading of empty service sleeves which had not been thought to be significant. Indeed linear elastic analysis with the shell model shown in Figure 4 had predicted an expected rotation causing only a few millimetres difference between the top flange levels. The real rotations produced created a problem with the final levels of the box top flanges and left much reduced tolerance for the slab construction.

Further recourse to the shell model revealed why the twist was so much greater than expected. The ratio of elastic critical buckling load to applied load (the critical load factor, α_{cr}) determined for use in the slenderness calculation was approaching unity for the worst load case. Whilst this ratio had been used to calculate the ULS resistance and demonstrate adequate strength, the second order deflections arising from both initial torsional imperfections and minor eccentric loading had not been calculated directly. Eurocode 3 provides a magnification factor to apply to first order deflections to calculate the resulting deflections including second order effects. This factor is $1/[1-(1/\alpha_{cr})]$. Applying this to the Bridge H01, the rotations predicted by a linear elastic analysis multiplied by the magnification factor gave figures of a similar order of magnitude to those observed on site.

Figure 5 - (a) Twisted box under eccentric load; (b) Addition of plan bracing

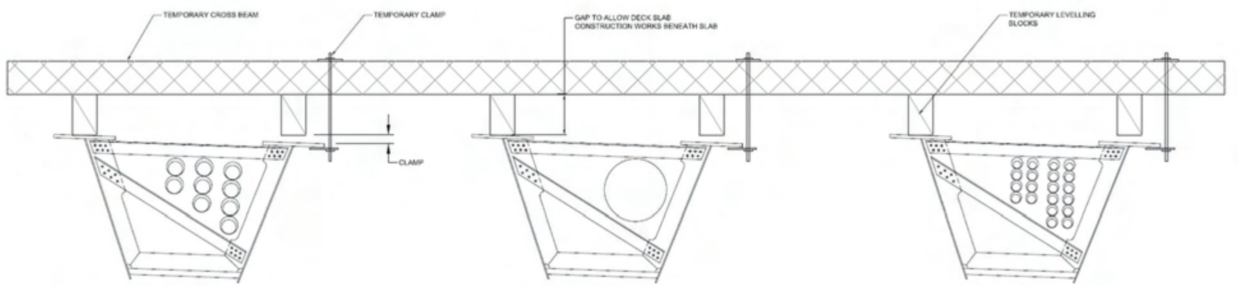

Figure 6 - Temporary cross bracing to reduce box twist

To remedy the situation on site, temporary cross-beams were used to straighten the twist (Figure 6). Initially the option was considered to pour a phased sequence of deck concrete with the cross-beams in place, then removing the cross-beams after deck concrete had cured. However, the final option chosen was to use the cross-beams to straighten the girders, retro-fit permanent plan-bracing with the cross-beams in place (Figure 5b), remove the cross-beams, then pour the concrete as planned. This was thought to offer the least risk. This went ahead without difficulty and the girder stability was maintained by the plan bracing. Clearly the detailing of plan bracing in the first instance would have mitigated the need for this additional construction activity but the decision to omit it had been made with considerations of safety foremost. In hindsight, it would have been better to detail plan bracing, paying particular attention as to how to detail its attachment with the minimum risk to erectors.

The results of this work led to national box-girder design advice being updated by way of an 'Advisory Desk' note published by the SCI in New Steel Magazine[2]. This was particularly necessary as the existing guidance in reference 1 did not draw attention to either the strength or deflection issues associated with open top boxes.

Footbridge L01

Bridge description

Bridge L01 provides a pedestrian and cycle link between the Northern Spectator Transport Mall and the Olympic Park. It will serve as one of the main gateways to the Park during the games and so a striking appearance was a key criterion for the design. Construction of the bridge commenced in May 2010 and is expected to be completed in June 2011. After the 2012 Games it will link the Hackney Marshes playing fields to the Legacy facilities of the park. Figure 7 shows the location of Footbridge L01.

The main architectural feature of the footbridge is the exceptionally slender arch with a dense distribution of steel plate hangers, referred to as slats (Figure 8). The arch spans 42m and each rib is formed from a single steel plate 400mm x 90mm. The vertical slats are formed of 135 mm x 25 mm plates at 135mm centres. The 5.5m wide steel deck is formed from two 550mm x 300mm box sections with transversely spanning "T" ribs at 540 mm centres. The deck is finished with concrete screed, waterproofing and resin bound aggregate surfacing. The deck is tied to the arch with a fully welded connection at deck level. Either side of this connection the vertical slats are replaced with a ribbed steel plate joining the deck and arch. The deck is integrally connected to the foundations which comprise reinforced concrete abutment walls on piles. The bespoke parapet at the ends of the arch match the vertical slats. At midspan, the square hollow section parapet rail is welded directly to the vertical slats.

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The visual concept for the bridge was developed by Allies and Morrison Architects and Atkins and the detailed design was carried out by Atkins. The key challenge of the design was resolving the structural integrity of the bridge without compromising the architectural vision. The resulting structure is a unique form of construction whose behaviour is a hybrid between that of a tied arch and Vierendeel girder; the arch rib achieves its high slenderness by exploiting the overall Vierendeel girder behaviour.

Structural design

The footbridge was designed to BS5400 Part 3[3] but reference was made to BS EN 1993-1-1[4] for checks on buckling of the structure. The bridge was modelled using beam elements representing the main structural elements. The lateral restraint provided by the steel deck plate was not included in the structural model but the section of hand rail welded to the vertical slats was included. Careful consideration needed to be given to the modelling of the foundations as the bridge is integral with them and the fixity to the foundations and foundation stiffness had a significant impact on the structural behaviour.

Figure 7 - 2012 Olympic Park plan

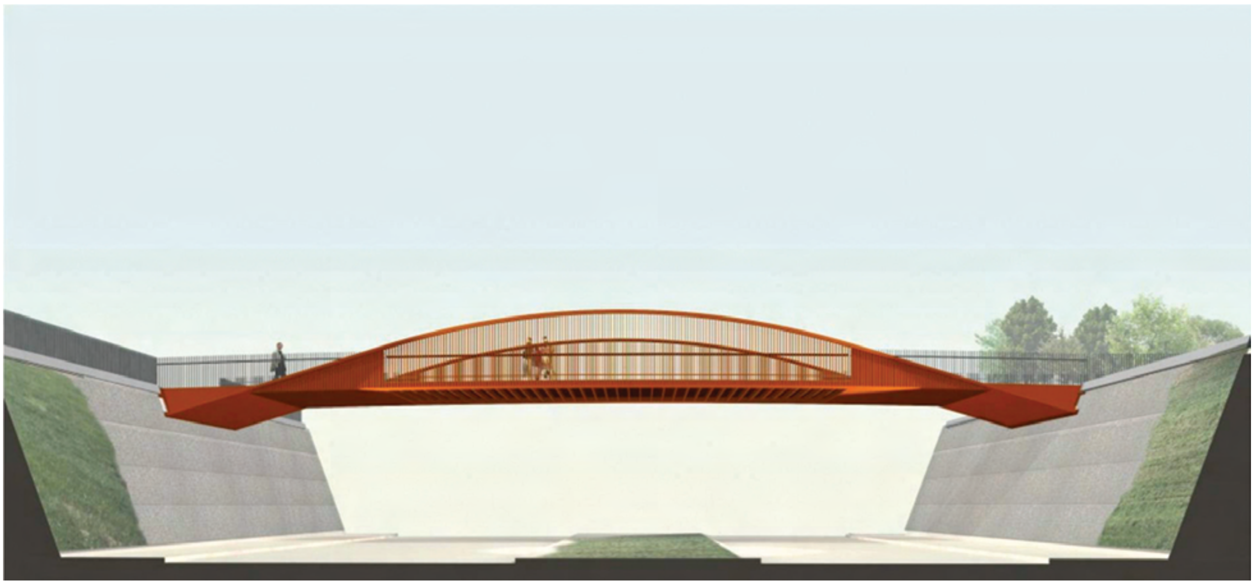

Figure 8 - Final concept for Bridge L01

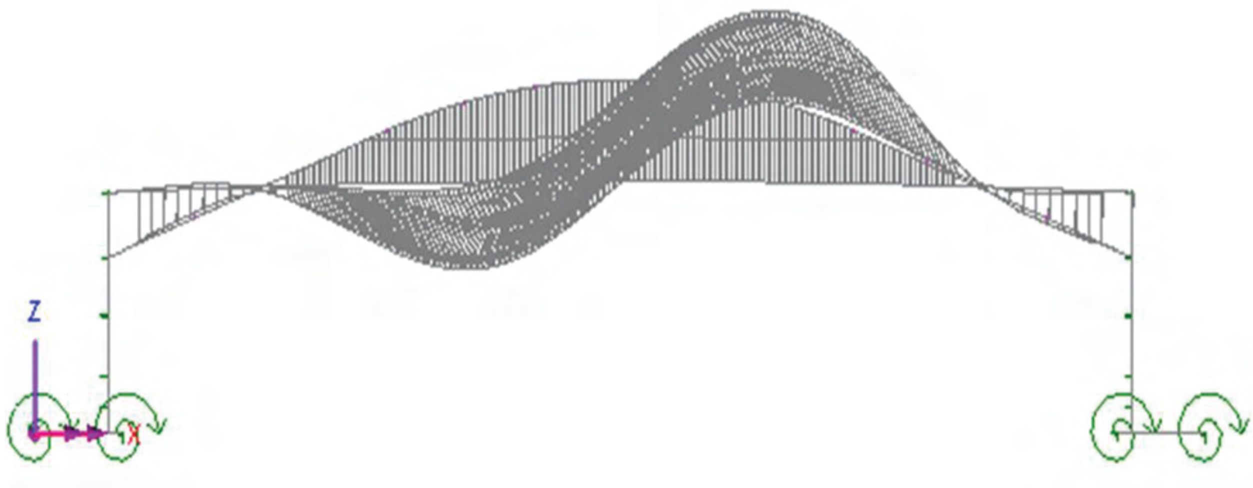

Figure 9 - In-plane buckling mode

The abutment walls were modelled using a grillage of beam members and the forces from the soil pressure were applied to the structural model as an applied load. The piled foundations were modelled using spring elements with an appropriate stiffness.

Global buckling

The arch ribs are necessarily very slender and rely on the vertical slats heavily for its stability. One of the key issues in the design was accurately assessing global buckling effects. The beam element model was used to carry out an elastic critical buckling analysis to determine the susceptibility of the structure to global buckling, both in and out of plane. The lowest of these buckling loads determined from this analysis was then used to determine a slenderness for buckling and codified buckling curves were used to determine a reduction factor to apply to the resistance of the arch and deck.

The structure was susceptible to two main modes of global buckling, in-plane and out-of-plane as shown in figures 9 and 10. The critical mode varied depending on the exact dimensions chosen for the main structural components, the foundation design parameters and the loading applied. Throughout the design iteration process the critical mode of buckling changed as dimensions of different elements were varied and the arch profile was finalised. In the final form of the bridge, the in-plane buckling mode involved the arch and deck and was critical for temperature loading on the structure. The out-of-plane mode just involved the arch and was critical for pedestrian loading effects. The out-of–plane buckling of the arch was resisted by the slats through u-frame restraint. The resistance to in-plane buckling was higher than might be expected for such a slender arch profile.

The greater buckling resistance is provided by the vertical slats which, although providing little stiffness individually, act together at 135 mm centres to provide significant restraint to the arch through their moment connections at each end, forcing the deck and arch to act together.

The British Standards provide little guidance on how to account for the buckling arched structures and so for this structure the approach provided in EN 1993-1-1 Design of Steel Structures Clause 6.3.4 was used. Clause 6.3.4 is written as a general method for checking out of plane (lateral) buckling of members and frames when the axial force and bending moment both give rise to out of plane buckling of the element(s) i.e. the axial force or bending moment applied separately would lead to lateral buckling of the element(s).

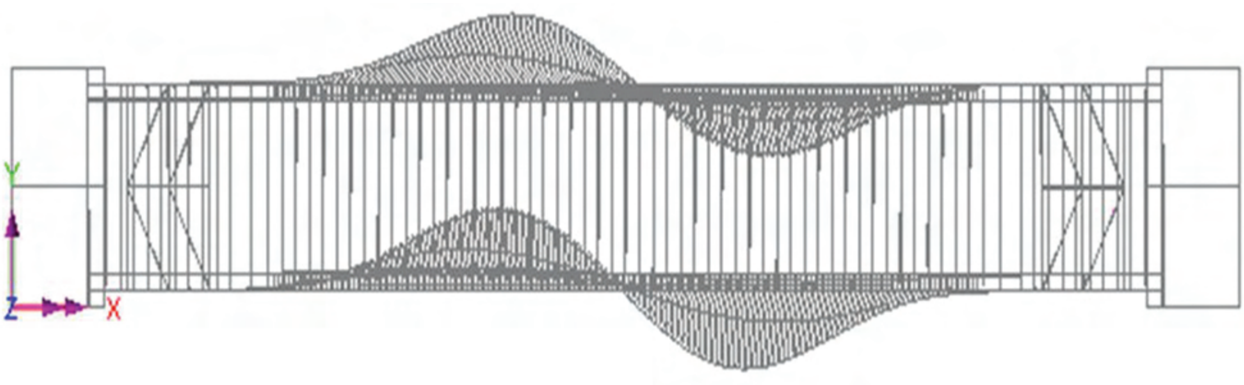

Figure 10 - Out-of-plane buckling mode

In such cases, it is logical that the cross section resistance used in the slenderness calculation be based upon both the axial force and the bending moment together, because both cause lateral buckling of the system i.e.

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}}$$

BS EN 1993-1-1 expression (6.64)

where:

$$\frac{1}{\alpha_{ult,k}} = \frac{N_{Ed}}{N_{Rk}} + \frac{M_{y,Ed}}{M_{y,Rk}}$$

$\alpha_{cr,op}$ is the load amplifier for elastic critical out-of-plane buckling. N_{Ed} and $M_{y,Ed}$ are the applied axial force and bending moment and N_{Rk} and $M_{y,Rk}$ are the characteristic axial force and moment resistances.

It was noted that the assumptions behind clause 6.3.4 were not satisfied for bridge L01 for two reasons. Firstly, only the arch axial force produces lateral buckling, not the bending moment. Secondly, the application of this clause to the buckling of the arch for in-plane buckling is outside the intended scope of the clause. Despite these apparent violations, the method was established as providing a safe approach through comparisons with the results obtained from a full non-linear analysis as discussed below. This was important to establish as the use of elastic analysis to do the design had obvious benefits over non-linear analysis given the number of load cases and iterations of member sizes needed to refine the design and the relative speed of elastic analysis compared to non-linear analysis.

An alternative and apparently more theoretically-sound approach for in-plane buckling would have been to base the slenderness of the arch on the axial force alone and then amplify the coexisting moments in accordance with the verification below. However the distribution of moment within the half wavelength of buckling varied significantly along the arch so the moment magnifier would have been very conservative in this case, and was shown to be so from the results of the non-linear analysis.

$$\frac{N_{Ed}}{\chi N_{pl,Rd}} + \frac{1}{1-(1/\alpha_{cr,ip})}\frac{M_{y,Ed}}{M_{Rd}} \leq 1.0 \qquad (2)$$

$\alpha_{cr,ip}$ is the load amplifier for elastic critical in-plane buckling.

One final point to note on the application of clause 6.3.4 of EN 1993-1-1 is that the UK National Annex limits its application to nominally straight members.

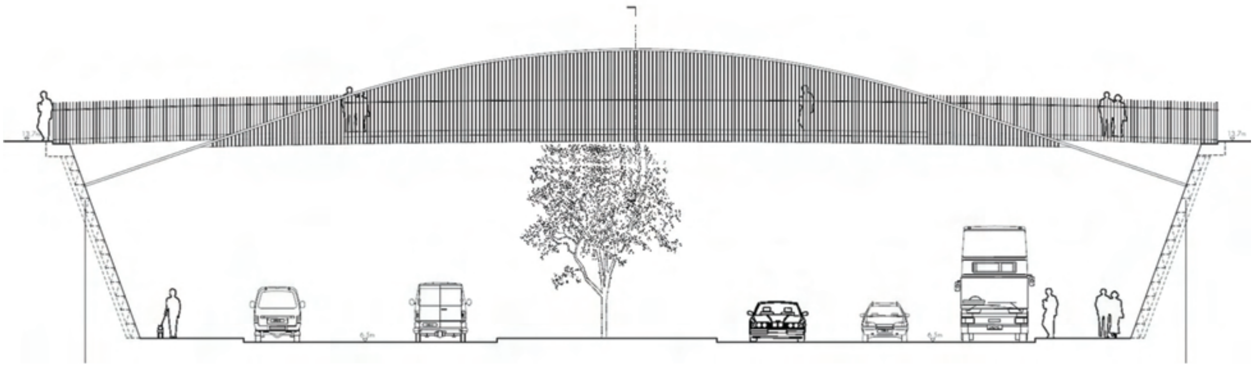

Figure 11 - Original concept for Bridge L01

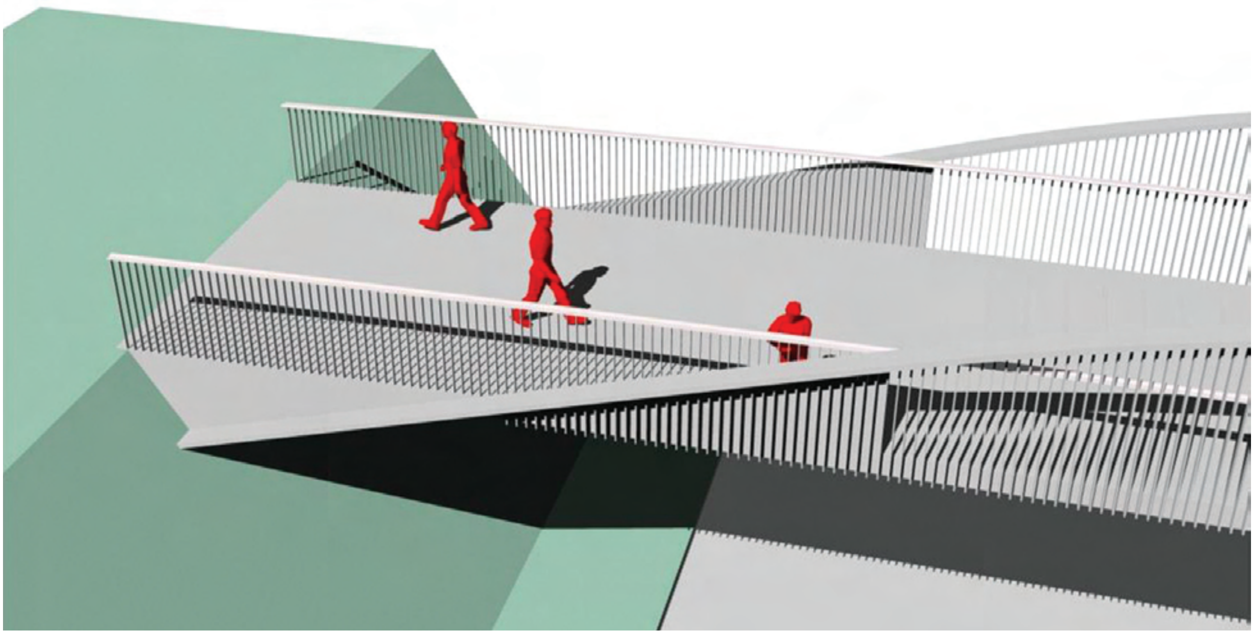

Figure 12 - Arrangement of slats at end of arch

The restriction is considered to be unnecessary by the authors of this paper and was not intended by the Eurocode drafters; moments from initial curvature are included in the calculation of $M_{y,Ed}$ satisfactorily. Indeed, the example of application of the clause prepared by the EN 1993-1-1 Project Team features a curved member[5]. For the purpose of this design, which was to British Standards, a Departure from Standard was obtained for the application of the Eurocode clause, disregarding the restriction in the UK National Annex.

Design of the vertical slats

The elastic critical buckling approach is sufficient for the design of the arch members but the buckling loads obtained rely on restraint provided by the vertical slats and the elastic critical buckling analysis provides no information about the second order effects generated in the slats. As the arch deforms, the slats are bent into double curvature and bending moments are generated even if there was none produced by the initial first order analysis. A method based on magnification of the first order moments (in the absence of modelling initial imperfections) is therefore inappropriate in this case.

In order to establish these secondary bending effects in the slats and their connections, a finite element analysis with geometric nonlinearity was carried out. Conservatively and for comparison with the codified member buckling rules, material nonlinearity was not included and the stress-strain behaviour was restricted to be linear elastic with stresses limited to first yield. Initial imperfections were introduced to the structure in the buckled shapes as shown in Figures 9 and 10. The magnitude of these initial imperfections was set in accordance with the guidance of EN 1993 -1 -1 for equivalent geometric imperfections which include allowances for both construction tolerances and residual stresses in the structure. The results of the nonlinear run were used to establish the increased stresses in the vertical slats in restraining buckling of the arch and to check that the arch buckling resistance predicted by clause 6.3.4 of EN 1993-1-1 was satisfactory.

In the former regard, it was found that the stresses in the slats were typically twice that predicted from first order analysis. In the latter regard, a comparison of the applied loads predicted to cause first yield from the elastic critical buckling analysis and from the non linear analysis showed that they were within 5% of each other, with the nonlinear analysis giving the slightly higher resistance.

Other design considerations

During the design process there were a few aspects of the initial concept that had to be modified to provide structural integrity. The original concept (Figure 11) had the arch unconnected to the deck below deck level. This long free length of slender compression member proved unfeasible and a number of options for restraining it were considered. The final solution was to continue the vertical slats below the deck to restrain the arch. These would be acting in compression and so a solid plate was provided to restrain the vertical plates. Cross bracing between the two arches was also provided below the deck. Analysis of the bridge also showed that in some loading situations the shorter slats above the deck were acting in compression. The buckling resistance of these flat plate elements is extremely low so a solid plate was again provided in this region to restrain the slats as shown in Figure 12.

Conclusion

Bridge H01 and L01 are very different structures, the first being an established form of construction and the second being a highly novel form. However, this paper identifies that existing industry and codified guidance is not always sufficient to ensure that a structure with unusual buckling behaviour is correctly modelled and the behaviour correctly accounted for. This paper serves to illustrate a suitable approach to the design of open top boxes for all designers to use and offers some cautionary notes for designers of unusual structures.

Acknowledgements

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Radio and loudspeaker public address for road tunnels

David Stening

Principal Engineer

Intelligent Transport Systems

Abstract

Tunnel evacuation modelling has shown that the public are generally reluctant to leave the perceived safety of their vehicles. Radio communications in road tunnels for emergency services have been well defined for many years but the ability of an operator to speak directly with tunnel users has traditionally required the use of emergency telephones.

Public address systems that use both radio break-in and loudspeakers have been installed in a number of tunnels in recent years. This paper considers an integrated solution that exceeds the requirements of the EU tunnel directive and provides an efficient response in the event of incidents.

Introduction

Tunnel evacuation modelling[1] has shown that in the event of an emergency, members of the public are generally reluctant to leave the perceived safety of their vehicles until it is too late. The EU tunnel directive[2] recognises this by stating 'Where there is a tunnel control centre, it must be possible to interrupt radio re-broadcasting of channels intended for tunnel users, if available, in order to give emergency messages.' Put simply, if a user has their car radio switched on, an emergency message can be given. Atkins has implemented a number of systems combining both radio and loudspeaker public address, the most recent of which has been installed in the Rotherhithe Tunnel in East London.

Objectives

Whilst the EU directive requires voice break-in for radio broadcast, best practice is to integrate this with a comprehensive loudspeaker public address system. Coupled with emergency signing and mobile 'phone re-broadcast, this provides the facilities required for efficient communication with users.

Speech intelligibility is the key to providing clear messages to tunnel users in an emergency. The purpose of the radio message is primarily to get drivers out of their vehicles and to follow emergency signs. They need to clearly understand the messages and be able to act on them immediately.

It is important that the emergency broadcast is heard simultaneously on all radio channels. It has to alternate with the loudspeaker public address so that the two systems are not heard at the same time as more detailed messages can be put over the loudspeaker system. The loudspeaker system is also zoned so that different messages can be given to users either side of an incident but the radio system cannot easily be zoned.

Car radio public address

In order to encourage tunnel users to keep their car radios switched on, the normal off-air broadcasts are rebroadcast into the tunnel for a number of radio stations that, according to published listening figures, attract the majority of the public. In an urban environment this would typically be 15 VHF FM stations, four MW AM stations, two LW AM stations and five Digital Audio Broadcasting (DAB) multiplexes, each of which carry multiple stations. In the event of an emergency, many car radios will be on and able to receive the emergency message. In addition, the use of the Radio Data System (RDS) on FM channels enables users' tapes and CDs to be interrupted. Each of the DAB multiplexes carries many stations that can vary at any time. Whilst the uptake of DAB has been relatively slow, car manufacturers are beginning to install them in their new models. This will become the norm in due course and may well replace AM or even FM. The UK Government is keen to free up radio airspace, as they are doing with digital television.

The received signal passes through the switch most of the time and is interrupted with the emergency message from the PA system when required. The AM channels are carried out in a similar manner.

VHF FM

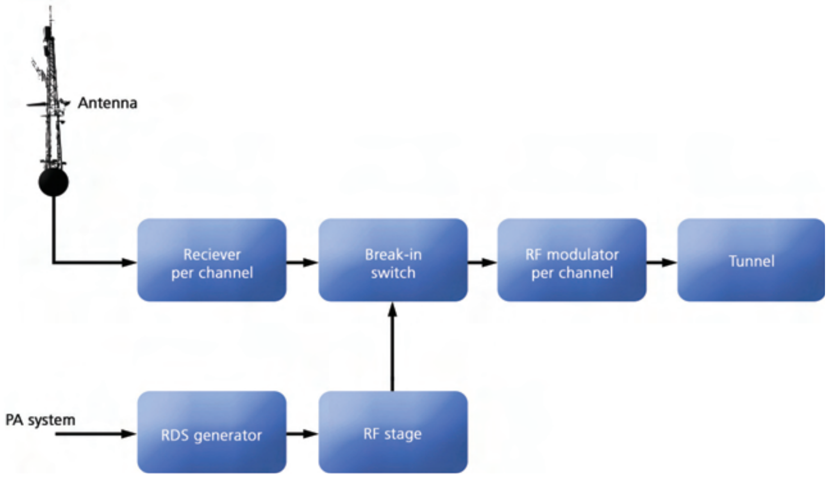

Figure 1 - A simplified block diagram of the radio voice break-in system for FM

Digital Audio Broadcasting

Digital Audio Broadcasting (DAB) is handled quite differently. There are currently up to five multiplexes in any area, each carrying many stations. The digital stream is extracted from each ensemble. The emergency messages are digitised and then inserted into the audio sections of the digital stream. The timing of the interrupt signal has to be synchronised with the broadcast message otherwise the security system built into car radios suspects that the broadcast could be a pirate station and mutes the radio.

DAB is still relatively new (approximately 15 years) and not all car radio manufacturers implement the alarm features available in the same way.

A number of methods of carrying out the DAB voice break-in have been considered and the broadcasters and car radio manufacturers have been consulted.

The first option to consider is to switch off DAB broadcasts and allow car radios to revert back to the FM broadcast. This relies on the software implementation of the DAB standards to search for an alternative station but is not implemented by all manufacturers. Most car radios switch over to FM but this can take up to 30 seconds.

The second option is to 'spoof' the radios to tune to one multiplex or to use 'local windows'. This is more expensive and has similar drawbacks to the first option, so is not worth further consideration.

The third option is to resolve all channels to audio and back again to RF.

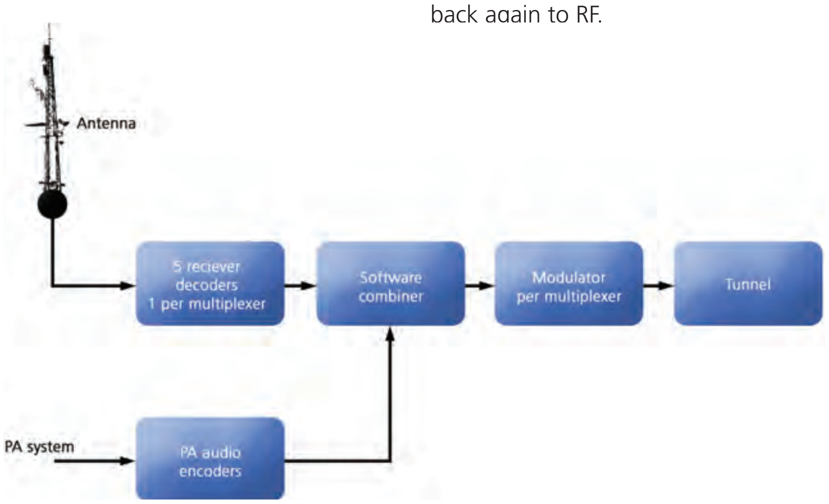

Figure 2 - A simplified block diagram of the radio voice break-in system for DAB.

However, this is discounted due to cost and because of the dynamic nature of channel allocations within each multiplex which would require regular manual updates.

The fourth option to consider is software encoding which inserts a digitised emergency message into each multiplex stream. This is the preferred method and was selected for the Rotherhithe Tunnel which is the first of its type in the world. It does however require a significant amount of computing power to process each multiplex in real time.

Loudspeaker public address

The advantage that loudspeaker public address has over radio is that it can be zoned so that specific information can be given to people in different areas. It also does not rely on drivers having their car radios switched on.

Public address is not common in tunnels mainly due to the high level of ambient noise and reverberation, both of which make speech intelligibility very difficult to achieve. Speaker design and the use of delay lines can minimise these effects. The sound from the nearest speaker is delayed electronically for a few milliseconds until the sound from the previous speaker arrives, ensuring all speakers in a zone are heard at the same time.

Rotherhithe tunnel speech trials

Whilst undertaking the Rotherhithe Tunnel project, site trials were carried out prior to full installation to determine the best speaker configuration and to check speech intelligibility.

The hard tiled walls gave a reverberation time of about eight seconds, which was a significant challenge for the system.

During the trial a number of small speakers similar to those in the Dartford Tunnel were installed at 3.5 metre intervals. Two larger speakers specifically designed for tunnels were set on stands 50 metres apart (Figure 5). The horn shape projects the sound along the tunnel roof. The trial took STI measurements as well as subjective listening tests.

INTELLIGENT TRANSPORT SYSTEMS

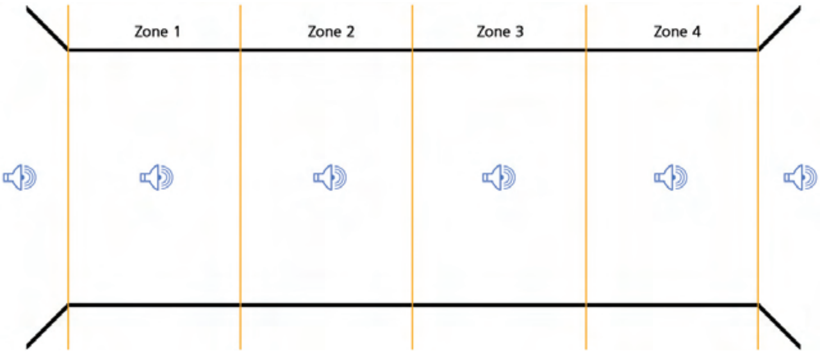

Figure 3 - A simplified layout of four tunnel zones that align with the fire ventilation plans plus one zone on each approach.

Figure 4 - A trial section of speakers in the Rotherhithe Tunnel.

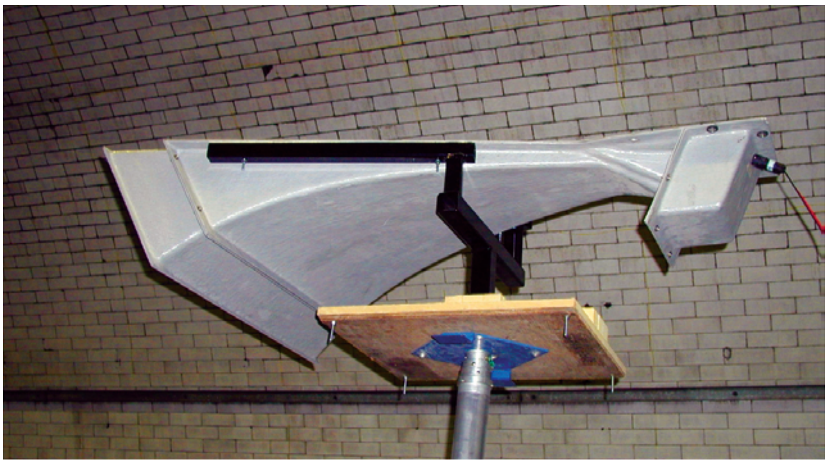

Figure 5 - A purpose built tunnel horn speaker being used in the trial.

While the ambient noise level was low, both types of speakers gave good results but when the fans were turned on and the ambient noise rose to over 87db, the small speakers merely added to the noise and it became very difficult to distinguish the words. Even with the nearest speaker switched off, and at a distance of 100 metres, sound from the horn speakers was still crisp and clear and every word could be heard both in and out of vehicles.

When considering the speaker gain settings, we considered that 6db above ambient noise gave the best all-round results. However automatic gain control is required to adjust the system volume depending on ambient noise levels. Typical ambient noise levels in tunnels are around 87db so an additional 6db gives a system level of approximately 93db.

Integration

It is important that the two parts of a public address system are fully integrated to avoid conflicting messages and to ensure that the radio and speakers to do not operate at the same time. A common user interface should also be provided. The public address system should also be integrated with other Intelligent Transport Systems (ITS). Static and variable message signs need to be co-ordinated with the announcement zones to avoid conflicting messages.

Three types of messages are to be provided, automatic messages triggered after operator confirmation of a fire alarm. These messages are location specific and are intended to direct the tunnel users away from the incident. In addition a selectable menu of messages is available, for example “Stay in vehicle and wait for instructions”, “Leave vehicle and follow emergency signs” or “Please switch off engines - there is a delay ahead”.

Messages use a short delay followed by chimes to attract the user’s attention. Once activated the system does not revert to normal off-air broadcasts until the incident is cleared. Pre-recorded messages need to be clear and professionally recorded. To cater for unexpected situations a microphone is to be provided on the operator’s desk.

Conclusion

The use of purpose designed systems can provide a suitable tool for tunnel evacuation communications in the event of a fire or similar incident. The combination of radio rebroadcast and loudspeaker public address, along with other ITS solutions, enables the operator to communicate effectively with tunnel users, provided they are carefully integrated and user-friendly.

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ITS (UK) Strategy to support carbon reduction and to address climate change issues

Keith McCabe

Principal Consultant

Intelligent Transport Systems

Abstract

ITS United Kingdom is the UK society for all who work in the Intelligent Transport Systems (ITS) sector and comprises the ITS (UK) Carbon Working Group with a remit to promote the use of Intelligent Transport Systems to reduce CO_2 emissions from road transport. Transport is a significant emitter of CO_2, nationally and internationally and therefore ITS can contribute significantly to a variety of initiatives that are aimed at reducing the carbon footprint of UK transport. This paper details a roadmap for the ITS industry to assist and support the transport sector in reducing carbon emissions.

Executive summary

The United Nations Climate Change Conference in December 2009 focused the world's attention on climate change and greenhouse gas reduction. The Copenhagen accord set an objective of limiting global warming to no more than 2ºC.

Transport is a significant emitter of CO_2, nationally and internationally. ITS can be used in a number of ways to reduce the CO_2 emissions from transport. Climate change may result in a significant increase in weather events that disrupt transport networks. ITS can assist with adaptation to the effects of climate change by reducing disruptions to travel as a result of weather events. This paper sets out a strategy on how ITS might be used to address the issues related to climate change and greenhouse gas emissions reduction.

The paper reviews the outcome of the Copenhagen summit. It highlights the key issues from a UK perspective.

The paper then highlights a number of existing opportunities for the industry and ITS (UK) in four main areas:

- Policy support – opportunities that centre on the key elements of the greenhouse gas reduction policy
- Step changes – opportunities that arise from potential step changes in policy (i.e. reducing and enforcing speed limits, road user charging etc)
- Changes to business as usual – how business practices may need to be addressed in a new way to meet the challenge of greenhouse gas reduction
- Related opportunities – other areas that need to be considered as part of a strategy to understand the wider aspects of the climate change challenge.

Climate change and carbon reduction represent a significant challenge for the UK as a whole. The ambitious targets for carbon reduction in the short term (up to 2022) present a significant challenge to the transport sector. For ITS technology and the ITS industry there is a significant opportunity to assist and support the transport sector in reducing carbon emissions. This paper should be seen as a step in the journey to reducing carbon emissions from the transport sector.

Copenhagen summit

The Copenhagen summit was a coming together of nations to formulate an accord to succeed the Kyoto Protocol which expires in 2012. In advance of the summit there were wide-ranging discussions in the media about the significance and potential outcome of the summit.

On 2nd December 2009 The Independent published a climate change special feature entitled 'Twelve days to save the world'[1]. In the special the President of the Maldives, Mohammed Nasheed, was quoted as saying, 'Copenhagen can be one of two things. It can be an historic event where the world unites against carbon pollution in a collective spirit of co-operation and collaboration, or Copenhagen can be a suicide pact. The choice is stark.'

The outcome of the summit was the Copenhagen Accord drafted by Brazil, India, South Africa, China and the United States of America. US President, Barack Obama, was quoted on the BBC on 19th December 2009[2] as saying the Accord had 'agreed to set a mitigation target to limit warming to no more than 2°C and, importantly, to take action to meet this objective'. He added, 'We are confident that we are moving in the direction of a significant accord'. The head of the Chinese delegation, Xie Zhenhua, added, 'the meeting has had a positive result, everyone should be happy'.

Prime Minister Gordon Brown, in his podcast of 21st December 2009[3] about the Copenhagen summit, said ' This weekend the world came together in the first step towards a new alliance to overcome the enormous challenges of climate change'. He also said 'I am convinced that Britain's long term prosperity lies in leading the necessary transformation to a low carbon, greener future. We must become a global leader not just in financing greener technologies but in the development and manufacture of wind, tidal, nuclear and other low-carbon energy. And as we look towards a new decade, be assured that your government will play its part in supporting the ambitions of our entrepreneurs and leading businesses and the expertise of our scientists and engineers in making this transformation'.

The verdict on the summit, it seems, is that it was neither a world uniting against carbon pollution nor a suicide pact. Instead it was perceived as a starting point to overcome the challenge of climate change. The key is to find and produce technological solutions to reduce carbon emissions. The opportunity for the ITS industry is to find the appropriate solutions to meet the climate change challenges of the new decade and beyond.

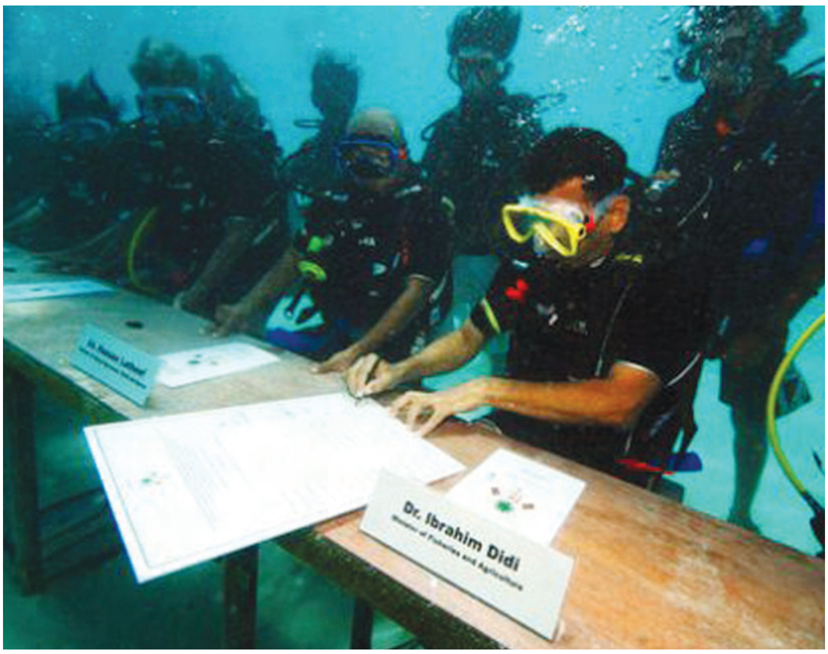


Figure 1 - Maldives underwater cabinet meeting

Transport and climate change

In October 2009 the committee on climate change produced its progress report to Parliament entitled 'Meeting carbon budgets – a need for a step change'[4]. The report describes the challenges that need to be addressed to meet the carbon budgets for the UK which have been set for three periods from 2008 to 2022 as shown in Table 1. Traded emissions are emissions traded on the EU emission trading scheme (ETS).

The report identified a number of measures and targets to reduce emissions from transport and outlines extended scenarios to increase the level of reduction. The main measures identified for the UK were:

- Reducing emissions per km of internal combustion engine vehicles from the current 158 gCO_2/km to $95gCO_2/km$
- Increase use of electric vehicles and plug in hybrid vehicles. The take up rate of electric vehicles and plug in hybrids is expected to be 240,000 by 2015 and 1.7 million by 2020
- Eco driving - aim to have 3.9 million drivers trained and practicing eco driving by 2020.

The report identified a number of areas where policy might be strengthened to facilitate a step change in emissions reduction (extended scenarios):

- Support for electric cars and plug-in hybrids, including the funding of plug-in infrastructure and large scale pilot projects
- Smarter choices. Phased roll-out of smarter choices (based on the methods established in the pilot projects) across the UK to encourage better journey planning and more use of public transport
- Integrated land use and transport planning. A new strategy to ensure that land use planning decisions fully reflect the implications for transport emissions.

	Bugdet 1	Bugdet 2	Bugdet 3
	2008-2012	2013-2017	2018-2022
Carbon Budgets ($MtCO_2e$)	3018	2782	2544
Percentage reduction below 1990 levels	22%	28%	34%
Traded sector ($MtCO_2e$)	1233	1078	985
Non-traded sector ($MtCO_2e$)	1785	1704	1559

Table 1 - Legislated Carbon Budgets

The report also identified a number of measures that could be implemented with changes in policy (stretch scenarios):

- Travel demand management measures including a form of road pricing to manage travel demand
- Enforcing and reducing speed limits.

The committee on climate change outlined the key issues in relation to reducing transport emissions in the UK. The opportunity for ITS industry is to determine how ITS can be used to support the emission reduction strategies. The next section outlines the issues in relation to these opportunities.

Key opportunities for the ITS Industry

ITS technology can be used to reduce transport emissions. There is a need to be able to demonstrate the ability of the existing ITS technology to reduce emissions and to develop new solutions in relation to some of the new opportunities that arise from the challenge of climate change.

Policy support

These are areas where ITS may be used to support key areas of carbon emissions reduction policy.

Electric vehicles

The key policy for reducing emissions is reducing emissions per km from vehicles. In the short term this is being undertaken by improving the efficiency of internal combustion engines and increasing the proportion of electric and plug-in hybrid vehicles. The ITS industry needs to investigate how this policy can be supported by ITS to facilitate the take up of lower emissions vehicles.

Eco driving

Eco driving involves driving the vehicle in a different manner that reduces the energy required to propel it. This involves less volatile acceleration and softer braking. Technology could be used to encourage particular driving styles using in-vehicle and infrastructure-to-vehicle systems. There are also other ways of improving fuel efficiency of driving namely: changes to the driving test, variable message sign messages, use of motorway speed signs etc.

Supporting policy step changes

The Committee on Climate Change assessed the impact of existing climate change policies and put forward a number of scenarios that could be implemented if there was a significant step change in climate change policy and travel behaviour. The opportunities for the ITS industry are to identify how ITS technology could facilitate or encourage these step changes.

Electric vehicle roadside infrastructure

The current aim is to have 1.7m electric and plug-in hybrid vehicles on the road by 2020. This may require an extensive network of roadside charging infrastructure. The ITS industry has a lot of experience in delivering roadside powered infrastructure. The opportunity could be to make a roadside charging point part of a network of intelligent infrastructure. This may include the booking of points and provision of information on availability, which could include the booking and payment of 'hot swap' batteries

Smarter choices / journey planning

The Committee on Climate Change expects there to be a high impact on emissions due to smarter choices. Sustainable Travel Towns have provided some evidence[5] for the potential of smarter choices through the use of car clubs, travel plans, travel awareness campaigns and car sharing. The opportunity here is to determine how ITS might support such initiatives.

Integrated land use and transport planning

The Committee on Climate Change estimates that the implementation of an appropriate land use and transport planning framework could result in a reduction in emissions of at least 2 $MtCO_2$ by 2020. Information on walking, cycling and public transport can play a key role in this area. The opportunity here is to determine how ITS can play a role in supporting these initiatives.

Travel demand management

The Committee on Climate Change identified a number of pricing measures that could assist in the reduction of travel demand. These include increasing fuel duty and road user charging. ITS techniques have been used to introduce road user charging in various locations around the world so the opportunity here is to identify how this might be undertaken in the UK and its potential impacts.

Enforcing traffic regulations

The Committee on Climate Change identified the enforcement of traffic regulations and in particular speed limits as a possible method of reducing emissions. Historically this has been undertaken at specific locations for safety reasons. The opportunity here is to understand how speed limits, and more broadly traffic regulations to support carbon reduction objectives, may be enforced and what sort of impact this might have on emissions reduction and complementary measures such as eco-driving. It could also include the enforcement of 'low emissions zones'.

Opportunities arising from changes to business as usual

ITS are currently in widespread use in transport applications in the UK. The reasons for the deployment of ITS historically have mainly been safety (reduction or avoidance of accidents) or economy related (reduction of congestion, improving journey times and reliability). The opportunity now is to understand what impact existing and new deployments of ITS have on carbon emissions. This section looks at three areas where this is likely to be an important issue.

Transportation of goods

There is increasing emphasis on reducing emissions resulting from moving goods around the UK. There is scope for ITS technologies to be utilised in order to reduce emissions from the freight sector. The areas of particular concern are the last five miles of deliveries, issues around out of town hubs and the use of the railways for carrying goods. Technology can play a role in providing information and statistics on the movement of goods as well as providing information in real time on the status of networks.

Energy consumption of equipment

One of the key emissions reduction policies is to reduce the carbon intensity of the energy supply network. Figure 2 outlines the current expectation for demand and carbon intensity of the energy supply network.

It should be noted however, that the major shift in carbon intensity does not start until mid-2020. Consequently, as ITS mainly use electricity to operate, there is a need to investigate how the industry's energy use could be reduced in the short term. Reducing energy consumption in the short term not only reduces CO_2 emissions but also reduces costs.

Network management

Network management is becoming an increasingly important technique with networks running at capacity for longer periods. UTMC and Managed Motorways are two techniques that are increasingly used in the urban and interurban environments. Normally the focus of these techniques is either economic (congestion reduction) or safety (accident prevention and reduction), however, there is a need to formulate techniques that are tailored for emissions reduction.

Issues related to climate change and greenhouse gas reduction

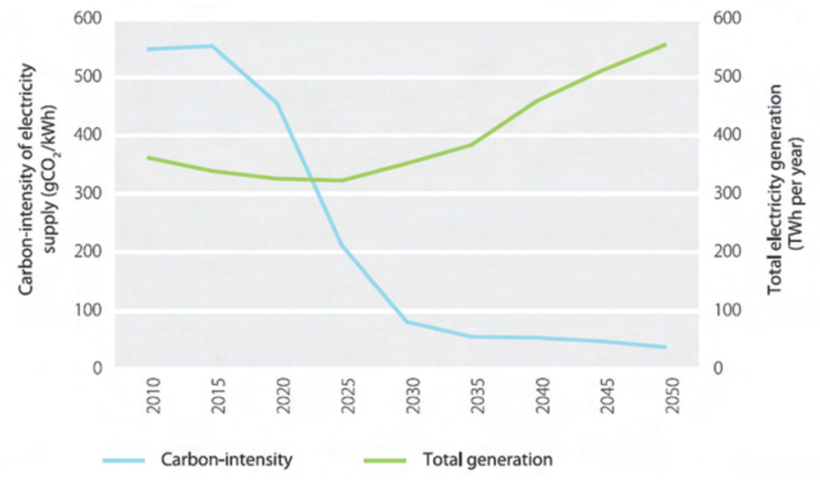
The reduction of greenhouse gas emissions is the main focus of this strategy, however, this cannot be viewed in isolation from other issues. In this section some of those other issues are highlighted.

Social inclusion and health

Addressing issues of emissions reduction is often seen as complementary to addressing issues of social inclusion and health. The opportunity here is to understand where the measures to address the problems are complementary and where they are not. The Department for Transport's Delivering a Sustainable Transport System[6] (DaSTS) guidelines take into account economic growth, social inclusion, health, quality of life and climate change as criteria for assessment. This wide-ranging method of assessing transport measures may become more widespread in the future which is why it is important to understand the relationships between the criteria and the impacts that ITS has on each element. This is becoming increasingly relevant in the justification for a new scheme.

Economic growth

The recent recession has put the need for economic growth very high on the agenda. The opportunity here is to understand the relationship between economic growth and emissions reduction. The low carbon economy is seen as a way of encouraging economic growth; the opportunity here is to understand what this means for the ITS Industry and the deployment of ITS measures. This area is key to understanding the opportunities surrounding the ITS (UK) theme for 2010 of 'Lean and Green ITS'.



Source: CCC based on AEA (2008) *MARKAL-MED model runs of long-term carbon reduction targets in the UK*

Figure 2 - Reducing the carbon intensity of energy generation

Climate Change Adaptation

IPCC reports and climate change forecasts, produced by the UK Met Office, highlight that a certain degree of climate change is inevitable. The Copenhagen Accord aspires to limit temperature increases to 2°C. In this context it is key to understand what impacts changes in climate and weather conditions might have on the UK. Managing change in real time when networks are stressed or disrupted is one of the key strengths of ITS. The opportunity here is to understand how ITS might be applied to the area of climate change adaptation.

Carbon tools and climate change information

Greenhouse gas emissions and in particular CO_2 emissions have to be estimated, based on particular activities that are undertaken. The tools used for that estimation need to be consistent and independently verifiable. The setting of baselines and targets for particular activities is also key to understanding aspirations and measuring achievement. There are a number of different tools available or in development that address these issues. The opportunity here is for the ITS industry to take a view on what tools and monitoring or measurement techniques are suitable for determining the impacts of ITS. The understanding of climate change and its impacts is an ongoing area of research. Within the industry and in the wider public arena there are different, competing views. The opportunity is to provide useful information to assist in developing policies and strategies for the future.

Conclusions

Climate change and carbon reduction represent a significant challenge for the UK as a whole. The ambitious targets for carbon reduction in the short term (up to 2022) present a significant challenge to the transport sector. There is a significant opportunity for ITS technology and the ITS industry to assist and support the transport sector in reducing carbon emissions. This paper should be seen as a step in the journey to reducing carbon emissions from the transport sector.

For more information on the ITS (UK) Carbon Working Group contact the chair of the Working Group Keith McCabe. Keith.mccabe@atkinsglobal.com

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Interfacing electrification and system reliability

Roger White

Professional Head of Electrification and Plant

Rail

Abstract

Integration is the term given to ensuring that the different elements of an electrified railway operate together to achieve the desired result. This is particularly problematic where interfaces exist between the high power electrification distribution or traction systems, and the low power signalling and communications control systems.

The challenge of integrating electrical systems will not be made any easier as client and public expectations of new rail transit systems require railway designers to incorporate more versatile and sophisticated electrification systems.

Railway authorities in the past have found difficulty in implementing a management strategy that will ensure electrified railway systems are integrated. This is made more complex where the privatisation of railways and the procurement process divides contracts into separate engineering disciplines.

This paper seeks to explain the challenge of integrating the electrification system with the sensitive signalling, communication control and low voltage systems. Additionally the paper will review the technical necessities of integrating the electrification system with large civil structures. The paper will demonstrate how railway authorities are being challenged by the difficult task of integrating these system-wide disciplines.

Interfaces between electrical systems

The electrical systems integration process for an electrified railway should ensure that traction power, control systems, traction and rolling stock and radio based systems are integrated successfully. Specific engineering interfaces exist between rolling stock to electrification; rolling stock to signalling; electrification to signalling and communications; civil to LV and HV electrification and stray current to third parties.

Interface of earth exists between all electrical systems. This paper will review in more detail earthing of AC and DC railways, AC/DC interfaces, lightning and stray current corrosion.

Earthing of AC and DC railways [4,10,13]

Electrical transportation systems (UK) by law are required to meet specific safety requirements:

- Health and Safety at Work Act 1974.
- Electricity at Work Regulations 1989.
- Railways (Safety Critical Work) Regulations 1994
- Construction (Health, Safety and Welfare) Regulations 1996.
- Management of Health and Safety at Work Regulations 1999.
- Control of Substances Hazardous to Health (COSHH).
- Transport and Works Act, 1992
- The Railways Act 2005
- EMC Regulations 2005
- The Railways and Other Guided Transport Systems (Safety) Regulations 2006 ROGS
- Construction (Design and Management) Regulations 2007.

Responsibility for regulation of safety in 2006 moved from the Health and Safety Executive to the Office of the Rail Regulator.

Earthing design objectives: The earthing system, its components and bonding conductors should be capable of distributing and discharging the fault current without exceeding thermal and mechanical design limits based on back-up protection operating time.

The earthing system should maintain its integrity for the expected lifetime of the installation with due allowance for corrosion and mechanical constraints.

Earthing system performance should avoid damage to equipment due to excessive potential rise, potential differences within the earthing system and excessive currents flowing in auxiliary paths not intended for carrying parts of the fault current.

The earthing system, in combination with appropriate measures, should maintain step, touch and transferred potentials within the voltage limits based on normal operating time of protection relays and breakers.

The earthing system performance should contribute to ensuring electromagnetic compatibility (EMC) among electrical and electronic apparatus of the low-voltage system in accordance with BS IEC 61000-5-2.

Design approach to the earthing of the railway: To ensure compliance with relevant standards there is a requirement for a robust Quality Assurance process including an Earthing Management Plan, Earthing Installation Code of Practice and Earthing Test Specification.

Additionally the Earthing Management Plan must address the design, installation, operation, and maintenance aspects of the railway.

AC/DC railway interfaces [14]

An AC railway is often designed in close proximity to other DC light rail, metro and trams. Where there is parallel running, over bridges or under bridges, there may be significant interactions between the two electrified railway systems. The amount and characteristics of disturbance will be dependent on the interconnection of earths, traction loads, earth faults, the physical arrangement, the electrical systems design and the local ground conditions.

Track circuit disturbance: DC traction return current flowing in the AC traction rails can interfere with track circuits. Similarly the AC return current can interfere with track circuits used on DC electrified lines.

DC stray current: The bonds between the rails and metal structures etc. which are necessary for electrical safety in the presence of high-voltage AC railway overhead lines will provide a path for stray DC traction return current to flow to earth, and may cause electrolytic corrosion to the structures concerned, as well as neighbouring buried pipes, metal-sheathed cables etc.

Touch and accessible voltage: The impedance of the rails at 50 Hz is much greater than the resistance of the rails at DC. This means that the return current from AC trains, and from short-circuits on the AC overhead lines, would cause significant voltage drops in the rails. This would bring a risk of electric shock if suitable means were not employed to limit the voltage on the rails.

Lightning protection [11,12]

Lightning strikes on or near railway equipment can generate large voltage surges that can disturb or damage railway operations. The various ways in which a lightning strike can affect railways are:

(i) Direct strike to the overhead lines

(ii) Direct strike to the aerial earth wire or gantries

(iii) Nearby strike to ground [induced voltages]

(iv) Strike to ground further away [ground potentials].

A direct strike to the lines can generate an overvoltage surge of several million volts. This will cause a flashover across the support insulators. The surge current will then find various routes to earth depending on their surge impedance values. A lightning strike may also generate a 25kV earth fault.

Entry points for lightning to the railway control systems are numerous and some are detailed below:

(i) Radio masts

(ii) Electrical supply points

(iii) Rail and track connections to signalling circuits

(iv) Signalling data, control and indications cables

(v) Ground coupling through circulating earth loops,

(vi) Induction into power, signalling and communications circuits.

Design approach on railways: The protection of buildings, viaducts, bridges and structures from lightning is by the provision of lightning conductors and an earth electrode system. This earth is segregated from the earth path used by signalling control and the electrical and plant (E&P) systems.

In the case of the station roof structure and viaduct there is a requirement to provide lightning earthing, to control earth potentials.

Additionally there is a requirement to ensure the insulation co-ordination and any necessary segregation required for sensitive signalling and control systems that are located on or close to the station or viaduct.

Surge arrestors: Gas plasma discharge arrestors have a fast response to transient overvoltage events. This fast response and the ability to handle very high current surges effectively suppresses transients. Low capacitance and high insulation resistance [>Gohm] produce a low leakage ensuring that there is virtually no effect on the protected system.

Installation of surge arrestors: Surge arrestors can be installed on telecommunications cables or on the installation equipment. Surge arrestors for commercial installations are rated at typically 250V. This is normally too low for the railway environment.

The surge arrestor operates when a voltage between the line and remote earth exceeds the level specified. This method diverts the transient energy away from the line circuit and into the earth of the network. This is good as long as the earthed system is not coupled in any way with the telecommunications cables. The design of the earth should be such that there is no close parallel coupling with telecommunications cables.

Stray current corrosion control (DC railways) [5,7]

The overall control strategy is to follow an approach that minimises the generation of stray current, controls its 'escape' and maintains separation between the rails and associated collection systems and the metro civil structures and external structures.

The driving force for stray current can be minimised by:

- Traction power substation design and locations;
- Rail return circuit design and bonding; and minimising the longitudinal electrical conductivity of supporting structures
- Maintenance of a high rail to structure insulation.
- Construction and maintenance of stray current collection systems to provide an efficient low resistance preferential path for current collection and return.
- Maintaining an electrical separation between the collection system and other conductive parts of the structure.

Design approach on DC railways: This strategy is designed to reduce corrosion threats to the rail infrastructure and to external infrastructure by minimising stray current leakage at source and retaining this as far as practicable within the rail system.

The design process should:

- Assess the interfaces between the railway and other third party infrastructure through a Stray Current Control Survey to identify and assess stray current and corrosion risk areas. (E&M Earths, Civil Structures and Utilities)
- Utilises design philosophy that follows the guidance given in BSEN 50122-2, BSEN50162 and UK Trams ORR Tramway Guidance Notes
- Undertake modelling techniques to quantify these threats and validate the control measures
- Provide verification processes that should be designed and applied at each stage of the project to provide assurance that the strategy has been correctly applied and to identify issues for investigation and action
- Ensure a consistent approach and awareness both across the project and between the below ground and above ground sections.

Compatibility of electrical systems [3,6,8,9]

The EEC directive 89/336 made the requirement that all electrical apparatus must not emit electromagnetic radiation that would prevent other equipment from functioning as intended.

The European EMC Directive 89/336/EEC as amended by 92131/EEC has been legally adopted by UK regulations under Statutory Instrument (1992) No. 2372 'The EMC Regulations' and came into force in 28 October 1992. The Directive applies to virtually all electrical and electronic products and systems for use in the European Union.

The new EMC directive 2004/108/EC[2] includes some simplification and clarification when compared to the original Directive:

(1) Manufacturers are required to perform an EMC assessment; including the application of harmonised standards

(2) Manufacturers are required to produce a Technical File [TF] to demonstrate conformity

(3) Manufacturers may choose to obtain an independent conformity statement from a Notified Body.

(4) Manufacturers are required to make a Declaration of Conformity [DoC]

(5) A Fixed Installation [FI] at a pre-defined location is required to conform to the essential requirement but not to follow the conformity assessment procedure and therefore does not have to carry the CE Mark

(6) The FI will usually consist of equipment carrying the CE marking installed as specified by the manufacturer(s) and specific equipment not otherwise commercially available can be incorporated, accompanied by documentation which indicates precautions to be taken for incorporation into the FI; the installation shall follow ‘good engineering practice’ which must be documented.

The railway environment: contains many sources of electromagnetic, electrostatic noise and electrical disturbances; and it is a hostile environment for low power circuits i.e. remote control systems, monitoring circuits, signalling systems, telecommunication circuits and radio networks. The complete railway electrification network requires to be monitored in terms of emissions and susceptibility to electromagnetic interference, conductive interference and radio frequency interference. This includes electrical circuits in the supply of traction power, the control of traction power, the operation of signalling, train control circuits, the operation of telecommunication systems and radio communications.

Electromagnetic interference will largely cause interference from the high power circuits of the electrification supply and traction drive into the train control systems, signalling systems and railway and publicly owned telecommunication systems, remote control, and monitoring systems.

The railway authority’s responsibilities [15]

The railway authority has a responsibility to ensure that projects include design specifications that address interfaces between electrical systems and also the management process, to ensure that systems do not adversely interact or disturb each other.

Application of railway and national standards

BS EN standards are used extensively in the specification of railway projects. The projects assume that if the standards are implemented then their design will always be adequate. The basic standard for earthing railways, EN 50122-1, is very clear on safety requirements for earthing and equipotential bonding. However, the standard is not a practical guide to earthing a railway. It is a performance specification and details what is required to be undertaken by railway administration. It does not provide any detail on how installation should be undertaken.

It should also be noted that the European Standard EN50122-1 does not specifically address

(1) earthing arrangements for lightning protection

(2) provide detail design and interconnection of earthed systems

(3) system operability and the design differences between rail return, auto feeding, booster return, single rail and double rail return systems.

The complications of railway earthing are significant and the specific requirements for traction power, signalling, communications, LV and lightning earthing all have to be addressed by the project, with potentially different requirements for at grade, in tunnel and on viaduct sections. It should be noted that the IEE Wiring Regulations BS7671 is pertinent to stations, concourses and other buildings; however, track, platforms, railway traction equipment, rolling stock and signalling equipment are excluded.

It is normal practice therefore for a railway administration to produce an accompanying installation guideline for contractors and maintainers. Without such a code it is inevitable that earthing disturbances will occur. Examples of Company Code of Practice are; Network Rail NR/SP/ELP/21085, Channel Tunnel Rail Link 000-GDS-LCEEN-10041-05; Indian Railways AC Traction Manual Appendix 11 code for bonding and earthing for 25kV AC 50Hz single phase traction system.

Risk (commercial, operational, safety)

It has become the norm for railway contracts to be based along traditional engineering disciplines and interfaces; for example, wheel to rail and pantograph interfaces. Railway authorities sometimes fail to recognise the complexity of electrical interfaces and provide a solution to manage the integration between different engineering disciplines.

The major cost element of a new railway is the civils discipline; the electrical systems are a relatively small portion when compared to the civil infrastructure costs. However, failure to integrate the electrical systems will probably mean disruption to the whole project during commissioning and delay early operational service.

Major Rail Projects are largely run by the Civil Engineering Discipline. Rail Authorities often fail to provide adequate project management effort to integrate the systems, placing the whole project programme at risk.

Specify design interface specification

The railway administration should be aware of the requirements for integration, and it is their responsibility to identify and include these criteria within the project requirements at the definition stage. Interface requirements must then be included within the Particular Design Specifications for each contract. They should also make all System Wide Contractors responsible for identifying interfaces, hazards and for participating in the integration methodology of the project.

If they fail to include this within the particular specification, then the contractors will not be obliged to ensure that this integration happens. The outcome of this can be disastrous for the project programme and the railway administration, which may be powerless to ensure that their multi-million pound investment operates as intended.

Management of electrical systems

The railway authority should specify the requirement for Management Plans to address quality assurance process and installation Codes of Practice. This is to ensure compliance with railway and national electrical and safety standards. This normally becomes the responsibility of the design and build contractor and has been addressed in more detail later in this article.

Design management responsibilities

The preliminary and detailed design stages of project management plans should be prepared to address the above system interfaces. The following are examples of management plans that should be incorporated within the overall systems engineering management plan.

This is normally the responsibility of the Design and Build Contractors.

Safety Management System [15]

The Project Safety Management System (SMS) is part of the Quality Assurance process that is required to address safety of humans. In an electrified railway the topics that need to be addressed include touch and accessible potential and the risks associated with the malfunction of signalling, and control systems leading to multiple fatalities.

All potential safety hazards associated with the operation of the railway including the earthed infrastructure of the railway network should be identified. The hazards associated with electrification supply points, overhead lines, traction return paths, operation of traction and rolling stock, effects in depots and station stops etc need to be addressed.

To achieve this, experts in the fields of electrification, earthing, signalling and rolling stock are required to be brought together to assess the system behaviour under normal train operation, under system faults and all degraded modes of operation of the electrification and signalling systems and the rolling stock.

Design interface specifications

Where there are known interfaces these should be recorded in an interface matrix and a specification prepared between the two disciplines or contractors. Such interfaces could also include the co-location of several different railways at an interchange station.

An example of this is London, where various lines on the London Underground DC metro railway are in close proximity to the 25 kV AC main line infrastructure. The interface specification should identify all of the systems that may be affected.

It is necessary that the responsibilities for the identified hazards in the project are assigned an owner (and accepted) and that the designer then mitigates any potential disturbance. It should also be remembered that utilities and neighbours must be considered when addressing interface specifications within the railway environment.

Integration Management Plan [6]

The Electrical System Integration Management Plan should set out a strategy to ensure correct operation of equipment for the project life-cycle to ensure that systems are integrated successfully.

The following disciplines and systems should be included: traction power and electrification (both rail authority and public supply), signalling and control, telecommunications, rail vehicles, permanent-way and civils infrastructure, and third parties.

Failure to integrate these electrical systems properly will introduce the probability of an electrical or control system failure. This could then have a subsequent disturbance to the operation of the railway.

Earthing Management Plan [4]

The content and detail should be based on previous experience of earthing other AC/DC railways and the performance requirements of relevant national and international standards i.e. European Standard EN50122-1, and British Standards BS7671 and BS7430.

It should set out a strategy which ensures the management of interconnection of earths, including: railway electrification and distribution systems, the distribution network operator, the low voltage (LV) distribution earths, signalling earths, lightning conduction and other inter-system earths.

The requirements of utility companies' earthing practices and their connection to the railway need to be recognised and integrated.

An Earthing Management Plan should include a 'process' to cover requirements at design, installation and operational stages of the railway.

Electromagnetic Compatibility Management Plan [3,6]

An EMC Management Plan is required and should be based on the requirements of EN50121, EN61000-5 and EN61000-6. It should set out a strategy for the project to ensure EMC is achieved. It should identify a quality assurance process including hazard identification, required deliverables, roles and responsibilities, EMC certification and test specifications.

The product standards provide emission and immunity tests under controlled environments. There are additional requirements for large infrastructures that require Installation Codes of Practice; this is often bespoke to the infrastructures railway systems being designed. These Installation Codes of Practice are often ignored by individual contractors who do not consider it there responsibility.

These Codes of Practice should be based on the requirements of the railway network, best practice in EMC design, installation and the EN61000-5 series of standards. In designing a railway that operates close to another railway, it is necessary that reference is made to local railway standards and codes of practice of both railways, as their emissions will impinge on each other.

Preparation of an EMC test specification should include EMC testing of individual apparatus and at the system level. Additionally site measurements should be undertaken of radiated electric and magnetic fields of the final installed railway system along its route.

DC Stray Current Management Plan [5,7]

The 'stray current management plan' should provide guidance on the management framework, quality processes and deliverables that are required through the design and installation stages of a project. Specifically this plan should detail acceptable pass/fail criteria for the level of stray current, after construction is completed.

This plan should detail deliverables including the 'stray current code of practice', test plans and monitoring plans. The Code of Practice should provide detail of the design, based on known proven technologies and evidence of best practice e.g. attention to electrical substation connection, high rail insulation, good stray current collection, good drainage construction supervision and quality control by qualified personnel, and the necessary testing and monitoring system.

Concluding remarks

New railway projects and the privatisation of railways have divided contracts and the design process into separate engineering disciplines. Railway authorities need to recognise the importance of integrating the electrical systems. Failure to do so will have a subsequent consequence on delivering a railway free from electrical disturbances.

This paper has endeavoured to demonstrate how railway authorities are being challenged by this difficult task of integrating electrical systems. This challenge will not be made any easier as client and public expectations of new rail transit systems require the railway designers to incorporate more versatile and sophisticated electrified railway and tram networks.

Acknowledgements

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The author and Atkins Rail accept no liability to anyone for any loss or damage caused by any error or omission in the work, whether such error or omission is the result of negligence or any other cause.

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Investigations into electromagnetic noise coupled from lighting to safety related communications equipment in an operational metro

Leslie McCormack
Principal Engineer
Rail

Abstract

High frequency ballasted lighting presents a well known electromagnetic interference threat. Switching frequencies from these types of lighting control-gear are typically 40kHz - 120kHz producing significant interference up to 10's of MHz. During a large scale modernisation of urban metro rail stations in London, UK, it was necessary to implement an unusual combined lighting and communications cable management system which co-located lighting and safety related communications systems due to heritage planning restrictions at a particular station. There were Electro-Magnetic Compatibility concerns expressed by the railway operator about the proximity of the fluorescent lamps and high frequency ballasts to the communications assets. The station was to remain open throughout the site works and so assurance was required that the proposed design would function satisfactorily once installed, as the cost implications of unsatisfactory functionality and associated delays, were prohibitive. This paper is based on previously published work[1] and details laboratory and site measurements which were performed to investigate the effects of co-locating these systems and demonstrate that they would function satisfactorily.

Introduction

Fluorescent lighting presents a well known electromagnetic interference (EMI) threat to nearby vulnerable equipment. For older lighting equipment this threat was at power frequency harmonics and was generated from the wire wound ballasts that were commonly employed to control the current flowing in the fluorescent tube. More recently however, the lighting industry has developed high frequency ballasted lights which have improved lighting performance and consume less power. Switching frequencies from these types of lighting control gear can be in the region 40kHz – 120kHz. The high frequency ballast behaves essentially like a switched mode power supply and can produce significant interference up to 10's of MHz[2]. High frequency ballasted lighting therefore presents a very different threat than that presented by earlier fluorescent lighting.

During a major modernisation program of urban metro stations in London, UK, heritage and space constraints at a particular location resulted in an unusual combined lighting and communications system Cable Management System (CMS) being proposed by the designer.

The CMS placed high frequency ballasted fluorescent lighting in close proximity to the station safety related communications system, upon which the station relied for operation. The CMS design was extruded aluminium with space for power cables to either side of the light fittings and space for communications cables immediately above the light fittings. Fluorescent lamps with high frequency ballasts were fitted in alternate compartments with Public Address (PA) speakers filling the gaps. The railway is a complex electromagnetic environment and Electro-Magnetic Compatibility (EMC) is a significant area of concern in many railway projects[3]. EMC concerns had been expressed about the proximity of the fluorescent lamps and high frequency ballasts to the communications cables which would be much closer than the 130mm separation required by the designer's own cable separation guidelines, EN 50174-2[4] & industry guidelines[5]. The station was to remain open throughout the site works and so assurance was required that the proposed design would function satisfactorily once installed, as the cost implications of unsatisfactory functionality and associated delays, were prohibitive.

Therefore preliminary laboratory EMC measurements were performed on a specially constructed short 10m representative measurement jig to assess the likelihood of interference to the communications system. The quality of the final installation in controlling the coupled interference was later confirmed through site verification measurements.

Measurements

In order to provide indicative information on the likely coupling of disturbances into the communications system cabling, a measurement jig was constructed from representative CMS and cabling. Measurements were then made of the longitudinal and transverse noise voltages (VL & VT) present on the victim circuit formed from typical shielded twisted pair PA cable. The Root Mean Square (RMS) psophometric noise voltage (VP) was calculated by applying a psophometric filter to the VT data. Following site installation, site validation measurements were performed.

Longitudinal and transverse induced noise voltage

Longitudinal voltage is a term often used in telecommunications engineering and refers to a common-mode voltage which is induced along the length of a transmission circuit. Excessive VL can be an electrocution hazard to maintenance staff and can affect the operability of equipment. The term transverse voltage refers to a differential-mode voltage appearing between the pairs of a transmission circuit. Excessive transverse voltage can lead to reliability problems for affected systems. In addition excessive transverse voltage in the audio band can result in performance degradation for audio circuits. Applying psophometric weighting to the transverse voltage data provides an indication of the level of degradation that the noise voltage may cause to the intended signal, as perceived by a human listener.

The measurements were performed over the frequency range 5Hz – 30MHz and were intentionally similar in nature to those already performed at other stations on the network[6] and in associated laboratory investigations[7]. For the shielded twisted pair (STP) victim cable the shield was left disconnected at both ends as this was thought to represent worst case site installation. For VL measurements the 2 legs of the twisted pair (TP) were connected together and tied to a local earth at the far (field equipment) end. At the near (measurement equipment) end the 2 legs of the TP were connected together and VL was measured between the combined lines and earth. The measurement analyser & transducer were earthed through a strap to the ground plane. For VT measurements, the shield of the STP was again left disconnected at both ends. The 2 legs of the TP were terminated at the far end by a 600Ω resistor. At the near end VT was measured between the 2 legs of the TP. London Underground Manual of EMC Best Practice G-222[8] limits VL to a maximum of 25V at 50Hz and the RMS VP to be 1mV. EN 61000-4-16[9] which is referenced by G-222, gives expected immunity levels of equipment to longitudinal voltages above 50Hz. The measured data was compared against these limits.

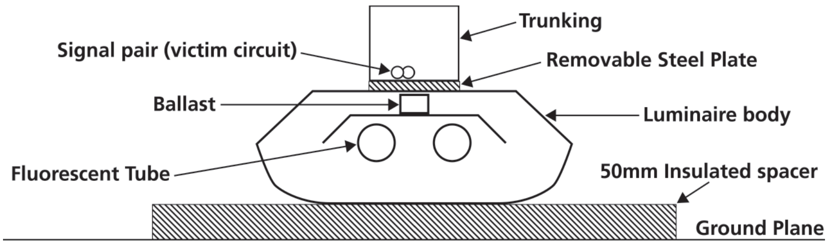

Figure 1 - Section view of CMS measurement jig setup

Laboratory measurement jig

The laboratory measurement jig consisted of a short 10 m section of representative CMS with fluorescent lamps, ballasts and PA speakers installed in adjacent compartments (Figure 1) as follows:

- 4 off 1800mm luminaire sections each containing twin 1500mm fluorescent lamps pre-wired with 3 core 1.5mm^2 flex to the connector of each fitting
- 4 off 800 mm Infill sections which contained 4 off PA rectangular speakers affixed with brackets centrally to the circular aperture of approximately 120mm diameter
- 4 off 2600mm CMS approximately 110mm x 75mm in dimensions
- (DRAHA UK) FIRETUF OHLS BASEC LPCB BS7629 Part 1 BS6387 CAT C, W, Z 300/500V H2 x 1.0mm^2 IEC 332 Part 3G shielded 2 core communications cabling for wiring speakers
- 4 off steel plates of approximately 1800mm x 100mm x 2mm in size.

Power was supplied to the lights from a local 230V supply. The measurement jig was assembled in a manner typical of site installation (Figure 2). During the measurements steel plates were added as a remedial measure to reduce the coupling into the communications cables as the original design had no provision for a solid floor in the communications cable trunking. Measurements were made both with and without the steel plates fitted. The paint finish on the extruded CMS was removed to allow for a good low impedance contact between the CMS and the steel plates.

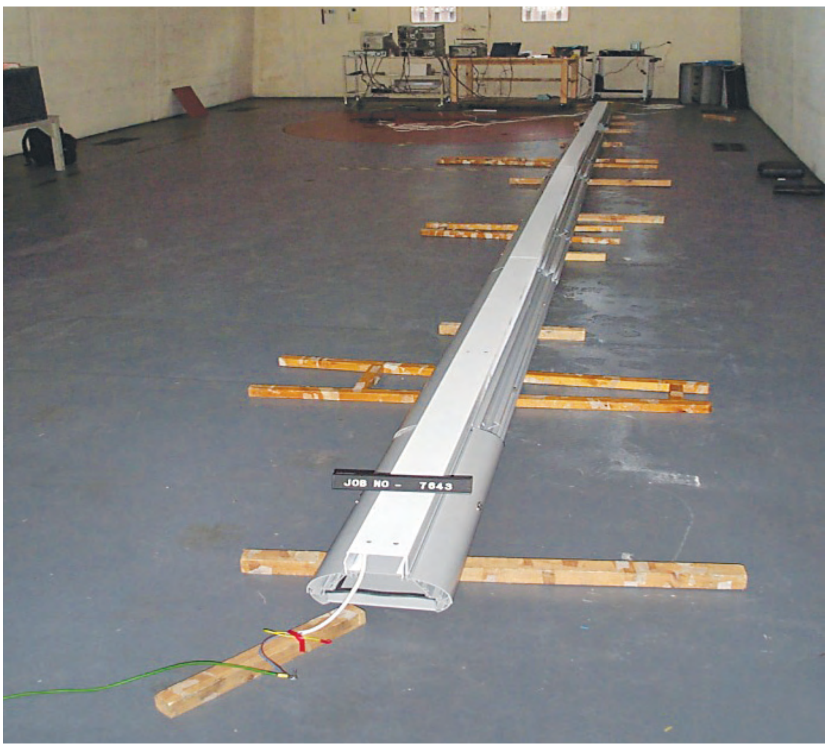

Figure 2 - Photograph of CMS measurement setup

Measurement results

Figure 3 shows an example of the VL measurement ambient in the frequency range 9kHz to 30MHz. Figure 4 shows the effect of energising the CMS luminaires. Emissions due to the luminaires are visible in the region 300kHz to 6 MHz. The limit derived from[9] is 3V (129.54 dBμV) under normal railway operating conditions and the measurement data showed that the coupled VL was below this limit.

Figure 5 shows typical onsite VP data over the frequency range 5Hz to 6kHz. The limit given in[8] was an RMS value of 1mV (60dBμV) over the frequency range 5Hz to 6kHz.

The CMS deployed onsite was expected to extend for approximately 100m and so it was assumed that actual VL & VP at the station would be higher than the laboratory measurement results. This proved to be the case when site measurements were made, however, both VL & VP measured onsite were significantly below the required limits.

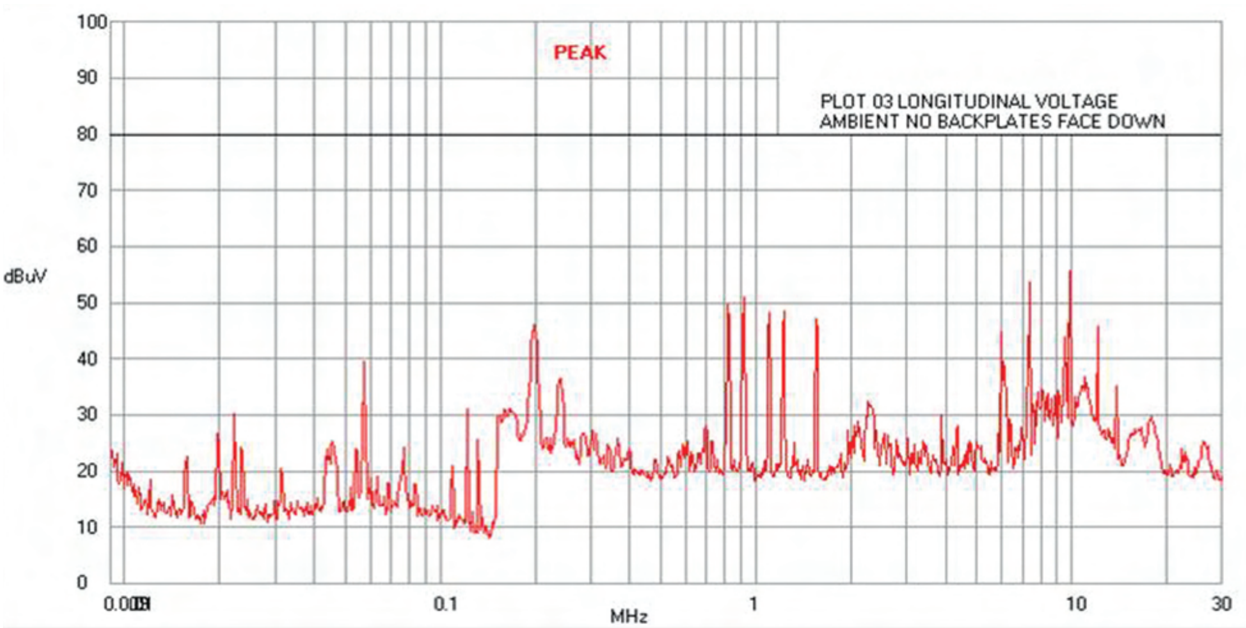

Figure 3 - Longitudinal Voltage 9kHz to 30MHz, Ambient, No Steel Plates

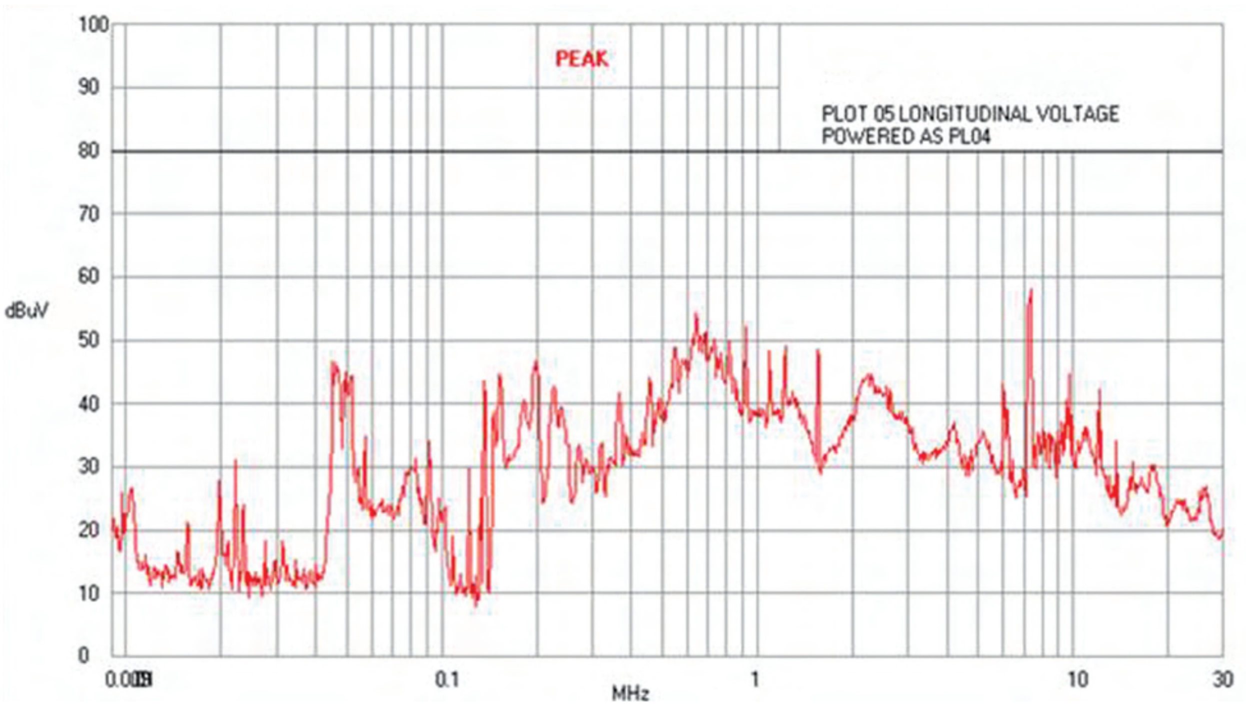

Figure 4 - Laboratory Longitudinal Voltage 9kHz to 30MHz, Lights Energised, (Without Steel Plates)

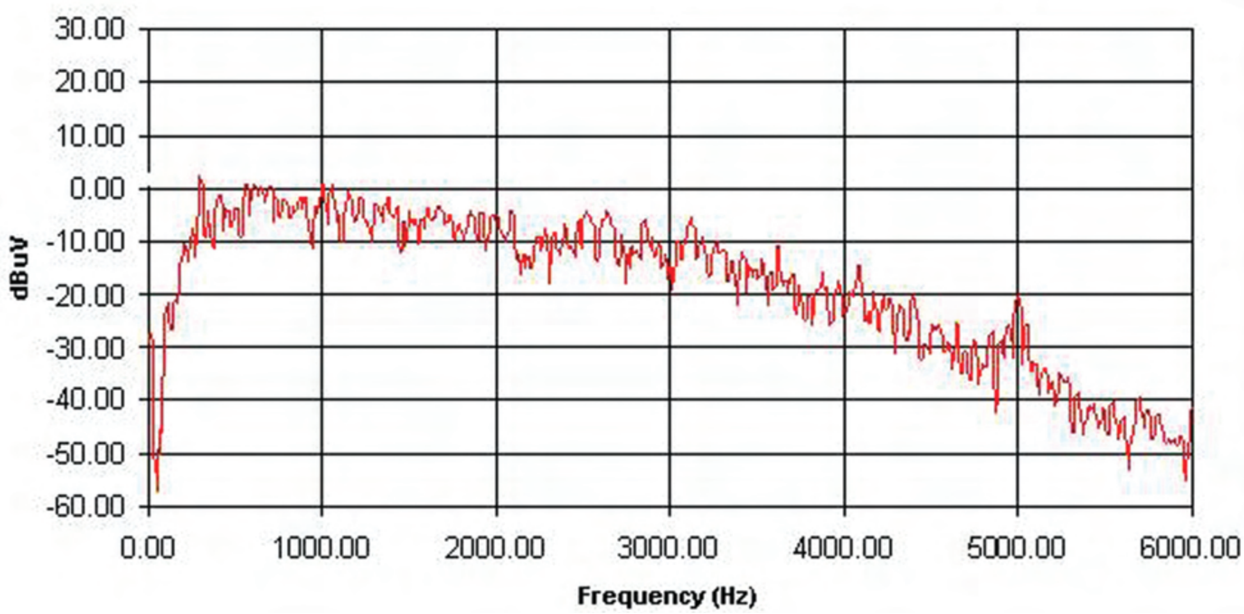

Figure 5 - Onsite VP, 5Hz to 6kHz, Lights Energised

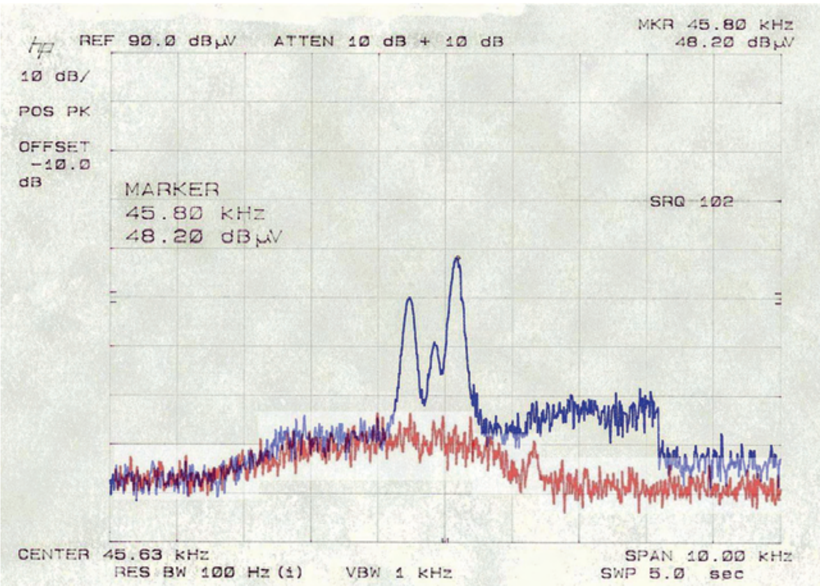

Figure 6 - VL Ballast Emissions in the 45kHz region (blue) & Ambient (red) with Steel Plates

Frequency (kHz)	No Plates	With Plates	Difference
	Level (dBµV)	Level (dBµV)	Level (dB)
1.973	38.2	27.1	10.1
409	32.5	30	2.5
455.5	41.5	36.5	5
500.5	35.7	33.7	2
546	45.9	42	3.9
637.5	48.5	45.9	2.6

Table 1 - Effect of the Steel Plates in the Laboratory VL Measurements

The effect of the steel plates in the laboratory measurements

Figure 6 shows the effect of energising the luminaries in the measurement jig. The emissions profile due to the switching frequency of the Tridonic ECG electronic lighting ballast is clearly visible. Removing the steel plates increased the levels at ~45kHz by 5.3dB.

Table 1 summarises the VL laboratory results with and without steel plates added. It shows that in the 2kHz to 640kHz region the effect of removing the steel plates from the CMS increased coupling by between 2 to 10dB. For VT the effect of removing the steel plates from the CMS increased coupling by up to 10dB. For Psophometric Voltage (VP) removing the plates increased the RMS VP value by 1.57dB.

Other considerations

The originally proposed CMS design did not provide adequate containment for the communications cables. The communications trunking had a steel roof and sides but no solid floor. The laboratory measurements demonstrated that well bonded steel trunking, which surrounded the communications cables, did provide attenuation at the frequencies of interest. Following the laboratory measurements the installer proposed a modification to the design which involved adding a well bonded steel plate between the communications trunking and the luminaires. This was a result of the effect of adding steel plates to the CMS (thus providing shielding to the communications cables above) being examined during the laboratory measurements. At the relatively high ballast frequency the dominant coupling mechanism was radiated coupling, rather than inductive cable related (per unit length) coupling. It was thought that the interference from the fluorescent lights was unlikely to be in phase and so would not significantly sum linearly with the number of luminaires deployed. This was confirmed during site measurements. It was reported that at a particular installation in Finland interference from luminaires became a problem over time as the luminaires aged[10]. However, the railway infrastructure maintainer responsible for the station had a policy to replace all fluorescent lamps on an annual basis, regardless of condition. This maintenance policy should provide protection from similar aging problems occurring at this site in the future.

Conclusions and acknowledgments

The laboratory measurements showed that the likely levels of VL and VP onsite would be below the limits and therefore provided the necessary confidence to progress with the installation & commissioning. The site measurements validated this and functional testing further confirmed the satisfactory performance of the installed communications system.

This paper reflects the views of the author alone. The author would like to thank colleagues within Atkins who have supported this work.

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Value Engineering on the Church Village Bypass

David Osborn

Major Projects Manager

Highways & Transportation

Abstract

The Church Village Bypass Contract was awarded to Costain in June 2007 with Atkins as lead designer. The first three phases involved developing an initial target cost based on the tender scheme (Stage 1) and if required carrying out Value Engineering (Stage 2) to develop a scheme that the funding authority would deem affordable and good value for money. Between August 2007 and February 2008, the integrated team developed a scheme with an estimated total out-turn cost of under £90M - some £35m less than the scheme cost at the end of Stage 1.

Introduction

The A473 forms an important section of the Welsh Strategic Highway Network running west to east across the South Wales valleys between Bridgend and Pontypridd (see Figure 1). To the east of Llantrisant, in the vicinity of Llantwit Fardre, Church Village and Ton-Teg, the existing route passes through a mainly built up area with direct frontage access by residential and commercial properties, and considerable pedestrian activity.

The existing roads are generally in poor condition, and this section of the A473 is unsuited to modern needs with substandard alignments, numerous junctions and accesses, road works and on street parking hindering the flow of traffic. Therefore a better road network was considered essential to relieve the area of the excessive traffic congestion currently experienced.

Figure 1 - Regional map

Scheme objectives

The aims of the scheme are:

(i) Relief of congestion on the existing road network

(ii) Environmental improvements to the settlements of Llantwit Fardre, Church Village, Ton-Teg and Efail Isaf. As a direct consequence of the reduction in traffic, anticipated benefits to these settlements are improved air quality, a reduction in noise and vibration, improved vehicular and pedestrian access and safer streets. In addition, the scheme incorporates a number of new community routes which will open up areas of countryside for walking and cycling

(iii) Improved road safety - the reduction in traffic travelling through the communities will lead to a reduction in accidents

(iv) Improved conditions for public transport

(v) Provision of improved access to the trunk road network and hence major employment centres for this substantial area of population.

History of scheme

A major bypass for the villages of Llantwit Fardre, Church Village and Ton-Teg was identified by the former Mid Glamorgan County Council in the early 1970s. The development of the scheme continued sporadically until the mid 1990s when the new Unitary Authority of Rhondda Cynon Taf (RCT) was formed following local government reorganisation. RCT developed the scheme with funding via a Welsh Assembly Government Transport Grant. The scheme was submitted for planning in 2004 and following comments from stakeholders and statutory consultees, further amendments to the scheme were made and a revised planning application consented in 2006. The necessary side Roads Order and Compulsory Purchase Orders were confirmed in 2007 following a public inquiry.

Tender and contract

The scheme was put out to tender in early 2007 as an Early Contractor Involvement contract with the first three stages as development phases under an NEC3 Professional Services Contract. Stage 1 involved pricing of the original scheme, Stage 2 was the Value Engineering phase (if required) and Stage 3 involved developing the design sufficiently to ensure the Employer's requirements were met and to accurately develop a Target Cost. The next two stages were to be a Construction Contract for Design and Build of any necessary advance/ enabling work (Stage 4) and the main construction works (Stage 5). Costain was appointed as Contractor in June 2007 with Atkins as lead designers and RPS as environmental designers. Capita Glamorgan acts as the Employer's Agent with support from Chandler KBS.

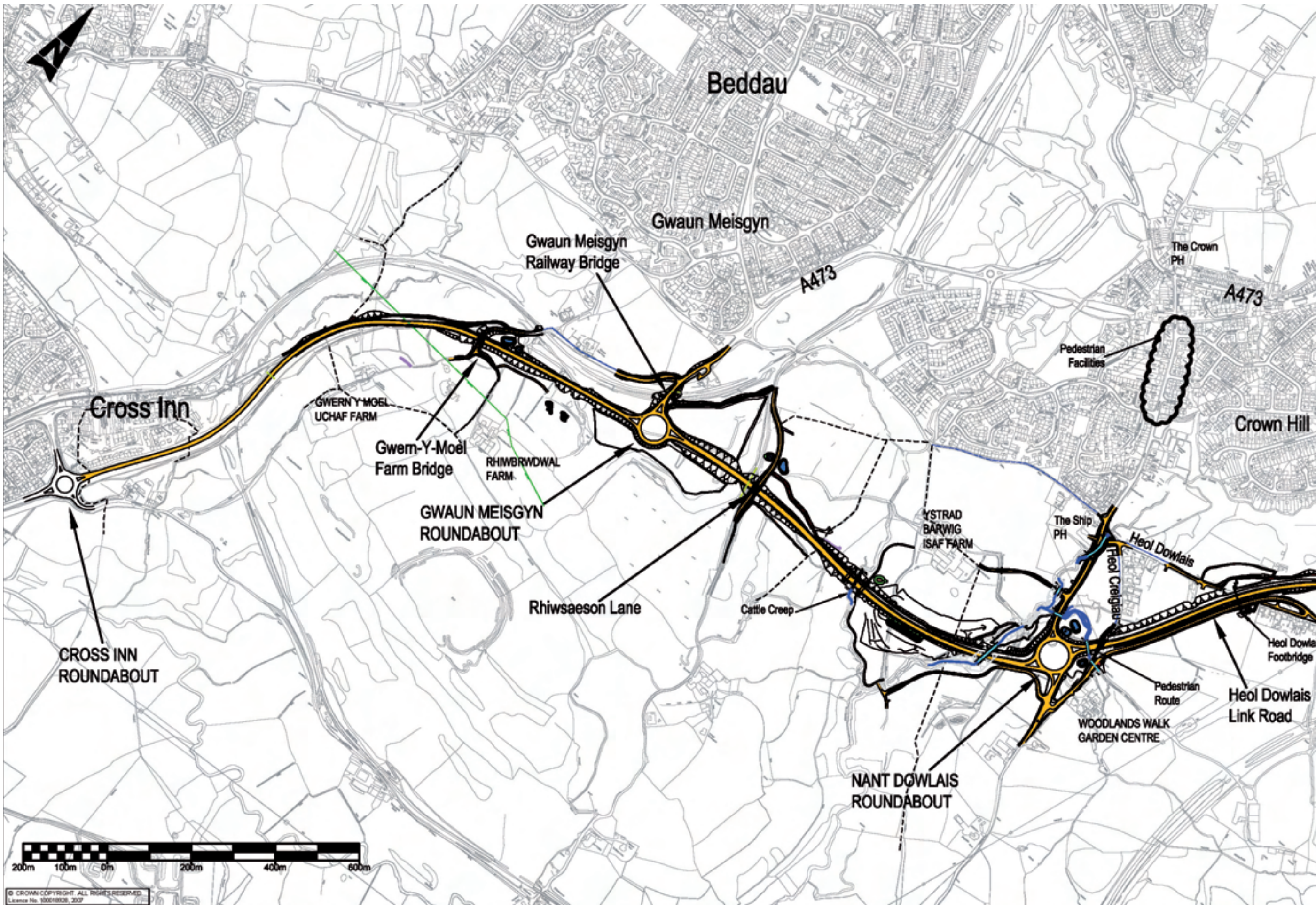


Figure 2 - Scheme plan

Original scheme

The original 'tendered' Church Village Bypass consisted of 7km of dual carriageway and included a community route for pedestrians and cyclists, nine road bridges (including two green bridges), two accommodation bridges, one cattle creep, two community route footbridges, two community route underpasses, 11 major culverts, 800m of retaining walls and 3km of minor road construction. There were three new at-grade roundabouts and a major grade separated interchange at Ton-Teg.

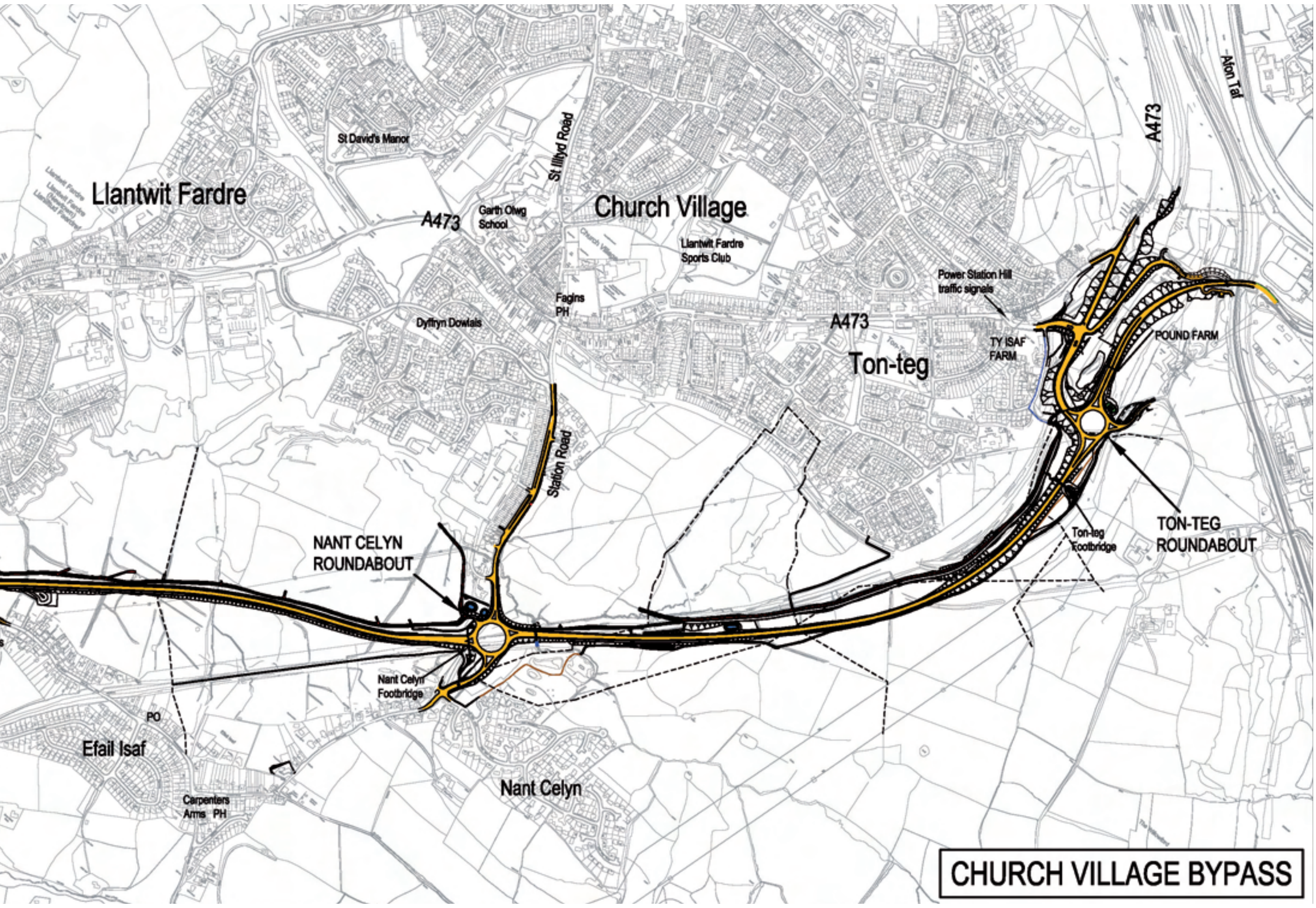
Value engineering

In accordance with the procurement strategy agreed with the Welsh Assembly Government, a value engineering exercise was carried out by a Client / Contractor / Designer integrated team. This exercise was undertaken to ensure that the project delivered a scheme which provided best value while meeting the stated primary objective of significantly reducing traffic flows along the existing A473 road corridor.

The VE exercise was carried out by five teams drawn from all members of the integrated team, each concentrating on an aspect of the scheme. The aspects considered were Alignment/ Earthworks, Structures, Junctions/ Side Roads, Statutory Undertakers, and Roadworks / Drainage. All proposals were assessed for cost and engineering and environmental impact before any necessary consultation was carried out.

A peer review of the VE proposals was carried out by at team of experienced engineers from across Costain and Atkins which gave a reality check to the proposals and generated further ideas for consideration.

The layout of the final scheme is shown in Figure 2. The two most significant changes were the redesign of the Ton-Teg interchange as an at-grade roundabout and the change in carriageway standard from D2 to S2.



Ton-Teg Interchange

The original grade separated layout (Figure 3) had been adopted to address the traffic split at the eastern end of the bypass between the destinations of Treforest/Pontypridd and A470/Treforest Industrial Estate as well as the significant level difference between the bypass and the existing road network. The revised layout was achieved by amending the alignment of the western approach from the A470/Industrial Estate (Power Station Hill) and linking the two arms of the existing A473 via a traffic signal controlled junction. The alterations to the Ton-Teg interchange (Figure 4) resulted in the deletion of two road bridges over the bypass, a 100m length viaduct and an accommodation bridge as well as over 400m of retaining walls. It also reduced the depth of the cutting through a PFA/tyre tip thereby reducing the excavated volumes of both materials.

Carriageway standard

Following a series of Value Engineering workshops it was agreed to review the junction strategy and carriageway standard of the bypass from first principles against the primary objective of 'relieving congestion to the existing A473'.

The traffic modelling programme SATURN was run with the omission from the scheme of the Gwaun Meisgyn, Nant Dowlais and Nant Celyn roundabouts in turn. Omission of any of the three roundabouts was discounted on the basis of the significant flows that would remain on parts of the existing A473 and the existing side roads.

Guidance in TD 46/97 on economic flow ranges recommended a D2AP based on the predicted traffic flows in the opening year. SATURN was then run with various options of carriageway standard. The options tested were wide single carriageway (WS2), single carriageway (S2) and S2 with dualled bypass approaches to the roundabouts (S2 modified). The modelling of the S2 (modified) option was discussed with TRL before being carried out.

Figure 3 - Ton-Teg Interchange - Original layout

Figure 4 - Ton-Teg Interchange - Final layout

HIGHWAYS

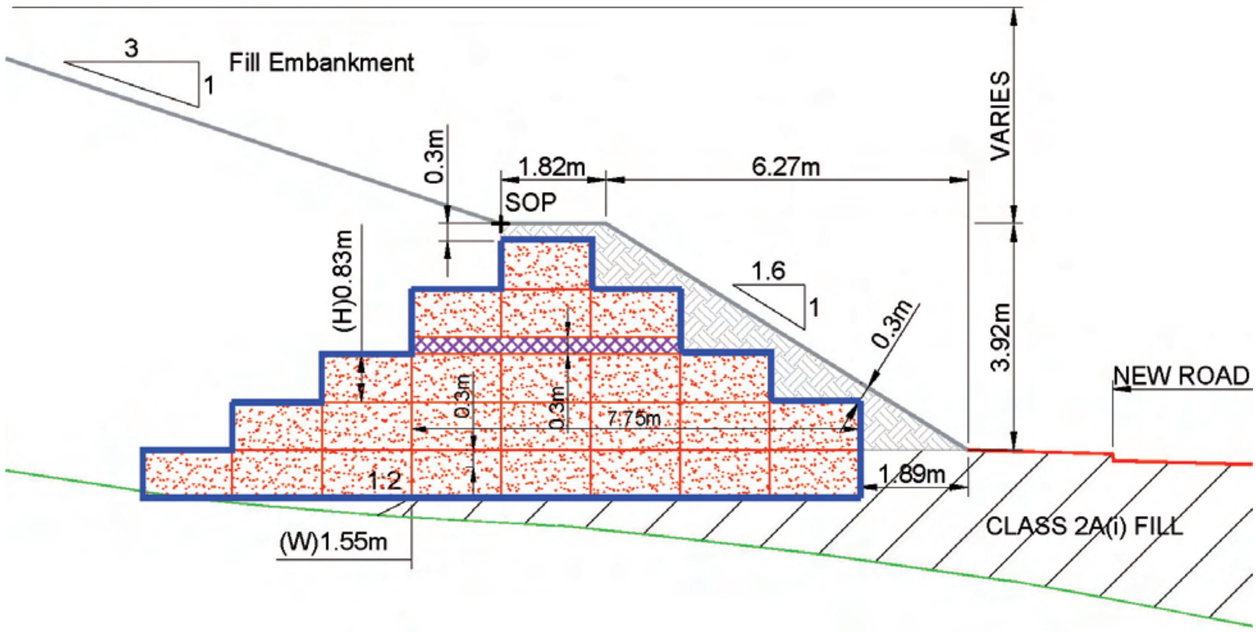


Figure 5 - Typical section of tyre bale wall

The initial consideration in testing these options was the reduction in traffic using the bypass compared to the D2AP scenario. It was deemed unacceptable by the team if the traffic flows on the bypass fell by more than 10% as this traffic would then use the existing A473. The S2 option failed to meet this criterion as the capacity of the roundabouts led to traffic using the existing A473. The WS2 and S2 (modified) options both met this criterion within the SATURN model and the capacity of the roundabouts was then tested using the ARCADY software and all roundabouts were found to have adequate capacity in the design year (2025).

Both WS2 and the modified S2 were assessed by the team against their cost, economic benefits, environmental effects and operational safety compared to the D2 option. Following this assessment it was decided that the scheme would go forward as an S2 with dualled approaches to the roundabouts. As an example of the effect of the bypass, the opening year flow on the bypass between the Nant Dowlais and Nant Celyn roundabouts is forecast to be 22,500 AADT. The flow on the existing A473 over this section is forecast to be 7,100 AADT compared to the 'do-nothing' flow of 21,600.

Major VE proposals adopted

Other significant VE benefits resulting from the five VE groups included:

(i) Alignment and earthworks

- The vertical alignment was redesigned to optimise the earthworks balance taking into account the acceptability of material as well as the quantity
- The volume of excavation in the two municipal tips in or adjacent to the site was reduced in one case and avoided in the other
- No material required disposal off site including 50,000m^3 of pulverised fuel ash and 500,000 old tyres. The tyres were baled on site and used to form a gravity retaining wall to support a landscape area (Figure 5)
- Use of earth bunds to reduce noise and visual effect of the bypass
- Treatment of unacceptable material to enable its use in construction e.g. mixing peat with topsoil to form a growing medium for landscape areas.

(ii) Structures

- Reduction in number of structures required by alignment changes and land agreements
- Use of pre-cast arches (Figure 6) to improve quality control, reduce construction time and reduce site hazards
- Use of corrugated steel culverts in lieu of in situ concrete, again to improve quality control and reduce construction time
- Deletion of two green bridges and replacement with three dormice bridges. The green bridges consisted of a standard overbridge carrying a local road but doubled in width to accommodate a trough containing topsoil and a hedgerow to give continuity of habitat across the bypass (specifically for dormice). The final structures are significantly lighter and have the added advantage of giving instant connectivity to the existing habitat rather than wait five to ten years for new vegetation to establish itself. The revised structures were favoured by the Countryside Council for Wales (CCW) who had to issue the relevant licenses and saved an estimated £0.75M over the green bridges.

Figure 6 - Gwern-y-Moel Bridge (Bebo Arch)

(iii) Junctions / side roads

- Alternative access arrangements for farmers resulting in deletion of accommodation bridges
- Diversion of an existing side road to avoid a skew overbridge.

(iv) Statutory undertakers

- Amending the scheme to avoid the need for diversions (including the £1M diversion of a high pressure gas main)
- Altering the phasing of the works to avoid expensive temporary diversions.

(v) Roadworks and drainage

- Use of precast manholes to reduce construction time and avoid snagging
- Use of envirokerbs at junctions and in urban areas in lieu of kerbs and gullies
- Extensive use of swales as part of a SUDS design.

Final scheme

The final scheme consists of 6km of single carriageway and includes a community route, two road bridges, two cattle creeps, three community route footbridges, one community route/equestrian underpass, three dormice bridges, seven major culverts, 10m of retaining walls and 1.5km of minor road construction. There are now four at-grade roundabouts.

The predicted reduction in traffic along the A473, from around 21,000 vehicles to 8,000 AADT (Annual Average Daily Traffic Flow) in the 2010 opening year, will lead to an improved environment, improved road and pedestrian safety, reduced noise and better air quality for residents and businesses located along the A473 corridor.

The reduced bypass footprint of the proposed scheme has also resulted in a reduction in the environmental impact along the route corridor. It has also allowed the inclusion of earth bunding along a significant length of the scheme to screen the road from surrounding properties and reduce the effect of traffic noise. Less land will be required from landowners affected by the proposals and there is a reduced impact on ecologically sensitive areas and the landscape.

The changes to the scheme required a new Planning Application and a new Environmental Statement which the team turned around in under six months, with planning permission granted in December 2008. The changes to the junctions and side roads also required an amended Side Roads Order to be prepared.

Scheme costs

In addition to meeting the main objectives of the bypass, the new scheme also led to a reduction in the project cost of over £35M, £9M of which resulted from the change in carriageway standard. This demonstrated value for money to the Welsh Assembly Government and enabled them to fund the scheme while also releasing much needed capital funding for other public projects throughout Wales.

The budget for the scheme including historic development costs, land and inflation is just under £90M with construction costs at £65M. The main construction works commenced in October 2008 and the scheme is due to open to traffic in September 2010.

Summary

The development of the scheme has been a genuine collaborative effort involving not just RCT, Capita, Costain and Atkins but also Costain's supply chain and numerous statutory bodies including the Environment Agency, the Countryside Commission for Wales and all the major Statutory Undertakers. The benefit of having an enlightened client who is prepared to go back to the primary scheme objectives to achieve a value for money scheme cannot be stressed too highly. The final scheme in engineering terms is a lot simpler than the original scheme. It is certainly on a smaller scale than the original, but the Church Village Bypass now consists of only 'must have' elements rather than containing 'nice to have' elements and, more importantly, it is a scheme that the Welsh Assembly Government considered good value and was willing to fund in full.

The scheme is currently expected to open on time in early September 2010 and within the out-turn budget of £90M.

Lessons learnt

The Value Engineering process carried out on Church Village has given us the following key lessons learnt;

(i) The team must be truly integrated, include all interested parties and must make full use of the client's invested knowledge of the scheme

(ii) Stakeholders must be engaged as early as possible (we had spoken at length with all the SUs in the tender period and even started to agree changes to some of the diversions should we be successful!)

(iii) A peer review of the proposals is an essential part of the process as it gives a reality check on the proposals and comfort to the team that their proposals are achievable and that all avenues have been explored

(iv) The client needs to decide early in the process whether he wishes to go back to the fundamental objectives of the scheme

(v) No idea is too outrageous – it may prompt another idea which is viable

(vi) The major constraining factor was the CPO boundary – ideally the VE should be carried out before the CPO procedure.

Atkins' involvement

Atkins has been involved with the scheme since the issue of the tender documents in March 2007, supporting Costain in preparation of the tender. Over 120 different staff from across Atkins worked on the development phases which included the Value Engineering and the preparation of the revised Planning Application and Environmental Statement. To date, over 200 staff from Atkins have worked on the detailed design and site supervision and the anticipated final fee for all stages is just over £3.5M.

VE Summary

Original Scheme	Final Scheme
7Km - new dual carriageway	6Km - new single carriageway
4 intermediate junctions	4 intermediate junctions
3Km side roads	1.5Km side roads
9 road bridges	2 road bridges
2 green road bridges	0 green road bridges
0 dormice bridges	3 dormice bridges
2 accommodation bridges	0 accommodation bridges
2 footbridges	3 footbridges
2 pedestrian underpasses	1 equestrian underpass
1 cattle creep	2 cattle creeps
11RC box culverts	7 steel arch culverts
500 Lin. m RC retaining walls	0 Lin. m RC retaining Wall
300 Lin. m reinforced earth retaining walls	0 Lin. m reinforced earth retaining walls
67 items of utility works	53 items of utility works
Total cut 942,000m^3	Total cut 524,800m^3
Total fill 851,000m^3	Total fill 524,800m^3
Ty Garreg tip excavation 298,000m^3	Ty Garreg tip excavation 80,000m^3
Tyre/pfa excavation 121,000m^3	Tyre/pfa excavation 55,000m^3
Rhiwsaeson Municipal Tip excavation 100,000m^3	Rhiwsaeson Municipal Tip excavation 0m^3

Drainage design on M25 motorway widening project (J16 - J23)

Brijesh Vats
Drainage Engineer
Highways & Transportation

Abstract

Drainage design on the M25 motorway comprises complex drainage networks and attenuation systems provided in conjunction with the other highway infrastructure and existing utilities' services in the confines of widening the motorway within the existing boundary. The online attenuation is provided in the form of modular geo-cellular storage tanks. Other features used in the design are hydrodynamic separators, oil in water monitoring, petrol interceptors and emergency impoundment facilities. This paper outlines some of the innovative design procedures employed in the analysis and design of the drainage system and presents the buildability and clash detection features developed and employed during the design to ensure that construction was 'right first time'.

Introduction and background

The M25 is one of the busiest motorways in Europe and a key strategic orbital route around London. It carries up to 200,000 vehicles per day and is heavily congested at peak time and during road maintenance and incidents. A long term strategy has been developed for the sustainable management of the M25, based on the multi-modal study carried out in 2002. Under this strategy, the Highways Agency awarded the Connect Plus consortium (a joint venture of Skanska, Balfour Beatty, Atkins and Egis) the M25 motorway widening project under a 30-year Design Build Finance and Operate (DBFO Contract). The contract includes the capital works to improve the capacity of existing three-lane sections. The contract also includes the responsibility for operation and maintenance of the motorway and its structures.

The areas to be widened have been split into 3 sections: Section 1 (J16 – J23), Section 4 (J27 – J30) and Section 6 (Hatfield Tunnel). The contract requires that Section 1 and Section 4 are completed before the London Olympic Games in 2012.

This paper focuses on the design of Section 1 of the scheme and discusses the analysis of existing outfalls, capacity analysis of existing drainage systems and the design of new sustainable drainage systems. Section 1 is divided into three subsections: Section 1X (J16 – J18), Section 1Y (J18 – J21a) and Section 1Z (J21a – J23).

Project constraints

There were a number of constraints imposed on the design, many of which are unique to widening projects. These included:

- All construction had to be within the existing highway boundary
- The existing drainage system had to be used wherever possible
- Online attenuation had to be used because no additional land was available for balancing ponds
- Space limited by hard retaining solutions, existing structures, communication and lighting features within the narrowed verges and central reserve
- Avoiding damage to existing utilities
- Programme constraints resulting from the impending Olympics.

Existing drainage system

The M25 was constructed between 1973 and 1986. The majority of the drainage currently serving the motorway is believed to be that which was originally constructed. The principal exceptions are where major alterations have been made, for example to incorporate climbing lanes between Junctions 16 and 17, and Junctions 18 and 19, or additional lanes through various junctions. The existing highway drainage system is a conventional system using combined surface water and ground water filter drains, except where the motorway is constructed on embankment, where kerbs and gullies draining to a sealed carrier pipe are used.

The assessment of the existing drainage system was carried out based on the as built information, Figure 1, and an initial survey provided by the contractor.

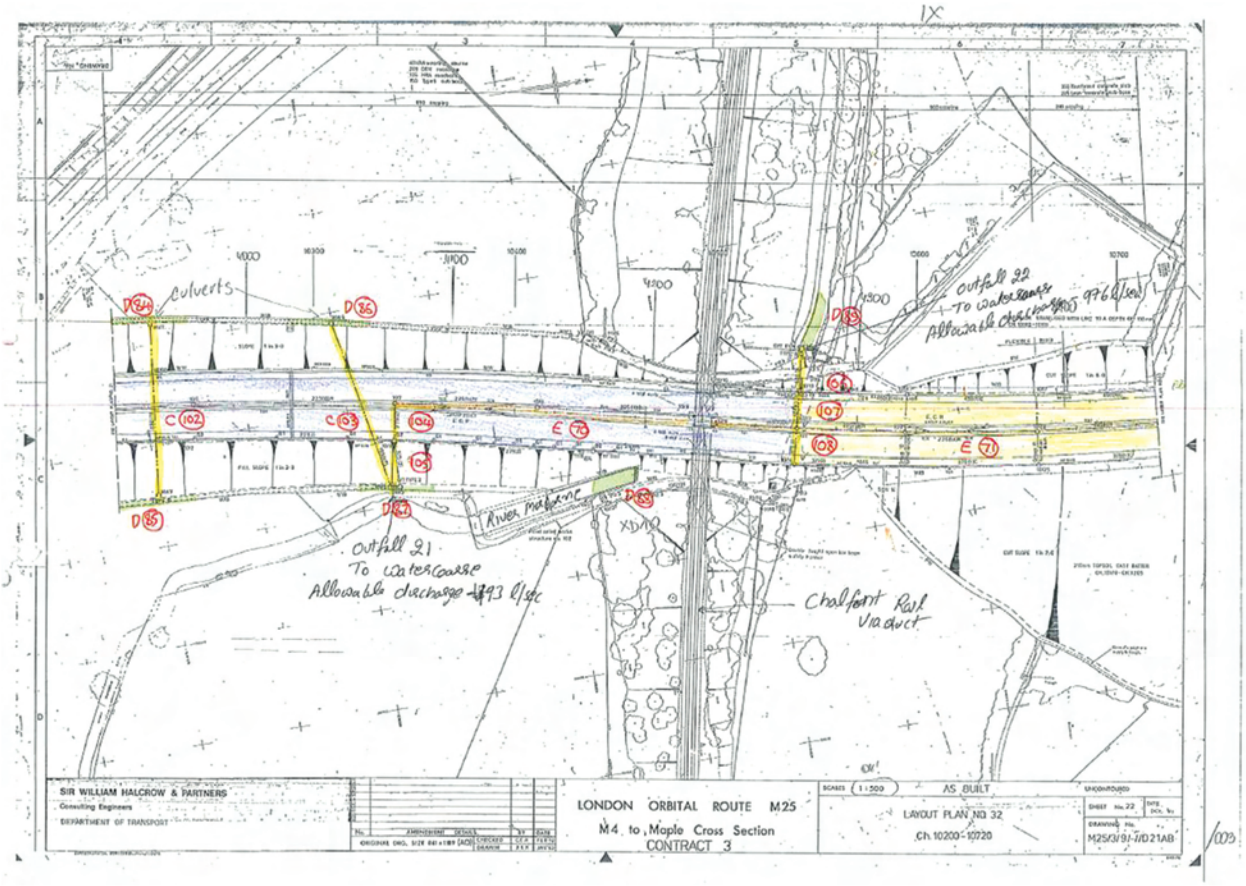


Figure 1 - Drainage as-built information at Chalfont Railway Viaduct

Proposed drainage system

The new drainage system has been designed in accordance with the Highways Agency's Design Manual for Road and Bridges (DMRB)[1] standards and Construction Works requirements[3].

In addition to the above, we have provided the following innovations in drainage system design in order to achieve the sustainability and environmental objectives:

Hydrodynamic separator/ downstream defender: The hydrodynamic separator is designed to remove settleable solids, grits, silts and particulate heavy metals and also retain oil and other floating debris arising from highway surface water run-off. The sediment capture performance of the device is significantly better than that of conventional catchpits and bypass separators. The provision of such elements in drainage systems is not explicitly covered by DMRB standard HD33/06

The system has not been previously installed on any Highways Agency scheme, hence a departure was required for this system. Installation of this on the highways scheme is consistent with the scheme objectives for sustainability and environment.

Oil in water monitoring: We have provided automatic penstock catchpits with an oil-in-water probe monitoring system to outfall in source protection zone 1 and 2 (these zones are defined by the Environment Agency for ground water sources such as wells, boreholes and springs used for public drinking water supply). This unique system was provided to satisfy the Environment Agency in providing an immediate response system in the event of a serious hydrocarbon spillage incident where the highway drainage discharges into the ground.

No Highway Agency standard or detail for monitoring hydrocarbon spillages combined with automated emergency closure device are available hence departure was required for this system.

The drainage system and its ancillary features have been designed to the design storms with a further 20% increase for climate change.

The increased flows emanating from the widened motorway area have been attenuated, as discussed below, so that the existing discharges to watercourses or existing soakaways are not exceeded. This additional attenuation has been provided in accordance with the Construction Requirements[3] to accommodate a rainfall event (including the allowance for the climate change) as follows:

- 1 in 100 years return period at outfall into watercourses
- 1 in 10 years return period at outfall into soakaways
- 1 in 30 years return period if attenuation is provided where overflow would be onto the adjacent carriageway.

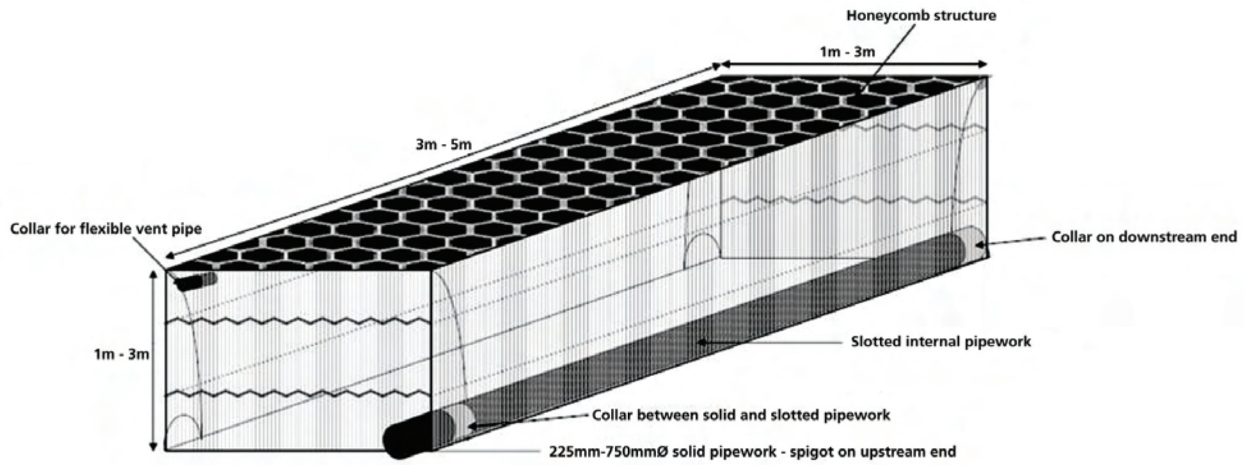


Figure 2 - Typical detail of geo-cellular tank

Attenuation in geo-cellular storage tanks

To provide the necessary attenuation discussed above, an innovative and sustainable solution has been provided using geo-cellular storage tanks which are constructed in segments (3m x 2.5m x 2.0m) as shown in Figure 2. These are honeycomb modular structures made from recycled PVC. These enable the amount of flooding in the event of storms to be restricted and so play a major part in protecting aquatic surroundings.

The use of prefabricated modular attenuation tanks is the departure as the provision of such a element in drainage system is not explicitly covered by DMRB HD 33/ 06, although paragraph 7.25 refers to the flow controls provided at such a system.

These tanks are relatively light and therefore much safer in terms of construction and maintenance than the conventional options of online attenuation, such as box culverts or oversized pipes.

Due to their light weight and minimum handling risk in comparison to conventional storage options, these tanks reduce the construction time. Installation of a geo-cellular tank is shown as Figure 3. The tanks are laid flat to generate the maximum attenuation benefits but the road profile is quite steep in some networks which necessitates a deep excavation for the tanks in these locations.

Figure 3 - Installation of geo-cellular tank

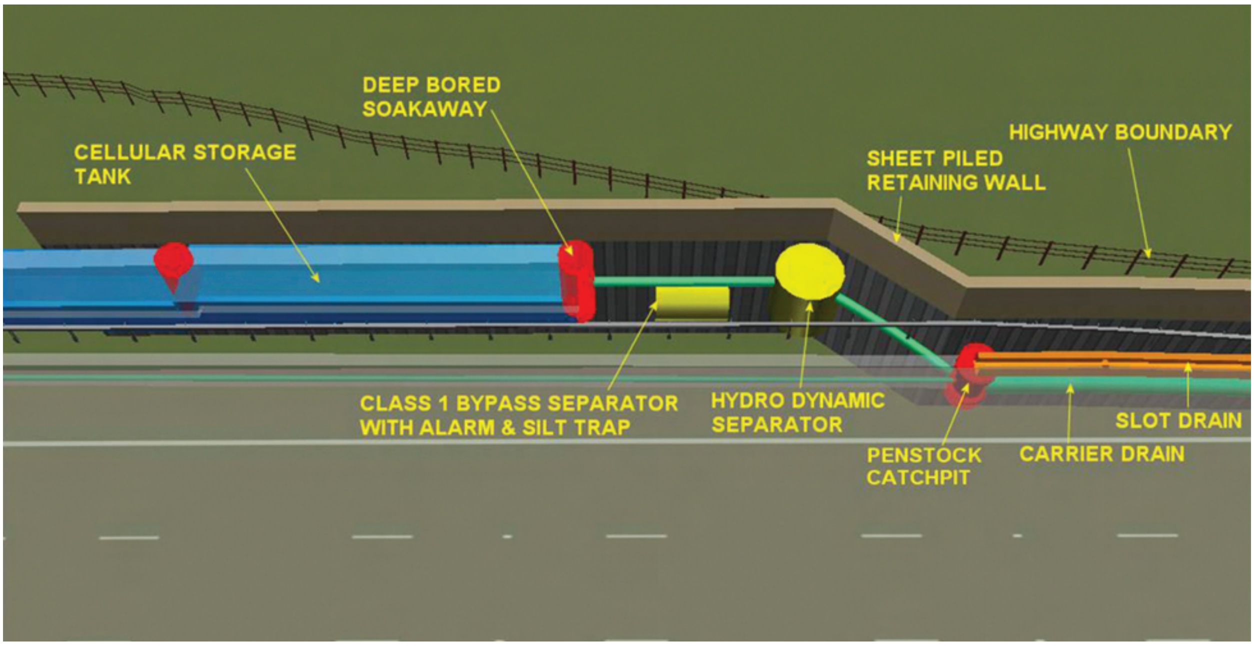


Figure 4 - Typical soakaway outfall arrangement

Design tools

In-house software tools were developed for the drainage team working on the M25 DBFO project using Visual Basic. These tools allow the fast and consistent design of the drainage network within Autocad and export the network as a virtual reality model for the clash detection. Once developed as an Autocad drawing, the network can be imported into the software package Microdrainage (using its System One module), to generate storm files.

A brief description of the tools we have developed on this project, Encore, SWAT and SWAN:

- Encore is used to prepare drainage networks by placing Autocad blocks and polylines by chainage along an alignment and offset from a feature such as the edge of pavement. For drainage network design, the blocks are manholes, soakaways, headwalls, catchpits, penstocks valve, petrol interceptor, downstream defender and emergency impoundment facility and the polylines are carrier drains, filter drains, slot drains, cellular storage tanks.
- SWAT is used to export the network prepared within Encore to form a Microdrainage storm file. This tool reads the plan layout of the drainage network together with triangulated height data from MX to write out a correctly structured storm file with lengths, co-ordinates, cover levels, catchment areas and other global parameters such as return period, FSR details, maximum rainfall, maximum slope and minimum cover depth. SWAN is used to annotate the Autocad drainage drawings from the analysed Microdrainage storm file. It also creates Virtual Reality files for automatic clash detection.

Pollution control

Pollution control devices have been provided at outfalls to reduce the risk of contamination of watercourses and ground water. Special attention has been given to the proposed pollution control measures for the soakaways in Source Protection Zone 1 and 2 to satisfy the requirements of the Environment Agency's Pollution Prevention Guidance Notes (PPGN) and the commitments given in the Environmental Statement[2] for the treatment of routine run-off, so that there is no overall detriment to the existing water environment, after the motorway is widened. A typical arrangement is shown in Figure 4.

The protection to the aquifers has been provided by the following:

- Each outfall in source protection Zones 1 and 2 has been provided with its own control systems which automatically close a penstock valve on detection of a high level of hydrocarbons within the penstock catchpit. These penstocks are mounted on the outgoing pipes in their catchpits (some require two); all need to be fitted with open and closed limit switches. On detection of hydrocarbons above the threshold the control system closes the penstock and sends an alarm to the Knowledge Management Centre (KMC). In the event of the system operating outside of a major spillage, the KMC can investigate and override a false alarm to open the penstock remotely and make the system fully available again.

Hydrodynamic separators/downstream defenders are proposed within the drainage networks where slot drains are proposed and the network outfalls to a soakaway. This system is designed to capture and retain sediment within the base unit, with oil and floating debris retained within the outer annulus irrespective of the flow rate through the device. The

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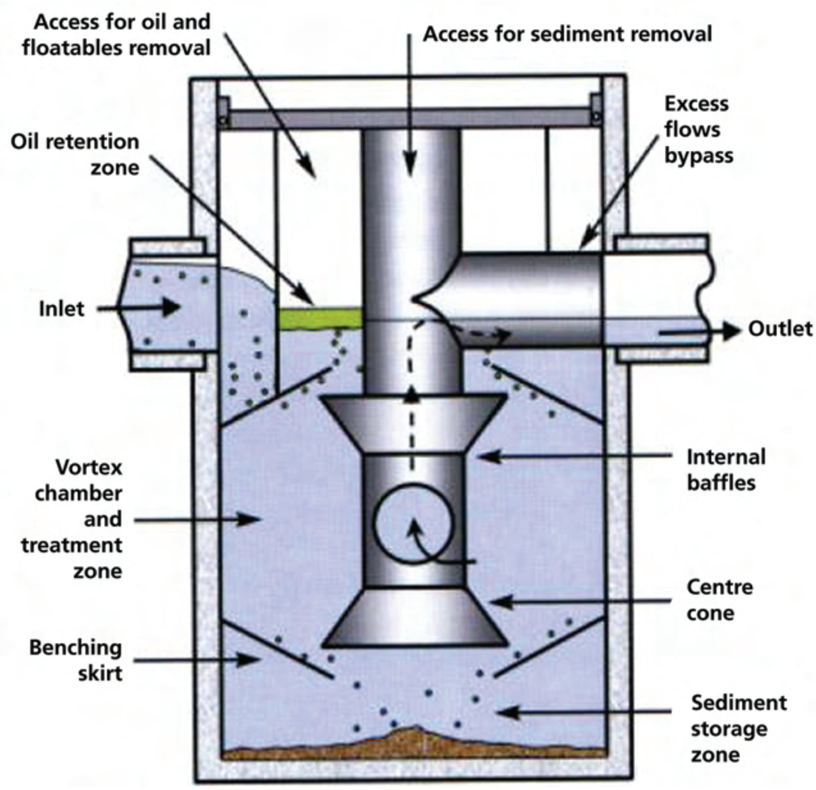

Figure 5 - Hydrodynamic separator

sediment capture performance of the device is significantly better than that of conventional catchpits and bypass separators and the hydrodynamic separator has been sized in accordance with the flow table based on Hydro International design flow rates for Downstream Defenders.

Based on the indicative percentage treatment efficiencies in HD33/06 and HA 103/06 bypass separators and therefore hydrodynamic separators are far more efficient than the 7% efficiency of filter drains in removing heavy metals.

Hydrodynamic separators are far easier to maintain and clean in comparison to cleaning filter drain material. Figure 5 shows a typical detail of hydrodynamic separator.

- Class 1 bypass hydrocarbon interceptors are provided on all outfalls. This facilitates the capture and cleansing of the first flush of the carriageway, limiting the hydrocarbons that are passed through to 5mg/l. The subsequent less contaminated flow bypasses the device. The interceptor also has an automatic alarm and silt deposition facility.

Each outfall is provided with an accidental spillage containment facility. This is provided either in the form of a 40m long, 900mm diameter pipe, or an offline Emergency Impoundment Facility (EIF) with a 25m^3 volume buried tank.

In addition to the above, Sustainable Urban Drainage System (SUDS) features are incorporated in the form of filter drains, balancing ponds and infiltration ponds with vegetative treatment and sedimentation forebay. These were adopted in order to provide the water quality and quantity treatment requirements at source in advance of the outfall point to the receiving pond. Figure 6 shows a typical arrangement for an outfall to an infiltration pond.

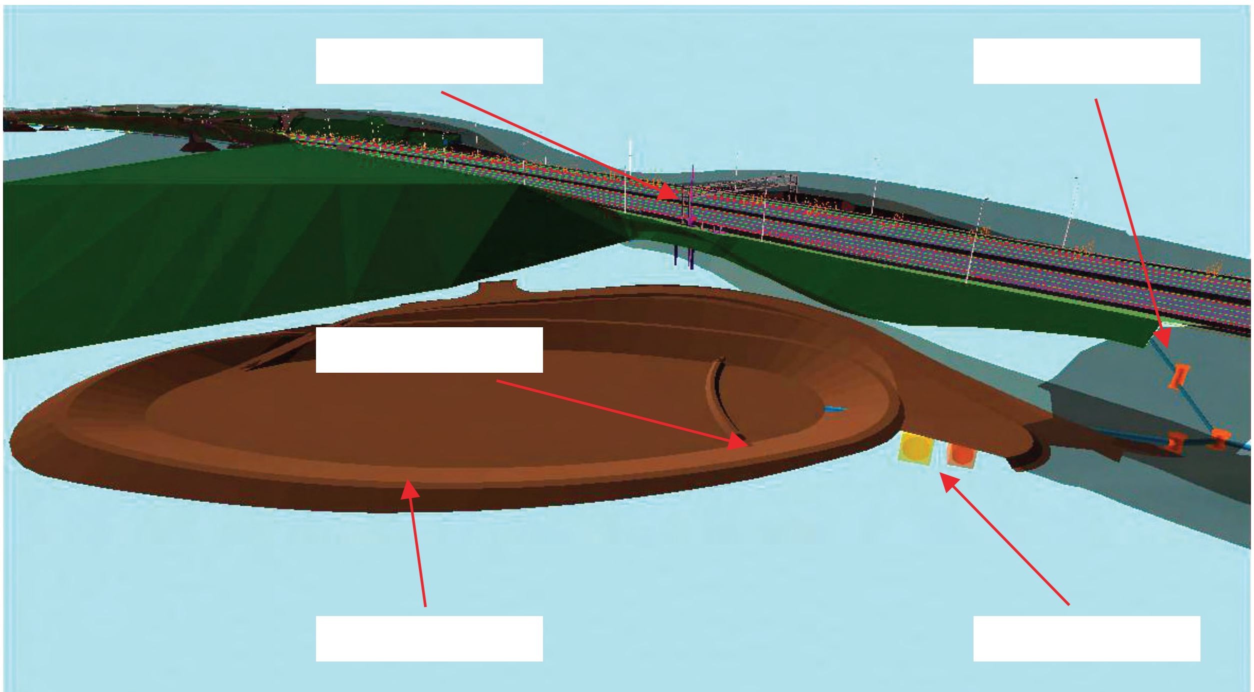

Figure 6 - Infiltration pond

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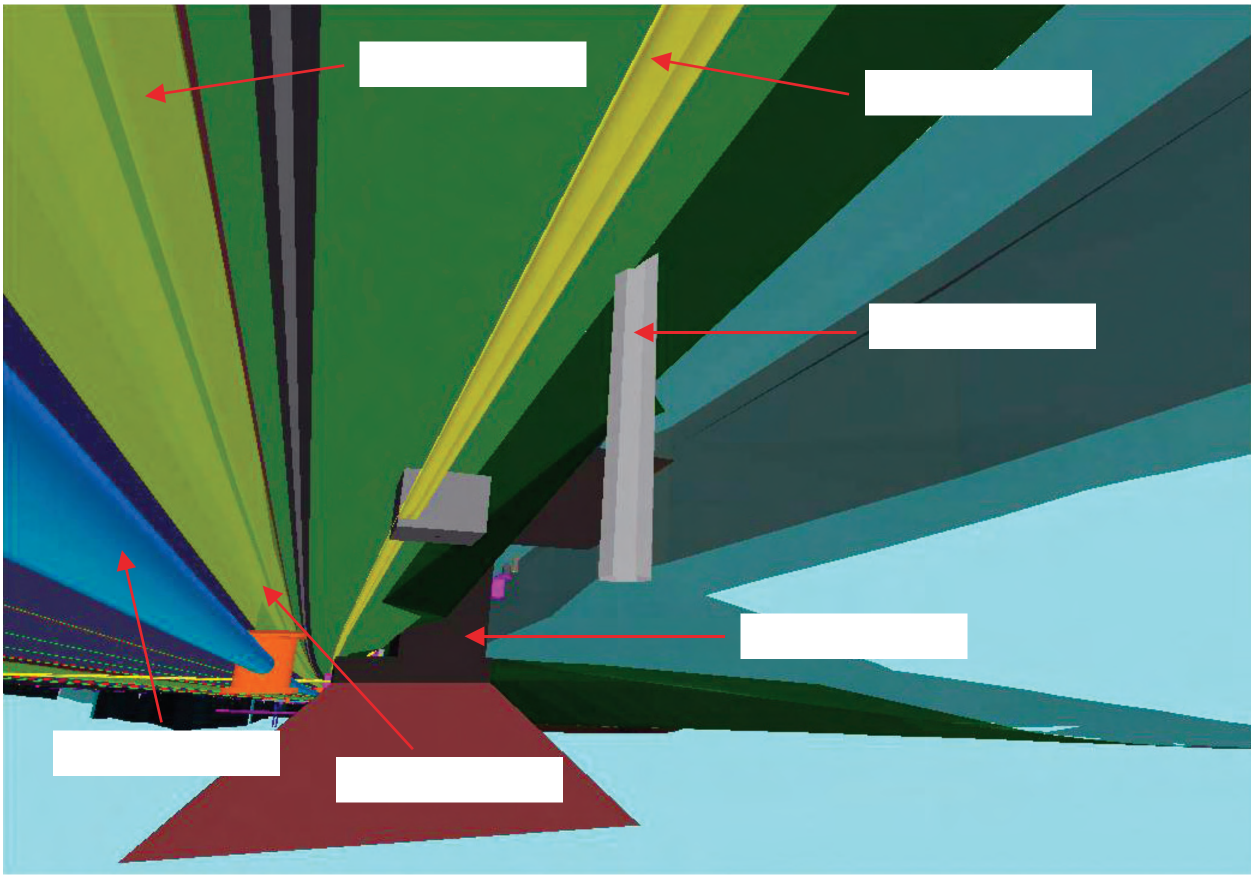

Figure 7 - Drainage features adjacent to bridge

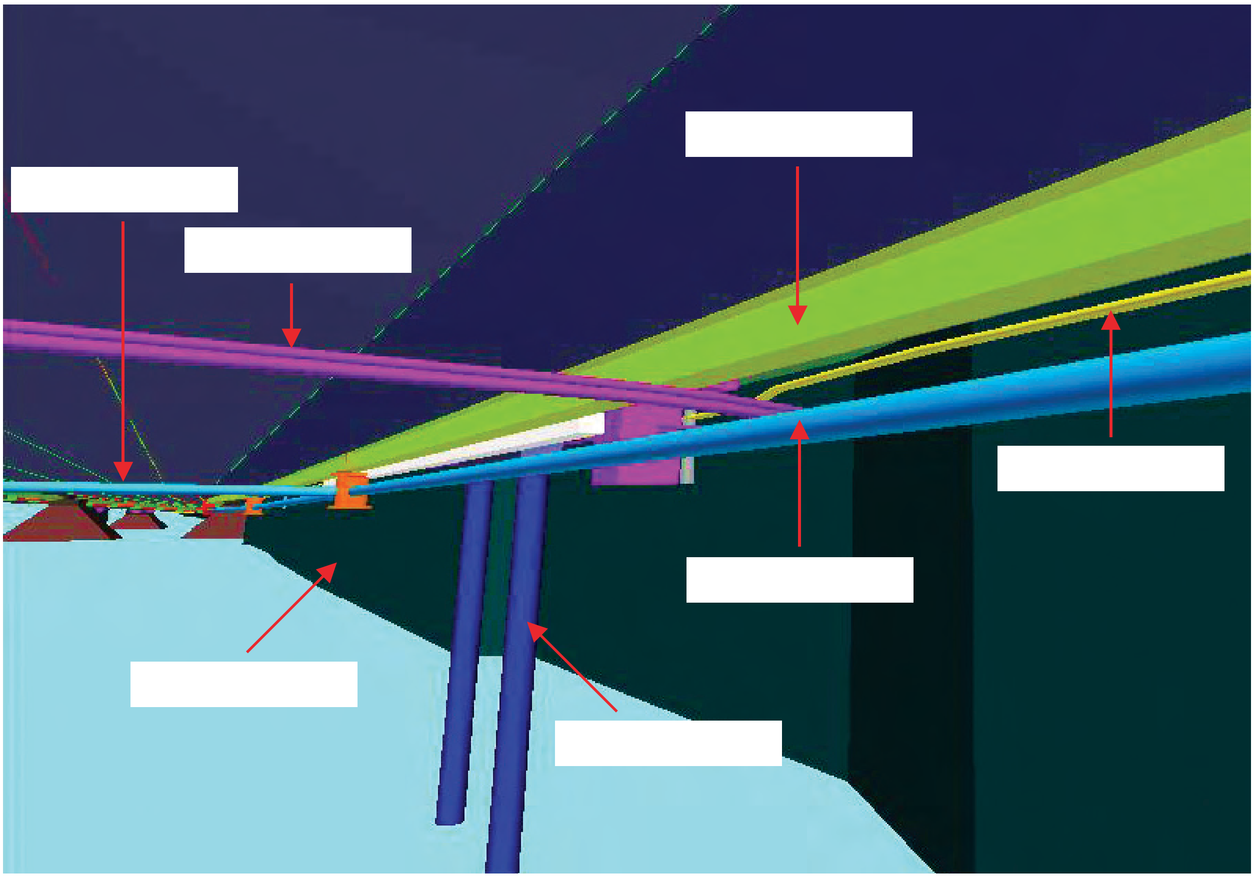

Figure 8 - Drainage features adjacent to gantry

Buildability and clash detection

The new drainage system had to fit in with other new and existing infrastructure facilities (such as communication and lighting ducts, road restraint systems, hard retaining solutions and existing structures) and existing utilities. Because of this congestion, a Virtual Reality model was used to check the design for clash detection and buildability. The Virtual Reality model checked for clashes between all the existing and proposed infrastructure: safety fence posts, drainage networks, sheet piles, lighting and communication ducts, gantry bases and structure foundations. This has assisted the preparation of the robust design prior to commencement of works on site to minimise re-design on site and the potential for abortive works. The Virtual Reality model was created with in-house software tools. It was imported to Autodesk Navisworks for clash detection. Figure 7 and 8 show snapshots from the Virtual Reality model.

Conclusion

In conclusion, we have successfully designed the effective drainage system for the complex widening of M25 motorway. This includes fitting in drainage features along with various utility services and other project constraints. In-house developed design tools were successfully used to achieve consistent and clash free drainage network design. This design was particularly complex because of the numerous constraints highlighted.

References

1. Design Manual for Roads and Bridges volume 4, section 2 Drainage
2. M25 Widening Junction 16 to 23, Environmental Statement Volume 1
3. M25 DBFO Schedule 4 Construction Requirements

Acid Phase Digestion at Derby STW - Context and preliminary optimisation results

Alexander Bjorn

Assistant Process Engineer

Water & Environment

Abstract

Maximising the revenue generated from renewable energy production is increasingly important to water companies in Asset Management Period 5 (AMP 5) following tough price setting by the regulator, and given the current wider economic landscape. The anaerobic digestion of sewage sludge is an important source of renewable energy. Acid Phase Digestion (APD) is reported to increase volatile solids destruction and gas production from anaerobic digestion.

Following research into existing systems on the market, an APD plant of Severn Trent Water's own design was retrofitted into the existing digestion process at Derby sewage treatment works (STW). The plant was designed with exceptional operational flexibility to determine optimum process parameter setpoints.

This paper relays the operational experience and data gathered by Atkins from the first half of the optimisation programme, and compares the performance data against the benchmark of previous conventional anaerobic digestion performance at the site.

Background

The water industry faces a significant challenge in aligning itself with the Government's national target of reducing CO_2 equivalent greenhouse gas emissions by 80% (against 1990 levels) within the next 40 years. Improved energy savings can be sought in all aspects of the industry, from reducing consumption, targeting leakage, and investing in new technologies. One such technology is the application of acid phase digestion to sewage sludge.

Anaerobic digestion has been the primary technology used to stabilise sewage sludge in the UK for over 30 years. In recent years, technological innovations have been developed to optimise this process. Acid Phase Digestion (APD) is just one of these. Anaerobic digestion takes place in three stages; hydrolysis, acetogenesis and methanogenesis. Acidogenic bacteria responsible for the first two stages prefer an environment of pH 5 and 1-3 days retention time, while methanogenic bacteria prefer pH 7.5 and 7+ days retention time.

APD works on the principal of physically separating the two sets of bacteria allowing both to operate in optimal conditions, improving the overall efficiency of the process. This technique serves the purpose of maximising volatile solids destruction (VSD) and therefore renewable energy production (from biogas), while simultaneously reducing solids for reuse.

Severn Trent Water processes around 150,000 tonnes of dry solids sewage sludge per annum, the majority of which is anaerobically digested. There is currently over 30MW of installed combined heat and power (CHP) capacity to harness the energy from the resultant biogas. Innovative technologies which provide even small improvements in efficiency have the potential to yield significant additional savings and revenue.

Intoduction

A number of proprietary systems based on the APD concept are on the market, including Monsal's Enzymic Hydrolysis and Veolia's Thelys systems. In 2004 Thames Water designed and built their own system, utilising the concept of acid phase digestion, at the 220,000 Population Equivalent (PE) Swindon Sewage Treatment Works (STW)[2].

Following research undertaken in 2004[6] and visits to other sites in the UK, Severn Trent Water were keen to take advantage of the technology's benefits. Economic appraisal of Severn Trent Water's version of APD showed it was 28% cheaper on capital costs than the Monsal Enzymic Hydrolysis process[3]. Severn Trent Water decided to follow Thames Water's example and undertake their own detailed design for APD at their Derby STW.

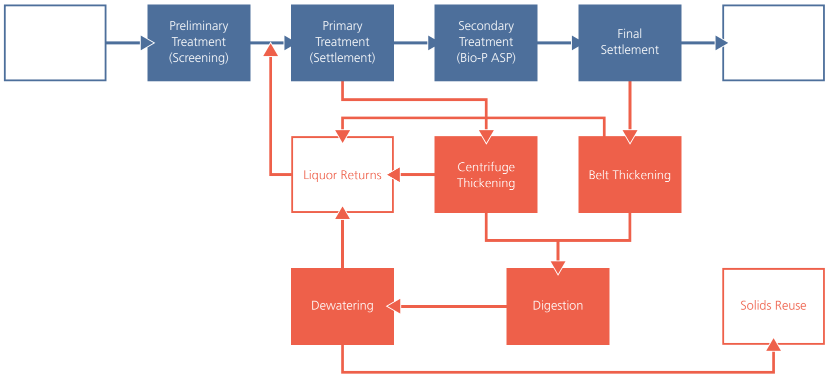

Figure 1 - Derby STW Process Block Diagram. Sewage treatment processes are shown in blue and sludge treatment processes are shown in red

Derby STW treats the sewage of 475,000 people, using conventional preliminary and primary treatment, followed by a biological nitrogen and phosphorous removal activated sludge plant. The sludge stream consists of primary sludge thickening by centrifuge and surplus activated sludge (SAS) thickened by belt thickeners, thickened sludge blending tank, APD, three Mesophilic Anaerobic Digesters (MADs) and dewatering of digested sludge by centrifuge, as shown in Figure 1.

The APD plant at Derby STW was commissioned in February 2009 and has since been subject to a programme of optimisation. This paper details six months of results from the first half of the optimisation programme. Figure 3 shows the layout of APD at Derby STW. Atkins played a key role in managing the optimisation programme with Severn Trent Water. Implementation of the series of reactor configurations, enabled the identification of a preferred reactor configuration in parallel mode. The work done to date and remaining optimisation will deliver the following benefits:

- OPEX savings largely due to the reduction in power bills enabled by the excess biogas produced
- A robust basis for a rationalized reactor design that can be applied to future APD plants at many other sites in the Severn Trent Water region, delivering the benefits of APD for reduced capital expenditure.

The methodology, results and conclusions are presented in more detail in the following sections.

Design

APD at Derby was designed as a full scale development plant, with exceptional in-built operational flexibility to enable the determination of design parameters for future plants. A key difference in the design of the plant from that of Thames Water, is that the system consists of two process tanks compared to one tank system at Swindon, and six tanks or more in the proprietary systems mentioned previously. This allows the plant to operate either in parallel or series mode[3], with the view to determining the impact of single or multi-tank systems.

Two 800m^3 APD tanks were constructed between the existing thickened sludge balance tank and the three existing MADs (each with a nominal volume of 3100m^3). A 200m^3 APD buffer tank (BT) was constructed immediately downstream of the APD tanks to ensure feed continuity to the MADs. The two APD tanks are fitted with an unconfined gas mixing system, but the balance tank has no mixing. The flowsheet for APD at Derby STW is shown in Figure 2.

Severn Trent Water have considerable experience in the design of MADs, having built dozens across the region in the last 50 years, and this was drawn on for the design of the APD plant. However, unlike MADs which are ‘fill and spill’ with a fixed top sludge level and variable retention time, APD tanks are designed as ‘fill and draw’ with a variable top sludge level and fixed retention time set point, providing unique design challenges. This is critical, as excessive retention in APD would allow methanogenic bacteria to establish themselves. Control of level allows us to establish exactly what the optimum retention time is.

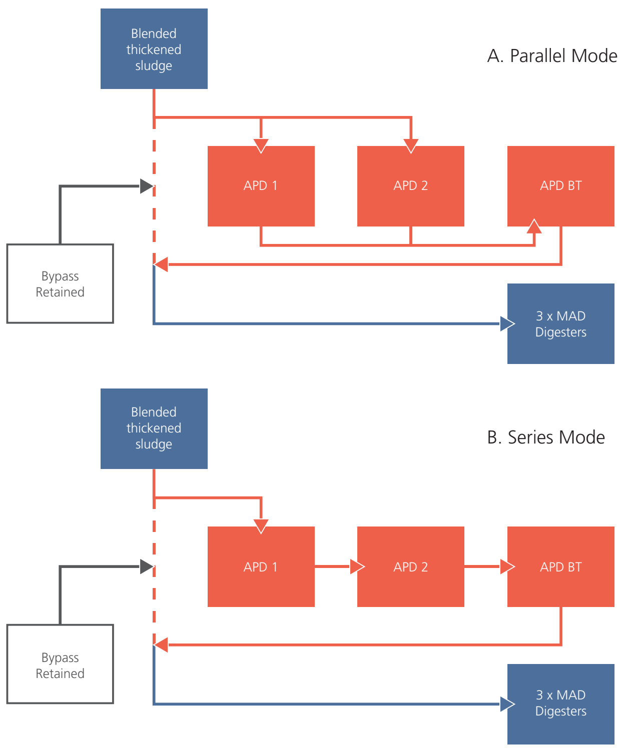

Figure 2 - Schematic of series and parallel modes. This paper is primarily concerned with the results of optimisation in parallel mode.

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Figure 3 - APD at Derby STW. The two tanks in the middle are APD 1 and APD 2, with their heat exchangers and recirculation pumps just in front. The smaller tank to the right is the APD BT. You can just see the edge of one of the larger MADs on the left of the frame. The green tank in the foreground to the right is the gas holder.

Table 1 - Details of optimisation stages discussed in this report

Stage	Retention Time	Temperature	Duration	No of Retentions
1	2 days	38°C	82 days	4.2
2	2 days	42°C	44 days	2.3
3	2.5 days	38°C	54 days	2.8
4	2.5 days	42°C	33 days	1.4

As well as additional tankage, a new 500 kW CHP engine was added to the existing 1126kW generator capacity to harness the potential energy of the expected additional biogas.

Since commissioning in February 2009, the plant has been following a programme of optimisation. Table 1 gives details of the four commissioning stages (based on changes in APD operating temperature and retention time) in parallel operation discussed in this paper. The duration of the stages was set at a minimum of twice the average total digester retention time, to allow the effects of changing a parameter to manifest themselves clearly in the data.

Results

Discussed in this section are a number of parameters measured during the optimisation trials. Each parameter provides a different measure of process performance and has its own benefits and limitations. As you move further away from APD, more factors can influence the results and more 'noise' is present in the data.

Volatile fatty acids (VFAs) are the product of acetogenesis, the second stage of anaerobic digestion. The key objective of APD is to break as large a proportion of the volatile solids in the sludge down into VFAs, and provide a feedstock rich in VFAs to the MADs for conversion into methane.

VFA production is an important indicator as it is a direct measurement of APD performance, and is not influenced by downstream factors like MAD performance and CHP efficiency.

VSD is the most common measurement of digester efficiency. Volatile solids are the proportion of solids in the sludge that can be burned off when ignited to 500°C, and can be presumed to be the organic fraction of total solids (although in reality it is more complex). It is this organic fraction that is amenable to conversion into biogas, and maximising the proportion destroyed through the digestion process maximises biogas production. However, this is not a direct measurement of APD performance, as MAD mixing, heating and loading rate (amongst other things) can all affect VSD.

Measuring gas production using online gas flowmeters on the gas collection pipework on the digesters is a simple way to quantify process performance.

Figure 4 - Vivianite scale collected from the APD heat exchangers. The face showing is smooth and convex, the side attached to the corrugated tube of the heat exchanger. The concave side of the scale is expesed to the flow of sludge and is typically fibrous and rough. The three small samples to the left are calcite and were found adjacent to the heating elements, they are not thought to significantly impede heat transfer. Biro for scale.

But as a measurement of APD performance, these are more factors that can skew results. Calibration of the flowmeters, moisture content of the gas and changes in pressure can all influence readings.

Energy production from the CHP engines can be measured most accurately of all that parameters discussed. But it is also furthest away from direct measurement of APD performance. As well as the factors influencing VSD and gas production, engine efficiency, gas flared off and engine down time all affect this measurement.

By looking at all available measures of performance we can increase our confidence in the assessment of the impact APD is having on the digestion process.

Operational experience

Initial optimisation was hampered by scale formation in the external concentric tube heat exchangers, and reactor temperature could not be maintained at the stage setpoint. Following laboratory analysis this blue-grey scale was identified as iron phosphate hydrate, or vivianite ($Fe_3(PO_4)2.8H_2O$).

Figure 4 shows a sample of vivianite scale collected from one of the heat exchangers following jetting. Vivianite scale is relatively uncommon in sewage sludge treatment, but it is known that the addition of iron salts to wastewater can result in the precipitation of ferric hydroxyphosphates and vivianite[4]. Some STWs dose ferric salts to remove phosphate from the wastewater.

Derby STW is an enhanced biological phosphorous removal (EBPR) plant, i.e. there is no ferric dosing, and as such vivianite was not considered a risk. But the Derby STW catchment receives wastewater from a local water treatment works (WTW), which is rich in iron.

This is thought to be the source of the additional iron required to form vivianite in the sludge stream. As an illustration, Table 2 compares Derby STW crude influent iron levels with those of another large Severn Trent STW.

The solubility product of vivianite is influenced by temperature[1], therefore the hot water target temperature in the heating loop was reduced from 65°C to 60°C to assess any impact on scale formation before more drastic measures (such as stopping WTW sludges discharging into the catchment, or steam injection heating) were considered. On closer inspection, however, SCADA trends showed the hot water temperature was subject to frequent spikes up to 80°C, albeit for relatively short durations (~60 min). Following a period of investigation, a faulty valve was found on one of the CHP engines. This was repaired and no more temperature spikes have been observed. Since the heating loop has been operating at a stable 60°C, the formation of vivianite has been reduced and this in conjunction with infrequent jetting of the heat exchangers enabled the optimisation programme to progress.

Table 2 - Iron levels in settled crude sewage at Derby STW and other STW

	Derby STW Fe content	Other STW Fe content
Average	11.64 (1.6) mg/l	3.68 (0.3) mg/l
Maximum	39.10 mg/l	5.24 mg/l

Data are January to July 2009 samples of settled crude sewage. (Standard Error)

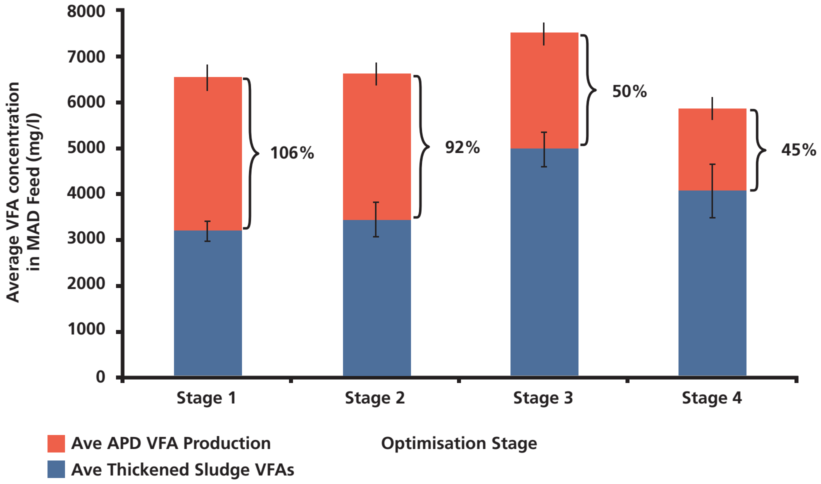

Figure 5 - Volatile Fatty Acid (VFA) increase by stage. The blue section is the average concentration of VFAs in the thickened sludge fed to APD. The red bar is the average concentration of VFAs produced in the APD process. The total value of the red and blue bars is the average concentration of VFAs fed to the MADs

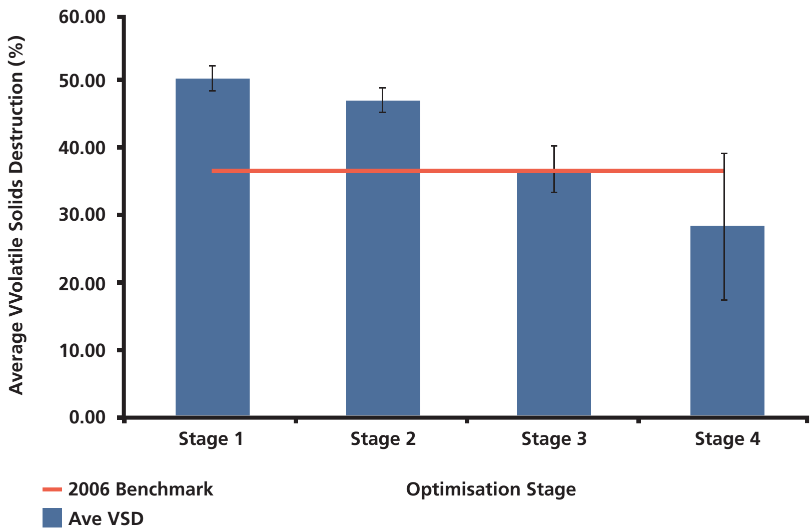

Figure 6 - The average overall VSD of APD+MAD against the 2006 benchmark for each of the optimization stages

VFA production

Average VFA concentration in the thickened feed sludge varied between the optimisation stages, as illustrated in Figure 5. The percentage increase in VFA concentration following APD is considered a truer reflection on relative stage performance than absolute concentration. Stage 1 resulted in the highest average percentage VFA production in APD with 106% of APD feed levels.

Volatile Solids Destruction

During the feasibility design of APD at Derby STW, three months of sampling was undertaken in 2006 from one of the MADs to determine the VSD benchmark, this was 36.5% VSD. Further sampling was undertaken during the three months leading up to APD commissioning (November 2008-February 2009), again to determine a performance benchmark. Sample results indicate VSD was unusually high with an average VSD for this period of 56.1%. The discrepancy between this figure and the 2006 benchmark is difficult to explain. As an extra check, the average VSD of the MADs was calculated using all the data available for 2007 and 2008.

Although variable, this showed an average VSD of 39.8 %, much closer to the 2006 benchmark. In this light, the 2006 benchmark is considered a more accurate measure of long-term MAD performance.

Figure 6 shows that VSD was highest for combined APD and MAD during Stage 1, at 50.2%, i.e. 14% more VSD with APD than with MAD alone. To put this into context, we can compare results with Thames Water's Swindon plant and results from two of Monsal's multi-reactor Enzymic Hydrolysis plants. The results from Derby fall mid-way between Enzymic Hydrolysis at Macclesfield and Bromborough; combined 58.8%[5], and those from Swindon STW; 40-45%)[2]. This is promising as the plant at Derby operating in parallel is effectively two single reactors. As such one would expect results to be similar to the Swindon plant. This bodes well for series operation, which will be a closer approximation of the multi-tank Enzymic Hydrolysis systems.

Gas production and energy production

Table 3 reveals a marked increase in total gas volumes following transition from 2 days retention to 2.5 days retention. However, this coincides exactly with gas flow monitors in MAD B and MAD C being recalibrated, and is not a true reflection of gas production. The gas flow monitor in MAD A, however, was not recalibrated and therefore the relative volumes measured between stages are considered accurate. Gas production in MAD A is highest during Stage 1, at 38°C and two days retention. This is more substantial as the gas mixing system in MAD A was out of service during the first three stages of the optimisation programme.

Energy production is the ultimate measure in assessing the effect of APD on the digestion system but energy production does not take into account the gas that is flared off due to CHP faults. Volumes of gas flared off were similar between the optimisation stages, and does not influence the assessment.

Table 3 - Average gas volumes generated in the MADs and average sludge feed volumes

Stage	Average Daily Gas Production Vol (m^3)				Average Sludge Feed Volume (m^3/d)
	MAD A	MAD B	MAD C	Total	
1	2791	2031	2917	7740 (197)	528 (19)
2	2111	2140	3139	7574 (350)	541 (19)
3	2544	3987	3464	9995 (190)	529 (16)
4	2392	3197	2517	8245 (366)	419 (20)

(Standard Error)

Figure 7 shows the total energy production during each optimisation stage and compares it to the same period in 2008. Note: this graph does not show relative performance between the stages because the stages had differing durations. One can see each of the optimisation stages with APD out-perform the 2008 period with MAD alone.

Table 3 attempts to quantify the increase in energy production. Stage 1, with the APD operating at 38°C and two days retention time shows the largest increase in energy production, at almost 18%.

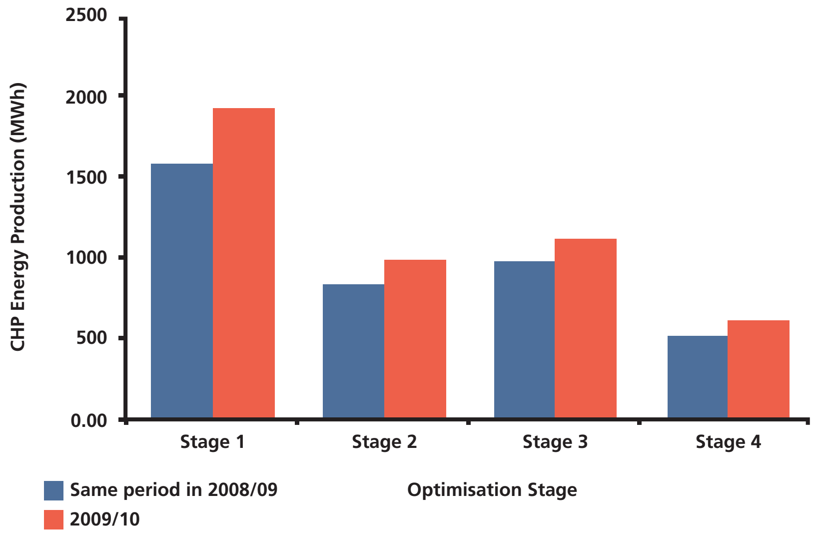

Figure 7 - Energy generation from the CHP engines at Derby STW for each of the optimisation stages, and the corresponding periods in 2008.

Table 4 - Value of energy generated from the three CHP engines at Derby STW pre-APD in 2008 and with APD in 2009, assuming an energy value of £75/MWh.

Stage	MWh	£	MWh	£	% increase in energy production	Value of additional energy(£)
	Measured CHP 2009	Value of energy production 2009	Measured CHP 2008	Value of energy production 2008		
38°C 2d	1940.6	145543	1593.2	119490	17.9	26053
42°C 2d	1000.4	75033	843.0	63225	15.7	11808
38°C 2.5d	1125.4	84402	983.7	73778	12.6	10624
42°C 2.5d	625.0	46875	522.7	39203	16.4	7673
					Total	56157

WATER & ENVIRONMENT

Conclusions

The data gathered Identifies Stage 1 as the optimum configuration of key process variables (retention time and temperature) for parallel operation of APD at Derby STW. Stage 1 shows the highest VSD (50.2%), greatest VFA production in APD (106%) and has resulted in the biggest increase in energy production compared to the same period in 2008 (18%). However, it should be remembered that no allowance has been made for seasonal variance in sludge composition (SAS to primary sludge ratio) when assessing relative stage performance.

The results allow us to quantify the financial impact of APD at Derby STW. Table 3 shows Stage 1 had the highest increase in energy compared to the same period in 2008, and this additional 18% increase equates to an annual value of around £115,000 of energy per annum, at £75/ MWh. On top of this, additional renewable energy production attracts additional Renewable Obligation Credits (ROCs). Extrapolating Stage 1 performance annually, APD could result in additional ROC income of £72,000 per annum.

As well as extra energy recovery, improved VSD results in lower sludge handling costs. The improvement in VSD from 36.5% of the 2006 benchmark to 50.2% during Stage 1 would result in 1,486 fewer tonnes of dry solids to dewater and dispose of per annum (based on average daily solids loading during Stage 1). This equates to around 5,400m^3 of sludge cake at 20% dry solids. Assuming £30 per m^3 for disposal costs this represents a saving of £162,000 per annum.

Additional energy consumption by the plant (pumping and mixing) equates to around £44,000 per annum. Therefore, total cost benefit minus additional operational expenditure (OPEX) results in a net financial benefit of up to £305,000 per annum.

These results show the optimum configuration in parallel mode is two days retention at 38ºC, and that this yields significant additional income from increased gas production and considerable savings in solids disposal costs over conventional anaerobic digestion at Derby STW alone. Further optimisation trials in series mode will determine any additional improvements in performance from a two-reactor system and final financial analysis of the plant can be undertaken. Once all data from parallel and series mode have been collated, key design parameters can be determined to optimize the capital cost of similar plants in the future. For example, providing two days retention time instead of two and a half days results in a reduction in reactor volume of 20%, and large savings in capital expenditure.

The financial benefits of APD are fundamentally linked to energy consumption, increased renewable energy production and reduced lorry movements transporting sludge cake. This will make a significant positive impact on carbon equivalent emissions associated with Derby STW, evidence of the key role such technology can play in the water industry's attempts to reduce its carbon emissions in line with targets.

The plant is currently undergoing testing in series mode. Once these results have been analyzed, a decision can be made as to the optimum configuration, parallel or series. This will be the key process parameter that will allow the future APD plant designs to be rationalized to reduce capital cost while maximizing operational efficiency.

Many thanks to Severn Trent Water for allowing us to share the results above.

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Dependent failure assessment in the development of a defuelling facility for nuclear submarines

Chris Marriott

Senior Reliability Safety Engineer

Defence

Peter Bate

Safety Manager

South Head Engineering Services Ltd

Abstract

This paper describes how dependent (i.e. common cause and common mode) failures have been accounted for in the development of a safety case for the 'defuelling' of nuclear submarines. A brief description of the submarines, the defuelling process and the safety case is given, followed by a detailed description of the assessment of dependent failures using the Unified Partial Model. The results of the assessments were fed back into the deterministic and probabilistic models of the defuelling process. Cut-off values were applied to multiplicative claims in the deterministic model, whilst beta factors were applied to equipment reliability claims in the probabilistic model. Beta factors have also been used in the derivation of the Probability of Failure on Demand (PFD) and associated Safety Integrity Levels (SILs) for the electrical protection system. Finally, an assessment of alternative methods of calculating beta factors was made and the results were found to be comparable, although the UPM proved to be the most flexible and transparent technique.

Introduction

Dependent failure analysis has been carried out as part of the safety case for a new submarine defuelling facility at Devonport Royal Dockyard Limited (DRDL). The new facility is required in order to defuel Swiftsure and Trafalgar class submarines, once they are decommissioned, prior to their eventual disposal.

The submarines

The Swiftsure and Trafalgar class are nuclear powered attack submarines (SSN), designed to stay underwater, undetected for long periods of time. Both are powered by the same Pressurised Water Reactor (PWR) technology.

The first Swiftsure class submarine, HMS Swiftsure, was launched in 1971 and was followed by five others, culminating in HMS Splendid in 1979. The Trafalgar class is a refinement of the Swiftsure class and was designed six years later. The first Trafalgar class submarine was launched in 1981 and the last, HMS Triumph, in 1991. The internal layout is almost identical to the Swiftsure class, although it is 2.5 metres longer. The Astute class will eventually replace both the Trafalgar class and Swiftsure class.

Pressurised water reactors

Pressurised Water Reactors (PWRs) are nuclear power reactors that use ordinary water under high pressure as coolant to remove heat generated by a nuclear fission chain reaction from nuclear fuel, and as the moderator (The moderator thermalises the neutron flux so that it interacts with the nuclear fuel) to maintain the chain reaction. PWRs are the most common type of power producing nuclear reactor and are widely used in power stations, ships and submarines. In Figure 1, nuclear fuel in the reactor vessel is engaged in a fission chain reaction, which produces heat, heating the water in the primary coolant loop. The coolant loop is kept under high pressure by the pressuriser so that it remains liquid at high temperatures (High pressure prevents the water from reaching film boiling. In film boiling, water below the surface vaporises to create a gas layer, thus reducing the efficiency of heat transfer). The hot coolant is then pumped through a steam generator (heat exchanger) where heat is transferred to a secondary coolant. This coolant evaporates to pressurised steam and is fed into a steam turbine, which then drives the propeller shaft through a speed reduction gearbox. The secondary coolant is then cooled down and condensed, before being returned to the steam generator. The transfer of heat is achieved without mixing the two fluids, which is desirable since the primary coolant will be contaminated.

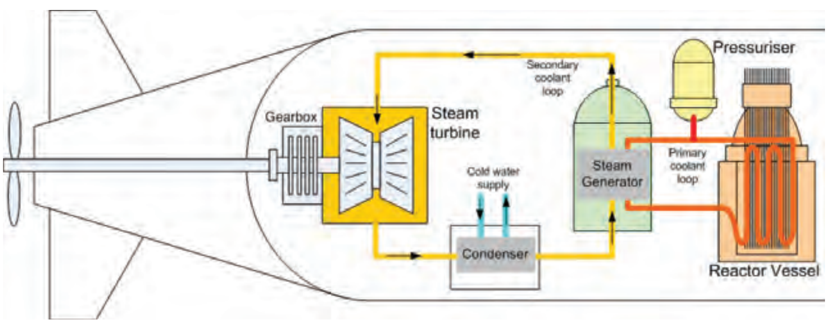

Figure 1 - Submarine nuclear propulsion

DEFENCE

The defuelling process

Defuelling is the process of removal of spent reactor fuel modules using a mobile Reactor Access House (RAH). After cutting a hole in the submarine pressure hull, the RAH is moved into position above the reactor compartment of the docked submarine. The RAH is connected to the submarine via the coaming which is sealed to the hull to create a barrier against the release of contaminants, and to protect defuelling equipment from the environment. The RAH is a large, seismically qualified structure supported on bridge beams across the dock, see Figure 2. It incorporates a Goliath crane with separate 37 tonne and 2 tonne hoists, a common long travel arrangement, a high integrity, dual channel electrical protection system and low level transfer trolley for transferring plant and equipment to and from the RAH. The defuelling process, which takes place over many months, is the process of using the RAH to remove the spent reactor fuel modules and other irradiated and contaminated equipment through the hole in the submarine hull.

The safety case

A safety case has been developed for defuelling using the RAH. The principal safety issues relate to the movement of irradiated items. The development of the RAH and its associated safety case has followed DRDL procedures and these are in compliance with the legislative requirements appropriate to a Nuclear Licensed Site. The process began with an extensive hazard identification exercise of the proposed design, which led to the development of Safety Functional Requirements (SFRs) for each of the safety systems making up the RAH. In accordance with International Atomic Energy Authority guidance[1] each of these SFRs have been categorised to reflect their significance in achieving or maintaining nuclear safety, based on the unmitigated consequences of the fault condition. Each SFR is then reviewed to identify the Structure, System or Component (SSC) associated with fulfilling the safety function.

A deterministic safety assessment was carried out to ensure that, for each of the significant hazards identified (i.e. in the event of an unmitigated design basis accident. A design basis accident is an accident which the facility must be designed to withstand without unacceptable radiological consequences by virtue of its inherent design characteristics or its fault responsive systems), appropriate safety measures were in place. Each of the safety measures is associated with a certain amount of risk reduction, and multiple safety measures may need to be claimed in order to meet the risk reduction target for the specific Postulated Initiating Event (PIE) frequency and consequence combination.

The association of hazard, PIE, safety measures and consequence can be depicted graphically as a 'bowtie'[2], as shown in Figure 3 for a 'Snagged Load' event.

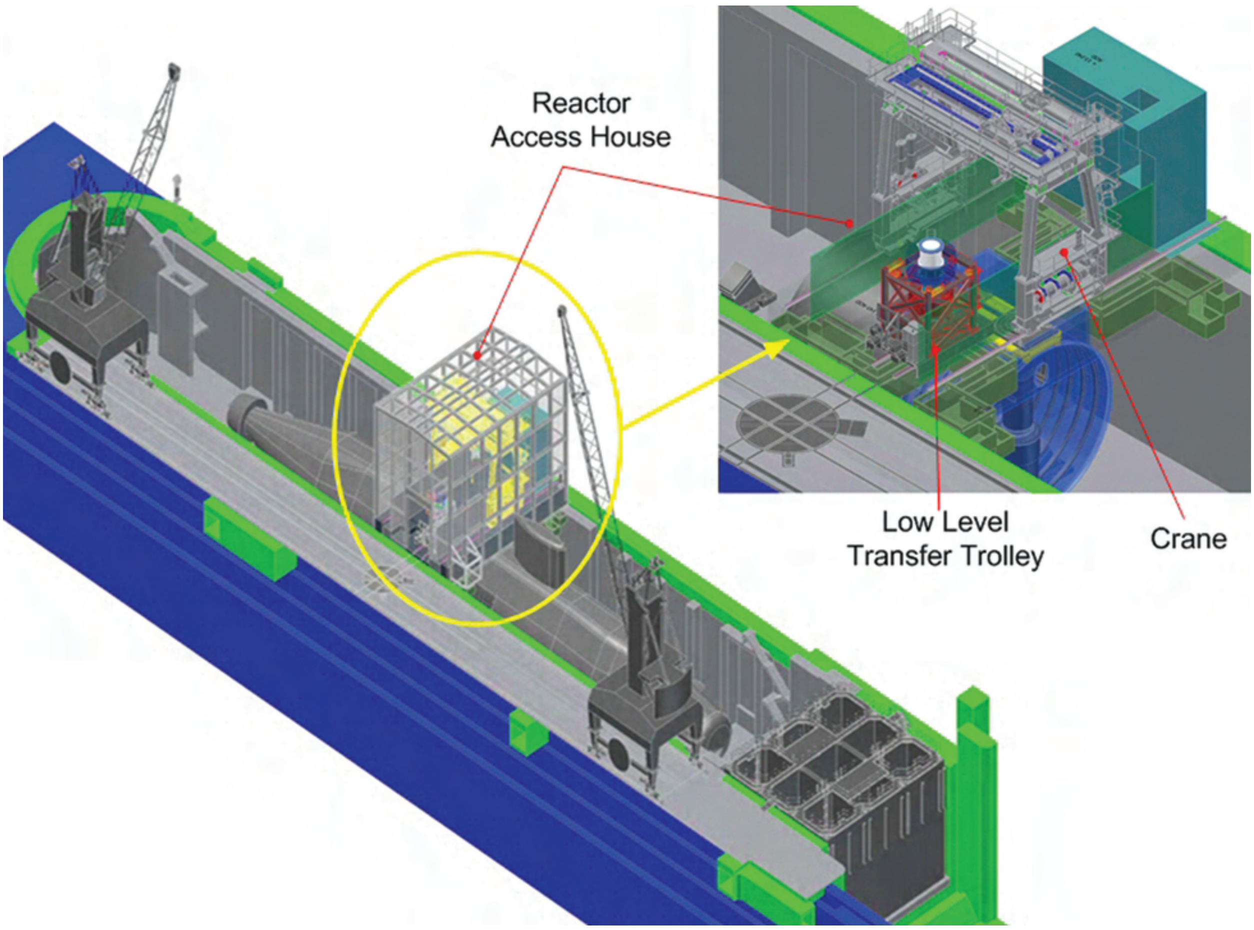

Figure 2 - The defuel facility.

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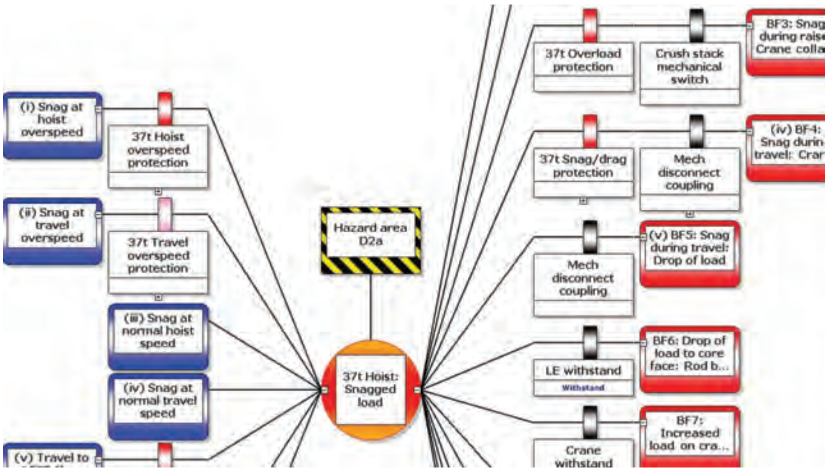

Figure 3 - Bowtie for 'Snagged Load'. Note that these (and subsequent) figures are for illustration only and do not necessarily reflect the final design of the system.

In parallel with the deterministic safety assessment, fault and event tree models of the proposed design were developed in order to provide a more quantifiable assessment. This 'Probabilistic Safety Assessment (PSA)' was used to ascertain whether the design would meet relevant safety targets for the site. These targets relate to possible on-site accidents and are used as part of the judgement as to whether radiological hazards are being controlled adequately and risks being reduced to as low as reasonably practicable. The probabilistic safety assessment covers all of the aspects relating to the risk assessment process, starting from the full list of initiating events provided by the fault schedule, and analysing the fault sequences using fault tree and event tree techniques to calculate the risk.

For each of the safety functional requirements which are implemented by electrical protection systems, there are associated Safety Integrity Levels (SILs) assigned. These range from SIL 1 to SIL 4, where SIL 1 represents the lowest integrity requirement and SIL 4 the highest integrity requirement[3]. The SIL sets a reliability requirement, or the Probability of Failure on Demand (PFD), which must be met by the components making up each functional element of the system.

The deterministic and probabilistic safety assessments together provide the safety justification for the design of the system and are thus a significant part of the overall safety case. Both assessments must take account of the possibility of dependent (i.e. common cause and common mode) failures as follows:

- Deterministic assessment: Wherever there is a multiplication of safety measure claims, there is the potential for dependent failures. A 'cut-off' value must be defined to account for these multiplicative claims.
- Probabilistic assessment: Where multiple similar components are used as components of a safety system, there is the potential for dependent failures. These dependent failures are modelled explicitly in the fault and event trees through the application of a 'modifying factor' which limits the overall claim on redundant items.
- The PFD calculations, which define the SIL for each safety function, must also include a 'modifying factor' to account for dependent failures.

Dependent failures

The term 'dependent failure' covers all definitions of failures that are not independent and, as used in this paper, encompasses both Common Cause Failure (CCF) and Common Mode Failure (CMF). The following formal definitions are reproduced from[4] for dependent, common cause and common mode failures:

- Dependent Failure: The failure of a set of events, the probability of which cannot be expressed as the simple product of unconditional failure probabilities of the individual events.
- Common Cause Failure: This is a specific type of dependent failure where simultaneous (or near simultaneous) multiple failures result from a single shared cause.
- Common Mode Failure: This term is reserved for common cause failures in which multiple equipment items fail in the same mode.

An example of a common cause failure is an over-voltage from a common power supply. For a common mode failure, this same over-voltage could lead, for example, to a system failure due to multiple relay contacts failing in the closed position. Common mode failures are therefore a subset of common cause failures.

Dependent failure can be a significant proportion of a system's failure probability. A study in the 1970s[5] presented evidence that dependent failures are significant and demanded that the design and operation of redundancy systems must include a concerted approach against them. The study indicated that design errors and maintenance errors are the most significant causes of dependent failure.

Guidance published in the wake of this report[6] sought to provide qualitative advice on the design and operation of redundancy systems. This included a detailed list of defensive actions ranked according to their assessed significance in the reduction of dependent failure frequency. It was recommended that functional and equipment diversity 'must be exploited to their ultimate' for high reliability systems. All other defences must be exploited to an appropriate degree; the greatest gains would be made in the elimination of design and maintenance errors, as highlighted above, and the adoption of fail-safe characteristics. Further, an independent reliability assessment was considered to be a significant aid to satisfactorily verifying that each defence was adequately applied. The guidance also presented an interpretation of the limited amount of data to give quantitative guidance at the system level to designers.

Methodology overview

The method of assessment is based on the Unified Partial Model (UPM), which is described in detail in SRD-R-13[7] and thus only covered briefly here. The UPM provides a framework for carrying out a structured assessment of the susceptibility of a system to dependent failures in an auditable manner. It has the advantage of allowing a derivation of a Partial Cut-Off Limit or a Partial Beta Factor, using a single working structure. The Partial Cut-Off method (The word 'partial' refers to fact that the cut-off / beta factor has been broken down into a number of constituents against which judgements can be made) can be used as a holistic approach, which involves assessment of the system level features (providing input to the deterministic safety assessment), and the Partial Beta Factor method is applied at the component level (providing input to the probabilistic safety assessment). An overview of the methodology is shown in Figure 4.

Human factors

Human factors assessments provided error probability data for input into both of the safety assessments, in particular the probabilistic assessment. Fault trees assume independence between failures in separate legs of the fault tree, i.e. that a failure in one task does not influence the likelihood of failure in another. Human error, however, is particularly prone to a number of potential dependencies that would challenge the assumption of independence. Many human errors are also sensitive to the outcome of a previous action, for example, the failure to reinstate a system following maintenance may directly affect the ability of an operator subsequently to detect and respond to an abnormal state. Consequently, human error dependency is considered explicitly and the modelling approach ensures that the calculated error probabilities account for any dependencies, thus they are not considered further in this paper.

Random versus systematic failures

Random failures are assumed to occur at any time for any component of the system. Systematic failures are those due, for example, to design or specification mistakes. There is an underlying assumption within a PSA that the failure rate for a component is constant. Confidence in the PSA will increase as the validity of a constant failure rate assumption increases. In turn, confidence in the assumption of constant failure rate will increase as the dominance of random failures over systematic failures increases. As such, the validity of and confidence in a PSA will increase as the system in place for preventing and minimising systematic failures improves. This is achieved by ensuring that design conditions are met through proper application of industry standards, engineering design standards and good practice, i.e. a Safety Management System (SMS). The better the SMS, the more confidence that design conditions are met, and the lesser the influence of systematic compared with random failures.

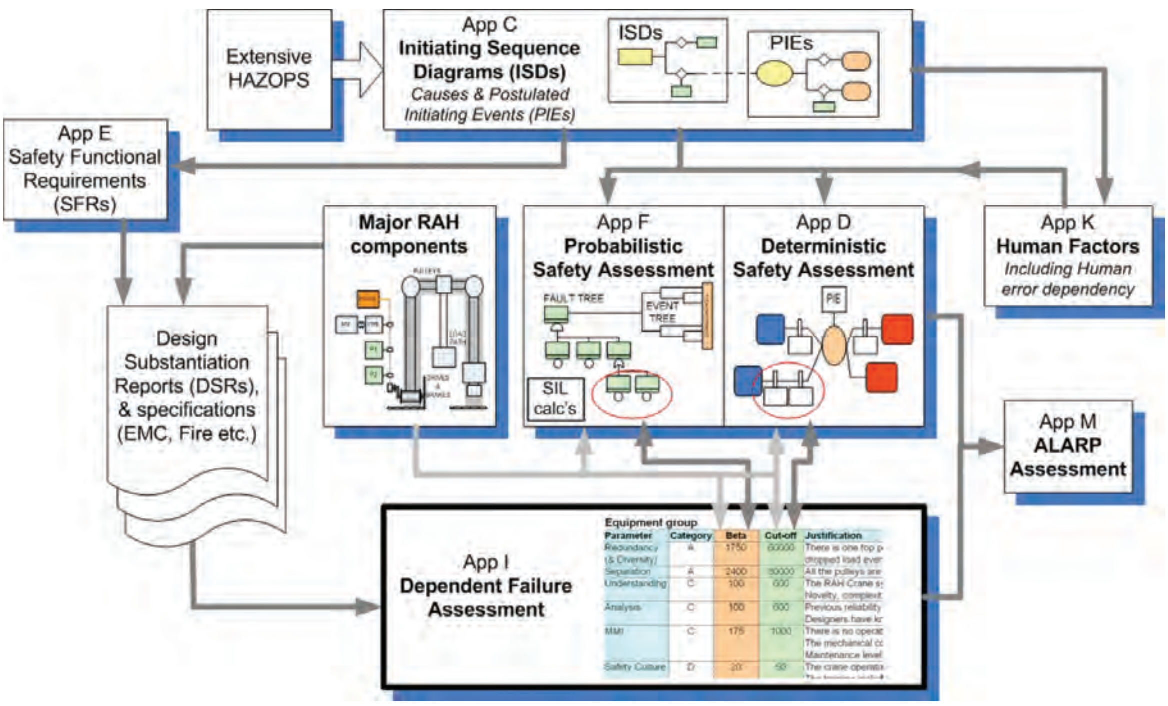

Figure 4 - Methodology overview

An effective SMS will help to minimise systematic dependent failures to a degree where residual dependent failures are more likely to be dominated by random effects. They are then amenable to effective analysis by methods like the UPM. In this respect, the SMS will influence both the validity of the UPM for analysing dependent failures, and the judgements made within the UPM. The better the engineering and management arrangements are within the SMS for minimising dependent failure, the more scope there is for making more optimistic judgements.

The UPM method

The UPM assessment method investigates a number of factors to determine how vulnerable the system or component is to the potential causes of dependent failures identified above. These factors break down into three main categories: Design, Operation and Environment. The methodology further identifies eight sub-factors that need to be considered in determining the ability of a component group to operate as required within the overall system and operating environment.

- Design
    - Redundancy (Diversity) - Design documentation
    - Separation - Design documentation and site survey
    - Understanding - Design documentation including function specification of requirements
    - Analysis - Design documentation and Failure Mode and Effects Analysis
- Operation
    - Man Machine Interface - Design documentation and site survey
    - Safety Culture - Procedures and operator training
- Environment
    - Control - Operating procedures and access to systems
    - Tests - Design documentation and operational experience

The judgement is further extended through the use of 'Feature Factors'. This is an extension of the judgement table which allows assessment between different areas of one system (for example, separation may be good in some areas but poor in others). An overall judgement of the sub-factor is then applied based on an 'average' of the feature factors.

In this way, the reasoning behind each element of the judgement is clear.

DFG S02-1: Loadcells (inc. amplifiers) P1 & P2

Parameter	Cat	Beta	Cut-off	Feature Factors	Justification
Redundancy (& Diversity)	C	100	600	C	Two load cell types (one for P2 ar manufacturers. Control - Compressive strain gaug P1 - Capacitive load cell from Eile electronics. P2 - Compressive strain gauge lo:
				B	Requires 24v DC supply & EMC c The load cells are located on the s the structure, however, this is robu
Separation	B	580	8000	B	The load cells (control, P1 and P2 separated as much as possible fr segregated by the intervening met
				B	The cabling for each of the chann and the protection channels are al
				B	Cross-over of the cabling does oc minimised and constrained to 90 c
				C	The cables terminate in different c etc., and therefore provide good s
Understanding	A	1750	60000	<10	Strain gauge load cells are mature configuration used in the RAH Cra industry so experience is claimed
				Big	Complexity - Strain gauge technol to provide linearization and conve
				Big	Novelty - Strain gauge technology

Figure 5 - UPM model for load cells. Note that these (and subsequent) figures are for illustration only and do not necessarily reflect the final design of the system

The methodology was applied to key components and systems of the crane protection systems and the results fed back in to the deterministic and probabilistic safety assessments. An example of a typical assessment for load cells is shown in Figure 5.

Application in the deterministic assessment

A procedure known as 'safety class analysis' is used to determine the number of safety measures required, based on the initiating event frequency of the hazard, the unmitigated radiological consequence, and the risk 'tolerability' of the site. In many cases, multiple safety measures are claimed against a single fault or hazard and thus there is a potential for dependent failures. A 'cut-off' value must be defined to account for these multiplicative claims.

The cut-off value for a particular system is derived using the UPM as described above. The schematic in Figure 6 shows how the bowties, UPM results and dependent failure assessment are used to evaluate each safety measure and assign an appropriate cut-off value. If the two safety measures are judged to be sufficiently independent from each other then no further cut-offs are required to limit their multiplicative claim.

Application in the probabilistic assessment

Dependent failures can be modelled explicitly within fault trees by incorporating the common fault directly as an event within the trees. However, this is cumbersome for large models and instead the dependent failures can be incorporated through the beta factor derived above (there are a number of alternative model types, including Multiple Greek Letter, Alpha Factor etc. However, these models require additional data which is not trivial to derive and can add unnecessary complication without necessarily improving accuracy). As an example, consider two safety channels whose unavailability (Q) is 1E-3 (for both channels C1 and C2). If dependent failures are ignored, then the total unavailability is simply QC1 x QC2 = 1E-6, which appears very 'safe'.

However, if the beta factor is, say,

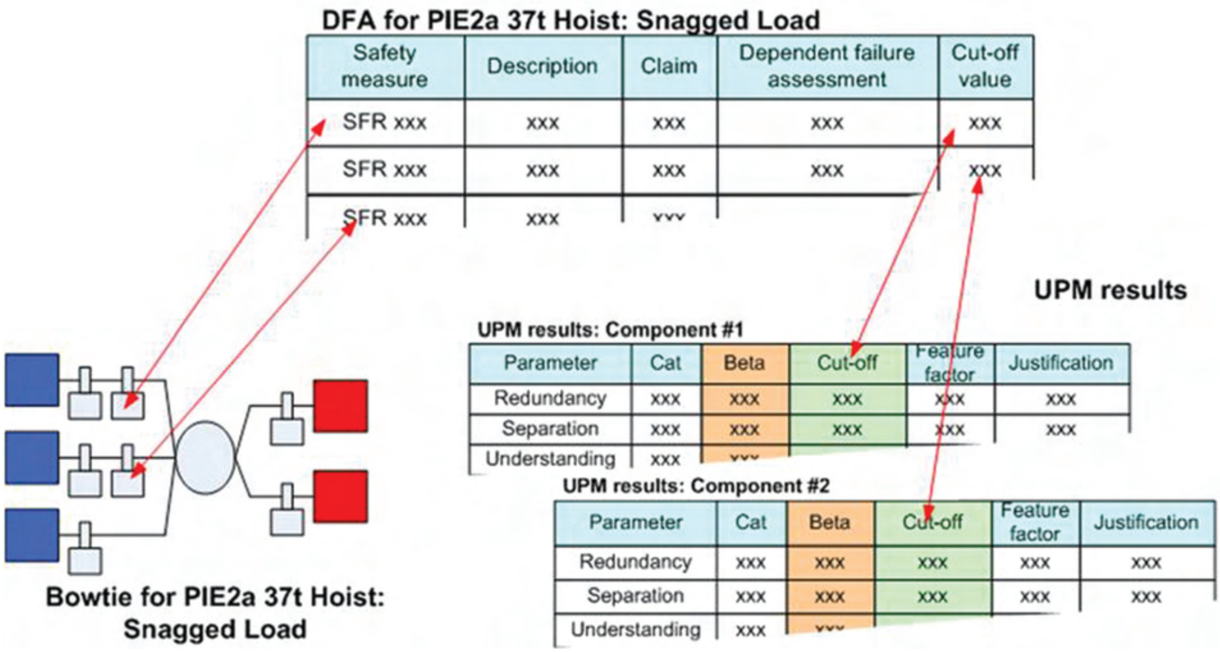

Figure 6 - Dependent failures in deterministic assessment

10%, then the total availability is calculated as 1.01E-4! This gives some indication of the significant effect that dependent failures can have on system reliability (or that over-conservative beta factors can have on the design). Dependency is considered to be one of the major limiting factors for high integrity systems, i.e. SIL 2 and above (another significant limiting factor on system reliability is human error, specifically in the maintenance and testing of SIL systems).

For each of the component groups exhibiting redundancy, a beta factor has been calculated and used to derive a 'CCF' value for the whole component group. These values have subsequently been used in the fault trees within the probabilistic safety assessment, as shown in Figure 7.

Alternative methodologies

Safety instrumented systems which have been designed to meet the requirements of BS EN 61508 require probabilistic evaluation to verify that risk reduction targets have been achieved. The importance of accounting for dependent failures has been recognised in BS EN 61508, which requires that this evaluation also includes a quantitative assessment of dependent failures.

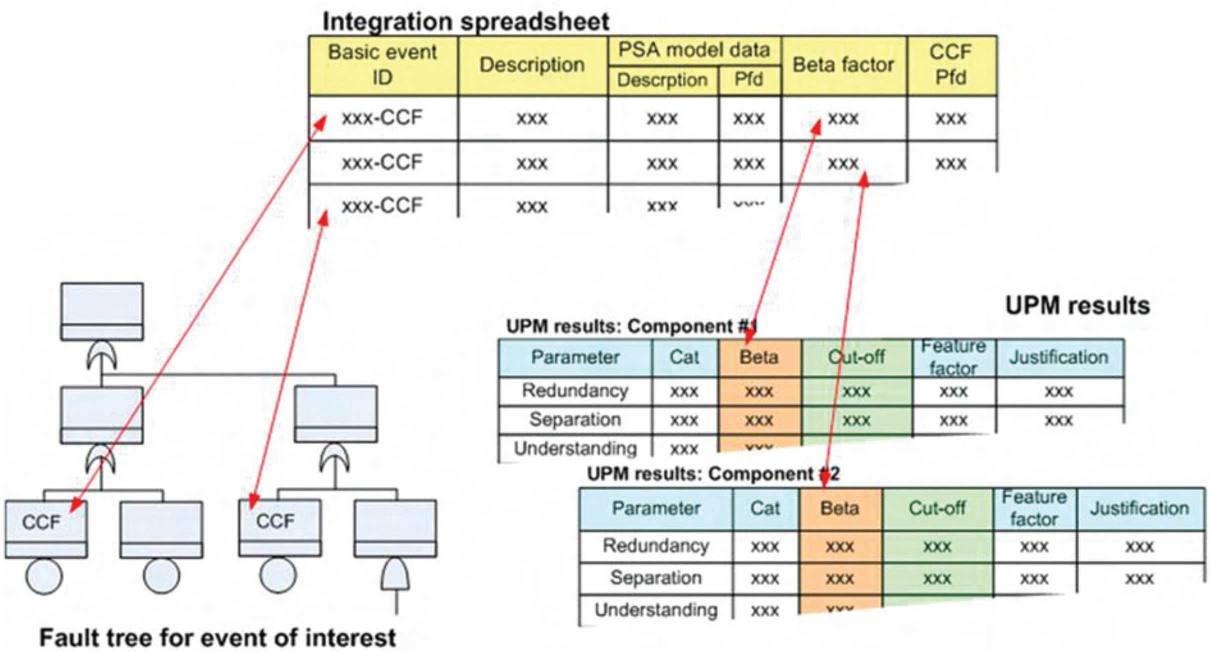

Figure 7 - Dependent failures in probabilistic assessment

An approach for carrying out this quantitative assessment is contained in BS EN 61508 part 6, annex D[8]. The guide suggests three avenues that can be taken to reduce the probability of potentially dangerous dependent failures:

- Reduce the number of random hardware and systematic failures overall
- Maximise the independence of the channels (this reduces the amount of overlap, whilst maintaining their area)
- Reveal non-simultaneous common cause failures while only one, and before a second, channel has been affected, i.e. use diagnostic tests.

The approach in BS EN 61508 part 6 requires that a series of questions be answered, for which points are scored. The total is then summed and compared against a table of values to derive a beta value. The 'Betaplus' method is very similar to the method advocated in BS EN 61508 part 6, if marginally simpler[9]. The Beta factor is calculated explicitly from the summing of individual factors, modified by consideration of the 'diagnostic frequency' (the diagnostic frequency is the interval between proof tests (or auto-tests) on the system. This relates to 'diagnostic coverage', which is an estimate of the proportion of failures which would be detected by a proof test).

Application to protection systems

The design of the two protection channels is such as to segregate and provide independence from the significant effects of faults and hazards. This has been achieved through the application of a number of good practices, including the physical separation of equipment and other process or equipment hazards, segregation and the provision of physical barriers, and arranging the layout to minimise the effects of a fault. Both the BS EN 61508 method and the Betaplus methods were used to derive a beta factor for the sensors/final element aspects of the two protection channels, and the result compared with that obtained using the UPM.

The comparison indicates that results are comparable with all three providing values of between 4.3 and 5% for the beta factor to be applied between the two protection channels. The method advocated in BS EN 61508 part 6 is heavily biased towards programmable electronic systems; indeed much of the reason for applying this method is to take advantage of the allowance for internal diagnostics within such programmable electronic systems. This same argument applies to the Betaplus method. The UPM method is, however, more flexible in its application: it can be applied to both the mechanical and electrical systems, provides more transparency in terms of the explicit sub-factor judgements, and is less demanding on the amount of data required.

Whichever method is used for a particular application, the calculation of a numerical value should not take attention away from the primary aim of the analysis. The primary aim is to highlight vulnerabilities to dependent failure, and provide guidance on where improvements in defences against such failures can be made.

Conclusions

A comprehensive assessment has been made of the potential for dependent failures within a modern submarine defuelling facility. An assessment of dependent failures was carried out using the Unified Partial Method. The results of the assessments were fed back into the deterministic model, providing cut-off values for multiplicative claims within the model. Similarly, beta factors have been applied to equipment reliability claims within the probabilistic model. Beta factors have also been used in the derivation of PFDs, and associated SILs, for the electrical protection system. Finally, an assessment of alternative methods of calculating beta factors was made and the results were comparable, although the UPM proved to be the most flexible and transparent technique.

Acknowledgements

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Gas migration risk at underground gas storage sites

Garth Raybould
Associate Director
Water & Environment

Abstract

The UK needs to develop more underground gas storage space to counter the effects of international politics and price fluctuations, but permit applications are being rejected because developers fail to assess the risks of gas escape and migration. Depleted field operators claim that reservoirs that have held gas under pressure for millions of years cannot leak; salt cavern operators say that salt is impermeable and self-sealing. These claims ignore genuine risks and actual incidents of gas escape. Depleted fields can leak because of overfilling and/or overpressuring and migration pathways may exist as a result of fracturing associated with hydrocarbon extraction. Salt caverns can leak through fractures caused by pressure and temperature cycling, especially in non-salt interbeds. Leaks can also occur from injection wells. Leaked gas can migrate through a variety of geological and man-made pathways to present a hazard to sensitive receptors. Detailed geological investigation is necessary to characterise and mitigate the risks.

Introduction

Underground storage of natural gas has been practised in the USA and Europe for half a century but is still comparatively novel in the UK. To date, only five facilities have become operational and about a dozen schemes are at various stages of planning or construction. The existing facilities are at Hornsea (Yorkshire), Rough (an offshore facility off the coast of Yorkshire), Hatfield Moors (Yorkshire), Hole House (Cheshire) and Humbly Grove (Hampshire). In addition, there are storage caverns for liquid petroleum gas (LPG) and other hazardous gases in Teesside, dating from 1959.

There is both a market demand and government pressure for more storage capacity to be created, in order to mitigate the impact of fluctuations in supply as the UK becomes increasingly dependent on imported gas. The majority of new storage capacity is likely to be underground, since underground facilities can have much larger capacities than above-ground tank farms and in principle can be operated more safely.

Even so, one of the most significant hazards associated with any pressurised gas storage facility must be the potential for gas escape. In the case of an underground facility the potential for gas migration through the ground to sensitive locations is a fundamentally important issue in the assessment of environmental impacts. However, in the UK to date, permit applications for underground gas storage (UGS) facilities have paid little or no attention to fugitive gas in the ground. Generally speaking this is not because of a lack of understanding of gas migration but because of an apparent perception among developers that gas cannot escape in the first place. Superficially, the reasons for this perception are understandable and are related to the nature of the rock formations in which the gas is stored.

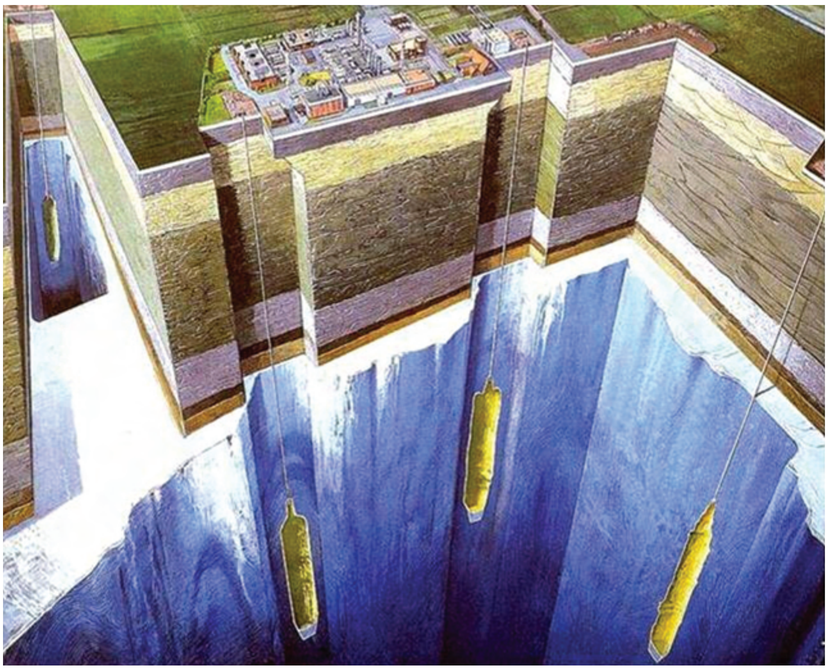


Figure 1 - Salt caverns

SUSTAINABILITY

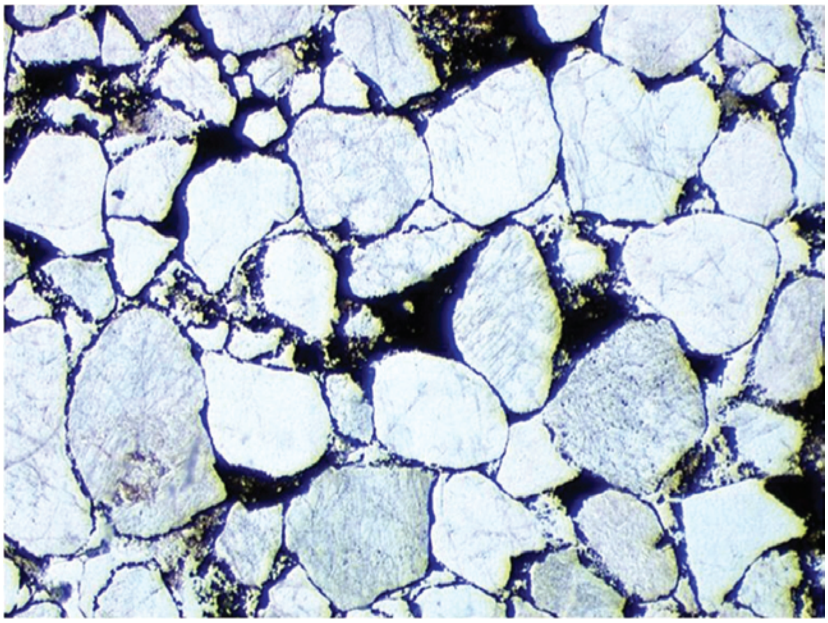


Figure 2 - Porous rock

UGS facilities are of three main types: salt caverns, depleted oil and gas fields and aquifers. Salt caverns are purpose-built voids, Figure 1, usually created by solution mining, in bedded halite (as at Hole House in Cheshire) or salt domes. Depleted field facilities are developed in rocks from which oil and/or gas has been extracted, making use of the vacated pore spaces, Figure 2, for the storage of injected gas (Rough, Hatfield Moors and Humbly Grove). Aquifer storage is similar in that it uses the pore spaces of porous rocks but the injected gas displaces groundwater directly. An individual salt cavern, particularly in a salt dome, can have a capacity of several million cubic metres. Existing depleted field and aquifer facilities (worldwide) have capacities up to two billion cubic metres.

The factors which reduce, or are perceived to reduce, the likelihood of gas escape from the storage formation itself are as follows:

- In the case of salt caverns, the extremely low permeability of the salt, coupled with a self-annealing property which minimises the prospect of gas escape along fractures
- In the case of depleted fields, the fact that the host rock has previously held gas and/or oil under pressure in natural circumstances, usually beneath a low-permeability cap rock
- In the case of aquifers, the pressure of the surrounding groundwater.

There is no gas storage in aquifers in the UK at present. Although several depleted hydrocarbon reservoirs have been or are being developed for gas storage, few have come before the planning authorities specifically as storage facilities because the re-injection of gas has been considered an extension of enhanced oil recovery (EOR) procedures, permitted by the original authorisations for oil production. Consequently, few depleted field storage proposals have been subjected to environmental impact assessment.

Gas storage proposals that have gone through the planning process are for either salt cavern storage or depleted gas field storage (where EOR would not have applied). In all cases, there has been an implicit assumption that gas migration would not be an issue because of the natural containment characteristics of the storage formations and associated rocks, and gas migration risks have not been assessed at the application stage. The following are some typical extracts from planning applications, environmental statements and supplementary documents:

"Given [the developer's] view that there is no potential for gas migration [from salt caverns], no environmental effects are likely to arise. Accordingly, there is no need for any assessment in the Environmental Statement of possible effects associated with gas migration. Nor is there any need to assess or consider the means to ameliorate any adverse impacts associated with gas migration."

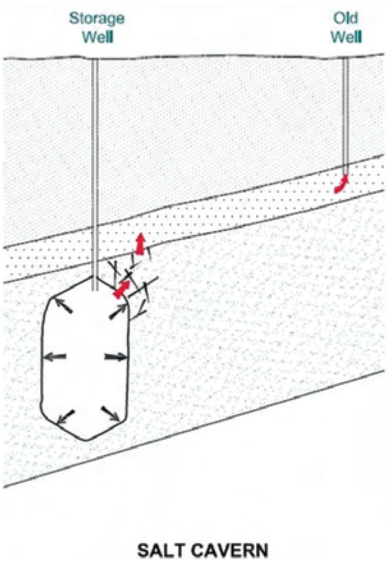
"As long as the initial reservoir pressure is not exceeded, then there is no reason to believe that gas will leak through the formations that originally trapped the oil and gas that has already been produced… but this is for the HSE and other bodies to monitor and appraise."

"The existence of an oilfield attests to the geological integrity of the structure and capability to retain hydrocarbons over significant periods (millions of years). Queries and objections relating to the safety and integrity of any existing or future infrastructure should not detract from this fact and bring into question the geological integrity of the…oilfield structure."

In other cases there is no mention at all of the possibility of fugitive gas, either below or above ground. Geological appraisal in environmental statements is frequently limited to an assessment of the potential impact of the development on geological conservation areas, or the impact of groundwater, contaminated land, waste disposal sites or unstable ground on the proposed development.

However, both theory and experience show that gas can escape from storage facilities into the surrounding ground and migrate considerable distances. This paper reviews the potential for gas escape and migration, the nature of gas migration pathways, the possible consequences of migration, approaches to risk assessment and possible mitigation procedures.

It should be noted that although some storage proposals have been permitted through the planning process under the Planning Acts, future projects of significant size are likely to be dealt with by the Infrastructure Planning Commission (IPC). For smaller projects, some storage operators may pursue storage



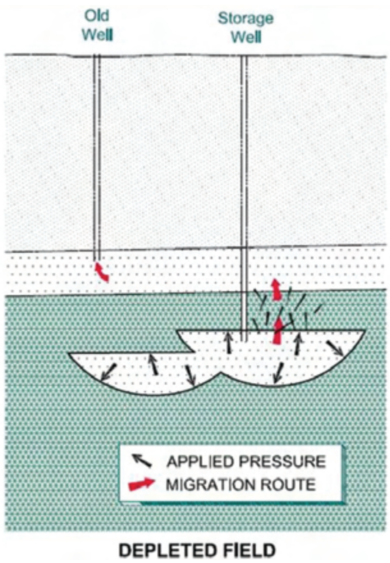


Figure 3 - Risk of over-pressuring

proposals under the provisions of the Gas Act 1965. This act provides for operators to apply directly to the relevant Secretary of State for authorisation to store gas in porous formations. Both processes could involve public inquiry procedures and possibly even parliamentary procedures but they are considered to be less vulnerable to local politics.

Gas escape from storage formations

In this paper the term 'storage formation' refers to the specific geological structure within which the gas is stored, namely the salt bed, depleted reservoir or aquifer, and not the associated pipework and other infrastructure. Gas escape from pipework is an important factor and is considered in a later section; this section describes the factors which can affect the natural containment properties of the storage rocks themselves.

It is useful to bear in mind that, whatever the storage formation, gas is stored under pressure. The higher the storage pressure the more gas that can be stored in a given volume, but storage pressure is limited by the geostatic pressure, that is, the pressure due to depth below ground. It follows that, in general terms, deeper formations can store more gas. Against this is the fact that, in reservoir and aquifer formations, rocks tend to be 'tighter' with depth so that the difficulty (and cost) of gas injection increases, but from a safety point of view deeper formations also have advantages in that they will be surrounded by tighter containment rocks, and migration pathways for fugitive gas will be longer. Consequently, depth as well as thickness is an important attribute of a potential gas storage formation.

The most obvious likely cause of gas escape from an underground facility is over-pressuring, Figure 3. Excessive storage pressure can cause over-filling of a depleted reservoir so that gas migrates beyond the boundaries of the original hydrocarbon trap, Figure 4. Excessive injection pressure can cause stress fracturing of the formation in depleted reservoirs and any form of over-pressuring can fracture salt caverns. Repeated fluctuations in pressure, which are an essential and inevitable part of gas storage, increase the likelihood of stress-related fracturing.

However, what follows relates to potential gas escape and migration even under conditions of apparently acceptable pressures. There is an apparent perception that as long as pressures are properly controlled there will be no escape of gas, but this is not necessarily warranted.

Salt

The attributes of salt which make it suitable for hosting storage caverns are its extremely low permeability and the fact that, through a creep mechanism, it is self-sealing if fractures occur. Ideally, the salt formation should have a uniform character throughout its depth, that is, with no intervening partings or beds of other rocks. The most suitable salt structure is dome or diapiric salt, which can have dimensions of thousands of metres in all directions and can therefore host caverns at considerable distances from other formations or vulnerable receptors. Dome salt hosts the majority of the cavern storage facilities of the USA, but no dome salt is present in the UK.

In bedded salt deposits the presence of intervening beds of other rocks adversely affects the containment qualities of the salt. Even if the rocks have a low primary permeability (through being fine-grained, as in mudstones and shales), they may develop a fracture permeability, and this is particularly likely in areas of folding and faulting. Caverns intersecting non-salt rocks are therefore more likely to leak their contents and the higher the storage pressure, the greater the rate of leakage.

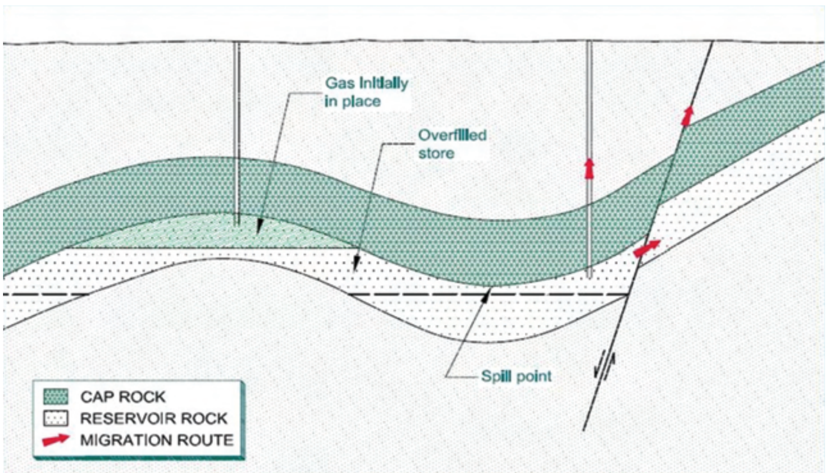


Figure 4 - Risk of over-filling

SUSTAINABILITY

Where non-salt units are continuous over large distances they can provide ready gas migration pathways; where they are not continuous they are less predictable and their effect on cavern design might be difficult to take into account. In either case, non-salt units can cause ledging and other irregularities in cavern construction. The requirement for very detailed characterisation of the salt is therefore clear, particularly in bedded formations.

Even rock-salt itself might not be free of persistent fractures despite its self-sealing properties. Movement on old faults in recent times (including movement induced by mining) could create open spaces even in salt. Re-sealing of spaces might be expected to take of the order of decades or possibly centuries.[1]

One of the advantages of cavern storage for an operator is that, compared with a depleted reservoir, the open nature of the void allows gas to be removed or injected very quickly, so that import or export of gas in response to weather changes or supply factors can be achieved on a daily basis if necessary. However, repeated pressure cycling can induce stresses in the cavern walls. The bigger the difference between the cushion gas pressure (i.e. the minimum pressure required to prevent convergence of the cavern walls) and the working (i.e. maximum) pressure, the more likely it is that such stresses will develop and that fracturing of the walls will result. Moreover, pressure changes are accompanied by temperature changes, which can have an even more deleterious effect on the cavern walls. The potential for microfissuring by these means has been confirmed by seismic monitoring in some cases.

Depleted reservoirs

Depleted reservoirs consist of rocks (usually sandstones or limestones) with sufficiently high porosities to have held oil and/or gas in their pore spaces in significant quantities, and occurring in trap structures which prevented the original hydrocarbon resource from escaping. The most simple model is an anticline in which the reservoir rock is overlain by a low-permeability mudstone or shale known as the cap rock.

Production of oil and gas generally removes only a small proportion of the original hydrocarbon under natural pressures and enhanced recovery procedures normally have to be used to extract higher proportions. For oil production, one such procedure is the injection of gas.

The fact that such structures have held oil and/or gas under naturally high pressures for tens of millions of years understandably leads to the assumption that gas injected after natural hydrocarbon production can also be held under pressure without loss. However, this is not necessarily the case: the hydrocarbon extraction may cause displacement and fissuring of the cap rock and other overlying rocks, particularly if artificial fracturing techniques have been used to enhance extraction from the reservoir; and in any event natural seepage is known to occur from hydrocarbon reservoirs. Examples of subsidence above producing oil and gas fields are well documented and all such events may reduce the ability of the original reservoir formation to contain gas.

One of the production activities that can induce subsidence is hydraulic fracturing, otherwise known as fracing or stimulation. This is a technique in which the flow of oil or gas is increased by injecting a fluid (usually a foam consisting of water, nitrogen, guar gel and a surfactant) into selected horizons from the production well to cause fracturing and thus release hydrocarbons from isolated pores; the foam also carries sand into the fractures to ensure that they remain open throughout the production process. Again, fracturing and subsidence induced by hydrocarbon production are commonly detected by seismic monitoring.

Even without induced fissuring, oil and gas from natural reservoirs can seep to the ground surface, a phenomenon that has been known from ancient times. Indeed, most of the important oil and gas producing areas around the world were first discovered by surface seepages, and leakage to the surface is currently occurring in about half of the 370 basins around the world with known petroleum reserves.

Leakage may be through pores or through fractures in the cap rock, and the rate of leakage will be governed primarily by the intergranular or fracture permeability of the rock and the gas pressure, once the capillary entry pressure of the pore or fracture system has been exceeded. Fracturing is more likely in folded geological terrains than in flat-lying ones.

Where surface seepages are natural and part of local knowledge they are unlikely to constitute a hazard because they can be avoided in the planning of new development, but they are additional evidence that the containment of gas in natural reservoirs cannot be relied on absolutely.

Aquifers

The principle of gas storage in aquifers is that the gas is stored at sufficient depth to be held in place by the pressure of the surrounding groundwater. There are many practical disadvantages to aquifer storage which are not necessarily relevant to the current subject, but it must be apparent that one problem is the need for very careful control of the gas injection pressure so that it is high enough to displace groundwater but not high enough to force gas beyond its intended storage location, or to cause fracturing or the re-opening of existing fractures. Generally, the geological characteristics of aquifer formations are less well known than those of depleted reservoirs and less controllable than salt cavern formations. Inevitably, even with the most thorough prior ground investigation practicable, aquifers do not have the same gas retention capabilities as salt caverns or depleted reservoirs. This means that some of the gas that is injected will escape from the formation and this is already well recognised in the US.

As stated previously, there is no gas storage in aquifers in the UK and in view of growing demands on water resources it is very unlikely that any will be developed or proposed in major onshore aquifers. However, future developments in saline aquifers in coastal or offshore locations are possible.

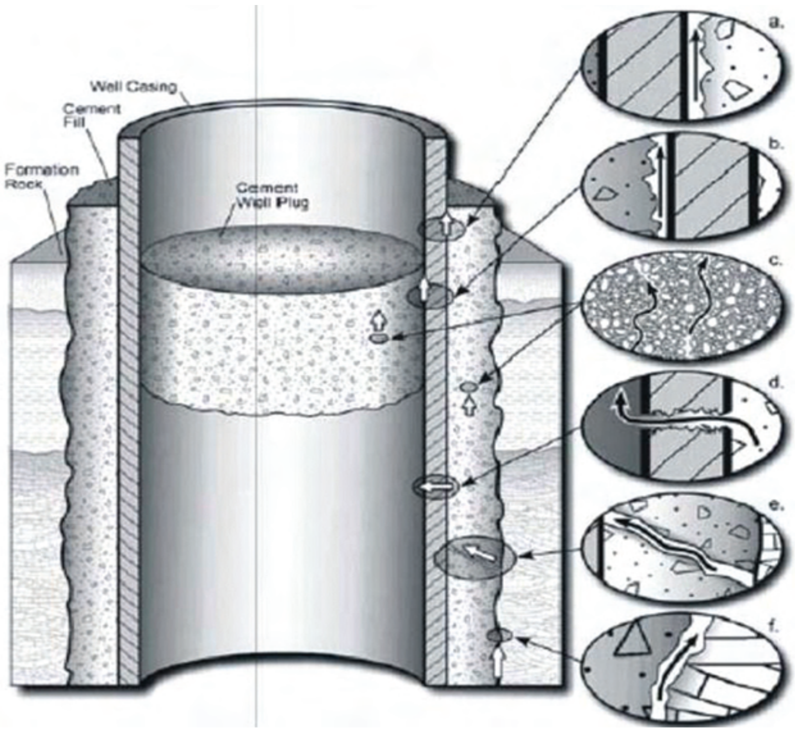


Figure 5 - Well leakage points (after Nelson et al)

Gas escape from pipework and other infrastructure

Wells have many mechanical components and connections and are therefore susceptible to failures of varying degrees that could cause loss of containment. The result is that well failures, whether at storage facilities or in producing fields, are not uncommon. Most known gas escapes from storage facilities have been from wells and related infrastructure rather than directly from the storage reservoirs. Figure 5 illustrates some of the possible leakage routes associated with well casings.[2]

Among the best-known examples of well failure at storage sites are those which took place at Yaggy, Kansas, in 2001 and at Moss Bluff, Texas, in 2004.

Yaggy

In Kansas, natural gas erupted as geysers and found its way into several buildings and other structures in the town of Hutchinson, causing several explosions and two fatalities. These events occurred about three days after a sudden loss of pressure indicated a major loss of gas at the Yaggy natural gas storage facility eight or nine miles away, where gas was stored in caverns in a bedded salt formation.

An investigation undertaken by the Kansas Geological Survey showed that, although the migration of gas was by geological pathways and old wells, the original leak was from the casing of a well just below the top of the salt and 56m above the top of the storage cavern.

Evidence given in a subsequent court case showed that, in converting the facility from liquid hydrocarbon storage to gas storage, the operators had caused significant damage to the original well casing; they had also failed to recognise this from neutron logging of the well, failed to line the well in spite of brine corrosion damage to the casing from the earlier storage operations, and failed to carry out pressure testing prior to gas storage.

Moss Bluff

In Texas, a gas escape and fire occurred at the Moss Bluff storage facility in August 2004. The fire developed in above-ground pipework in the area of the storage caverns. It burned for six and a half days, six billion cubic feet of gas was lost and an area within three miles of the facility had to be evacuated.

Subsequent investigation showed that the gas release and fire resulted from a series of unusual events. First, a breach occurred in a well string inside a cavern at a depth of 1100m. This occurred within ten days of a check that had shown no signs of a separation. The breach allowed gas to enter the above-ground brine piping. An emergency shut-down valve came into operation but the resulting "water hammer" effect caused a breach in the above-ground piping at a point where there had been unexpected internal corrosion. This breach became ignited and escaping gas fuelled the fire. After about 24 hours a valve structure gave way, resulting in a larger gas release, so that the flame height increased from 30m to 300m. At this point the evacuation was ordered. The fire was eventually brought under control only when all the gas from the cavern had been released and burned.

Other cases

In other cases, leakage from wells is known to have been responsible for gas escapes at Montebello, California in 1980, Mont Belvieu, Texas in 1980, 1984 and 1985, Teutschenthal, Germany in 1988 and Magnolia, Louisiana in 2003.

The frequency of well failures at hydrocarbon production and storage fields around the world, whilst not high in relation to the total number of operational well-years, is high enough to make failure amenable to numerical analysis and prediction for risk assessment purposes.

Gas migration pathways

If gas escapes from a reservoir, cavern or well its possible migration routes towards sensitive receptors can include the following:

- Permeable beds and bedding planes
- Cavities in soluble strata (including wet rock-head in salt areas)
- Faults and other fractures
- Mine-workings (used and disused)
- Wells and boreholes (used and disused)
- Utility routes (used and disused), including pipe bedding materials
- Combinations of any of the above

These are generally self-explanatory. The first three are natural geological pathways and indicate the importance of obtaining detailed geological information on the whole succession in the proposed storage area and not simply on the storage formation. The information is likely to come from a combination of surface mapping, drilling, down-hole wireline logging and seismic profiling, to identify details of lithology, porosity, permeability and structure.

Permeable beds

In bedded salt formations, non-salt units that could be intersected by caverns could consist of high-permeability sands or of finer-grained rocks in which a fracture permeability has developed. Fracturing is likely to be more intense in a folded and faulted locality than in a flat-bedded geological terrain. The presence and extent of such permeable units needs to be established as far as possible. In depleted reservoirs it is necessary to identify the structural spill-point (which is not always necessarily identified at the production stage) and whether the permeable reservoir formation continues beyond that point, or if there is a connection to other pathways. In either type of storage formation a high hydrostatic pressure in the permeable unit can restrict gas migration, and in such cases gas containment is a matter of balancing gas pressure against hydrostatic pressure, as in aquifer storage schemes.

Permeable beds can act as migration pathways even if they are not in contact with the storage formation. The migration of gas over a considerable distance from the leaking well at Yaggy was facilitated by the presence of a fractured dolomite bed, which was intersected by the well near the leakage point.

Cavity systems

Extensive cavity systems are a well known feature of some limestone terrains and need to be taken into account where such rocks form part of the overburden to the storage formation. The irregular nature of their development means, of course, that they are notoriously difficult to map and predict.

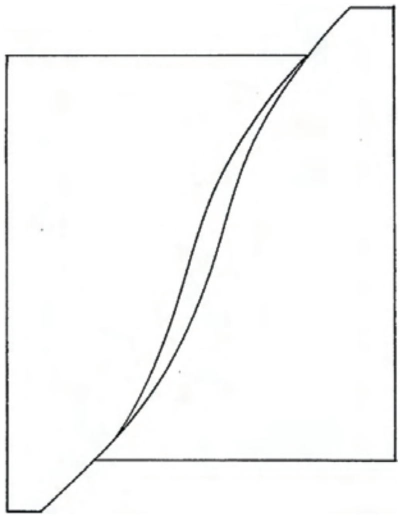


Figure 6 - Theoretical opening on curved fault plan

A particular form of cavity development occurs above some salt units. Rock-salt itself, although very soluble, has an extremely low permeability and groundwater tends to perch above it. Where the overlying rocks themselves contain small proportions of salt, as is often the case, solution of the salt by moving, perched groundwater destabilises the rock and can give rise to an extensive, highly permeable collapse system immediately above the salt formation, sometimes known as wet rock-head. Again, although such systems can be identified by drilling they are extremely difficult to predict or map in detail. Wet rock-head is often wrongly assumed to form only at shallow levels, but in fact it should always be assumed to be present unless there is clear evidence to the contrary.

Faults

Simple geometric considerations suggest that in some cases, especially where they result from lateral tension and have irregular surfaces, fault planes can have open voids, Figure 6, or at least fragmented zones with open space between the fragments. Such faults could be pathways for gas movement and such pathways might exist in very brittle rocks near the earth's surface, where lithostatic pressures are low. Hence, in quarries, cliffs and other surface exposures faults with open pathways might be observed. Faults with such open structures could act as migration paths for any gas that escaped from a shallow storage facility, particularly in an active earthquake area, and a number of such events have been documented.[3]

The presence of faulting may be identified at different scales from seismic profiles, from structural mapping and from drill-core features such as brecciation and slickensiding, and information from all available sources needs to be combined to form as clear a picture as possible of the intensity of faulting and the likely nature of fault planes and associated damage zones.

The capacity of rock-salt to anneal itself following fracturing reduces the potential for gas to escape from salt caverns along faults, but operators still tend to avoid major fault zones for cavern construction, both because of the risk of renewed fault movement in known earthquake zones and because of the possibility of fractured non-salt units, which do not self-anneal. Geomechanical investigation of cavern sites should always take account of non-salt rock failures which could lead to gas migration.

The mapping and modelling of fault architecture in hydrocarbon reservoirs has become a very advanced art, because of the compartmentalising effect of faults and the implications this has for accessing all the productive parts of a reservoir.[4] The use of detailed analytical tools, based on seismic and drilling data, has become common in major oil and gas fields to ensure that production drilling is targeted most effectively. The same tools could be applied to assessing the likelihood of faulted pathways (i.e. either fault continuity or interconnection between faulted permeable beds) in and near gas storage formations, although to date this level of sophistication does not appear to have been applied to storage proposals. There is a significant cost associated with detailed fault modelling and in many cases monitoring (together with defined response procedures) is seen as a more cost-effective approach to risk management.

Whilst there is evidence that faults and associated fractures may act as pathways, they might in fact more frequently be barriers to gas movement. At depth, openings on fault planes are unlikely to exist because of high pressures which, in simple terms, press the walls of the fault together. This is particularly true when the crust is under compression, but even in tensional situations the lithostatic pressure, that is, the sheer weight of overlying rock, will tend to have the same effect. Moreover, the movement of one body of rock against another creates a fault gouge, which can be envisaged as a paste filling in any irregularities between the fault walls. Permeable fault gouge is not unknown, but again it is associated with very brittle rocks. Experience in the oil and gas industry is that even in the permeable rocks that form reservoirs, faults tend to be seals rather than pathways: compartmentalisation is as much to do with fault sealing as with the juxtaposition of high- and low-permeability rocks.

In areas where the stratigraphic succession contains argillites (claystones and related rocks) fault planes are characteristically filled with a very fine fault gouge or clay smear which tends to have a permeability even lower than that of the adjacent rocks. Such low-permeability gouge forms even when the adjacent rocks have only a minor proportion of clay in their composition.[5]

Much of the published work on fault sealing relates to predominantly sandstone sequences because the driver for the research is the identification of seals and barriers within hydrocarbon reservoirs. However, a number of workers have found extremely low permeabilities in clay gouges, together with gas entry pressures notably higher than those likely to be used in most storage facilities.[6,7] Significantly, in respect of the pressure cycling inherent in a gas storage process, Faulkner and Rutter[8] investigated the effect of pressure cycling on gas permeability and found a reduction in permeability after each phase of depressurisation.

In summary, the effect of faulting can be either positive or negative in respect of the safety of gas storage and needs careful assessment for every individual proposal.

Similarly, the effects of reservoir depletion and subsidence are not necessarily predictable and need investigation on a case-by-case basis. Surface subsidence can be identified from precise level surveys of the ground surface or from satellite imagery, but whether the predominant effect on the overburden succession is a reduction of permeability through compaction or an increase through microfissuring requires specific investigation in each case.

Mine-workings

Mine-workings and other man-made tunnels, whether current or disused, are pathways for gas movement in the same way as natural cavities. However, in some cases their locations might be more precisely known than those of natural cavities provided adequate records have been kept. In salt areas the potential interaction of old mines, whether conventional or solution-mined, with new caverns has to be borne in mind. This includes attention to old infrastructure such as connecting pipelines, particularly because the saline environment increases the likelihood of corrosion in all parts of the pipework (as at Yaggy).

Wells and boreholes

Storage wells can be escape points for gas as described earlier, but clearly any well, borehole or shaft, whether used or disused, can be a conduit for escaped gas. The well does not need to connect directly to the storage formation but may simply be a link in a chain of pathways to the surface. In the Yaggy incident, for instance, the final release of gas to the surface in and near the town of Hutchinson was by means of old wells and boreholes that intersected the fractured dolomite bed.

Utility routes

Near-surface utility routes which intersect other possible pathways such as permeable ground can themselves be pathways. Utilities such as water, gas, power and TV lines are frequently laid in trenches in which the bedding or backfill material is highly permeable, creating pathways right up to and sometimes into buildings.

Receptors

The most significant potential consequence of gas escape and migration is fire or explosion. Methane, the principal component of natural gas, is flammable or explosive when its volumetric concentration in air is between about 5% and 15%. Whilst in an open system the ignition of a gas-air mixture will cause a fire, in a confined space it will cause an explosion. Gas is therefore at its most hazardous when it is able to collect at explosive concentrations in spaces such as basements, undercrofts, unventilated buildings, underground utility chambers and the like, particularly when a source of ignition such as a switchbox is present.

Although an explosion is the most catastrophic possible consequence of gas migration, gas is also an asphyxiant, so that people or animals may be at risk even where no ignition source is present.

An additional potential receptor is groundwater. Reports in published literature of groundwater contamination by natural gas are few, and methane itself has a very low solubility, but in theory the dissolution of the more soluble trace components of natural gas could over time lead to water pollution. It may be noted that research on potential CO_2 sequestration in US oilfields has indicated significant changes in groundwater chemistry as a result of CO_2 migration.[9]

Risk assessment

As stated previously, the number of well failures in producing oil and gas fields worldwide is sufficient to allow numerical analysis and prediction of well failure for risk assessment purposes. A rigorous method for identifying and evaluating the potential for such events is a failure modes and effects analysis (FMEA), which identifies possible failure points for a given well construction (casings, liners, packers, cement seals, joints, valves, etc), coupled with a fault tree analysis (FTA) to quantify the likelihood of loss of containment for that construction.

An increasing volume of literature arising from CO_2 sequestration research promotes the use of numerical modelling for deterministic risk and uncertainty analysis in connection with gas storage and migration. Some workers have noted the difficulty of realistically simulating complex geological situations in numerical models, but further work could yield important advances.

Monitoring

Safety cases for underground gas storage schemes usually include proposals for monitoring, particularly of gas pressures and pressure losses, as a precaution against the possible escape mechanisms discussed above. The rationale is that within a short time of an unexpected pressure loss being detected the storage formation can be depressurised or safety valves operated to minimise the loss. This is clearly beneficial to operators, not least from a commercial viewpoint, but is not adequate for environmental and community protection, as the following recent example shows.

Developers suggested that a pressure monitoring system which detected a pressure loss of 0.025% (a typical lower limit for such systems) would be adequate to protect nearby communities against the possibility of gas escape from salt caverns at a depth of 300m and maximum pressure of 70 bar. However, a simple calculation shows that by the time this loss is detected by instruments, from a cavern of 750,000m^3 capacity (i.e. holding 52.5 million standard cubic metres of gas at 70 bar), before any action is taken a quantity of gas equivalent to about 13,000m^3 at standard temperature and pressure could have escaped; diluted by near-surface air to explosive concentrations (say 10% gas in air) this would be 130,000m^3 of explosive gas. Clearly, even more gas can escape by the time the cavern is depressurised or other action taken.

Hence, monitoring cannot be a substitute for adequate investigation, design and operational standards.

Conclusions

More underground gas storage is needed in the UK but permission for new facilities should be dependent on adequate assessment of gas migration risks. The self-sealing property of rock-salt or the longevity of depleted reservoirs does not necessarily mean that facilities in those formations will be without risk, and a full geological and geomechanical appraisal should be carried out in each case.

Gas can escape from storage formations and from pipework and other infrastructure. Escaped gas can follow a network of natural (geological) and man-made pathways to sensitive receptors including built development, natural habitats and groundwater resources. Detailed investigation and modelling of these possibilities should be carried out and the results incorporated into facility designs and operations in order to ensure the safety of persons and property in the vicinity of the geological store.

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Quantification of sustainability principles in bridge projects

Phillippa Spencer
Senior Engineer
Highways & Transportation

Chris Hendy
Head of Bridge Design and Technology
Highways & Transportation

Robert Petty
Engineer
Highways & Transportation

Abstract

Quantifying civil engineering projects in terms of sustainability and meeting carbon reduction targets is a new challenge for the civil engineering industry. Whilst the development of carbon accounting tools identify areas of bridge design and construction that have the greatest carbon emissions, quantifying sustainability overall has been less well studied. The sustainability index for bridges described in this paper is a significant step towards facilitating the systematic quantification of the sustainability of schemes through a simple and graphical tool.

The output identifies where improvements can be made on a design and allows comparison of alternatives. It can be used throughout the design process to monitor decisions, the success of design changes and to inform decisions on future projects thereby improving the sustainability of designs.

The overall sustainability index rating provides a means of enabling targets to be set for the desired sustainability performance of bridges produced by an organisation. The methodology can be adopted by Clients so that, once the key attributes are set and weighted accordingly, designs can be benchmarked across their whole bridge stock.

This paper presents the background to the development of the sustainability index, its key features and examples of its use and benefits.

Introduction

In recent years, engineers have become increasingly aware of the social, environmental and economic impacts of projects or, in other words, the "sustainability" of projects. The most famous definition of sustainable development is that by Brundtland[1] which stated that "Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs". Despite the widespread awareness of sustainability as a concept, there are few means of quantitatively assessing the sustainability of designs in a holistic manner which addresses societal, environmental and economical considerations.

With the introduction of the Climate Change Act and implementation of the mandatory CRC Energy Efficiency Scheme for public and private sector organisations, the focus of addressing climate change through carbon emission reduction has become a dominant aspect of sustainable design.

This has led to the development and use of carbon quantification and footprinting methods in many industries. Whilst quantifying embodied carbon in designs is possible through the numerous carbon calculators that have been developed, such as the Environment Agency calculator[2], quantitative assessments of sustainability as a whole is less developed.

In developing a system that quantifies sustainability, several issues need to be considered, particularly which sustainability principles are most relevant to a specific civil engineering project, how the key criteria are weighted and how the results are reported, interpreted and refined to hone a design. It is relevant to note that future structural design codes will include consideration of sustainability as a core aspect of the design. For example, Model Code 2010[3,4] includes performance requirements for sustainability, but these requirements rely on future development of suitable indexes in order to assess the structures.

This paper presents the development of one such solution for assessing the sustainability of bridge designs in the form of a Sustainability Index for Bridges.

A solution for quantifying the sustainability of bridges

The Sustainability Index for Bridges is a tool that determines an index value for sustainability which can be used in optimising a bridge design and construction. A key feature is its simplicity and ease of use, thus making its use on projects appealing.

The tool provides the following quantitative functions:

- Identification of key issues relating to sustainability, allowing them to be addressed in a systematic way
- Facilitation of design alternatives to be compared in terms of sustainability performance
- Allows chosen designs to be optimised in terms of sustainability performance

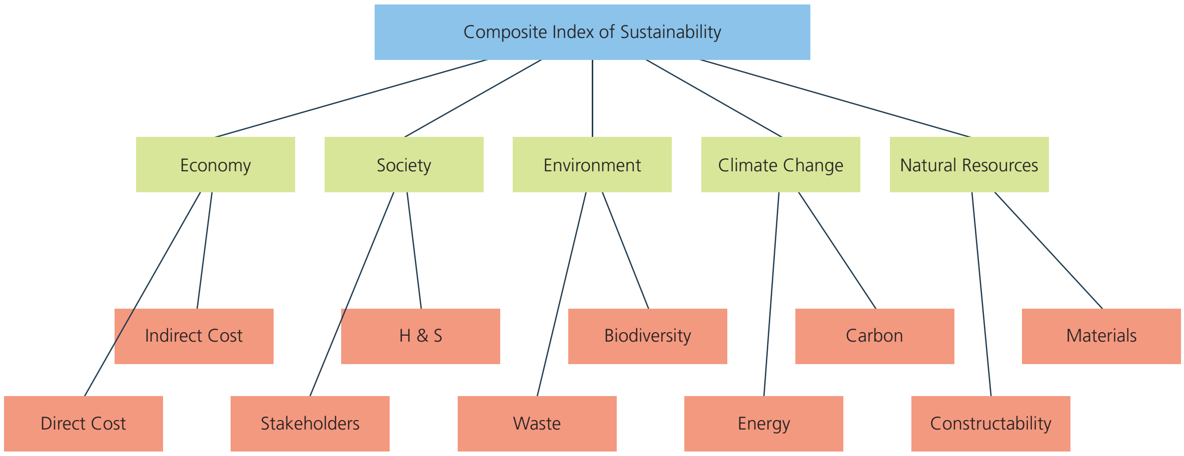


Figure 1 - Initial framework for indicators discussed at workshop

- Production of an overall measure of a bridge project's sustainable performance based on pre-determined criteria, allowing it to be benchmarked against other projects.

Additionally, the Sustainability Index:

- Provides a graphical representation of the project's impacts that can be readily understood by those outside of the project
- Raises the awareness of sustainability issues in bridge projects, which assists in developing engineers' and clients' knowledge and appreciation of the issues involved.

The index aims to inform clients and designers of the impacts of the proposed choice of structural solution and facilitates the evaluation of the scheme against the criteria considered most applicable to the project. The tool is designed to be used at the initial stages of bridge design to select preferred options and then, subsequently, to refine options at detailed design as further details become known. The outline design stage is usually the time when the greatest contribution to improving sustainability can be made and, hence, when the tool can add the greatest value.

Sustainability attributes for bridges

In order to develop the sustainability tool, the most relevant sustainability attributes for the design, construction, operation and removal of a bridge were investigated. The starting point was to identify top-level categories which would address all aspects of sustainability with respect to bridge design and construction. The "three pillars of sustainability", environment, society and economy, formed the basis of the categorisation but "climate change" and "natural resources" were also identified as top-level headings (despite being a sub-set of environment and/or society) because of the particular importance attached to these issues by society at present.

Under each of these five categories, sub-categories were also identified to provide a framework for a workshop aimed at identifying the main potential impacts of bridge projects with respect to sustainability. The workshop involved participants from a technical network of experienced bridge engineers in Atkins across the UK. Figure 1 shows the initial framework used to identify the full list of attributes.

The inclusion of health and safety was discussed at length and eventually excluded because it is addressed through CDM Regulations and Designers Risk Assessments. It was also considered that identifying a score for health and safety would create political difficulties with questions being raised about why one scheme was safer than another and why schemes in general were not 100% safe.

From the workshop, an initial list of 50 potential attributes was refined down to 13. Prioritisation was a democratic process with votes being cast to select the top 13 attributes. There was no pre-determined rationale for selecting 13 attributes; this was considered a manageable number and the value of adding further attributes was considered to diminish rapidly with number. The final attributes under each category were as follows:

- Economy
  - Initial cost, whole life cost, user delay during construction
- Society
  - Aesthetics, user delay during construction
- Environment
  - Extent of loss of habitat, noise during construction, noise during service
- Climate Change
  - Carbon footprint (construction), carbon footprint (maintenance)
- Resources
  - Use of recycled materials, consumption of natural resources, ease of modification/demolition

Other final selections would of course have been possible, but the selection of any consistent set against which all structures could be benchmarked was considered to be the most important criterion.

Development and description of the Bridges Sustainability Index

On final selection of the sustainability attributes, a scoring system was developed to enable performance against each attribute to be rated and assessed. Some attributes, such as those involving costs and embodied carbon, suited a quantitative system. Other attributes, such as aesthetics, suited a qualitative system based on a series of structured questions. The index therefore uses a combination of both systems; sometimes, both are used within the same attribute. Table 1 explains the purpose and significance of each of the sustainability attributes and provides an indication of the assessment method for each; the full list of assessment criteria is too lengthy to provide here. It is acknowledged that such an approach will not be unanimously agreed in terms of the questions used in the assessment or their weightings, but the basic concept of the sustainability index is flexible and the questions can be adjusted over time as necessary to grow the consensus. The most important aspect of the index is that it can be used to assess one or more bridges objectively against a sustainability benchmark value derived from a larger parent group. This in turn helps focus attention on the areas of a design that could be readily refined to improve sustainability.

The sustainability index only covers the bridge itself and assumes it to be necessary. It does not therefore consider the sustainability of alternatives to a bridge (although it can easily be adapted for this purpose), nor the potential benefits of alternative traffic provision across it, the latter being deemed to be fixed. A key aspect when using the index is, therefore, to clearly define a boundary around the structure to delineate the elements that are considered in the assessment. If a drawing is prepared to summarise the results as discussed below, the boundary can be shown on that drawing. Figure 3 shows an example.

The rating of the attributes generates a visual summary graphic and an overall sustainability index score as shown in Figure 2. The graphical plot serves as a convenient summary of the relative impact of each attribute. The overall index score ranges from zero to one; the higher the index, the greater the potential for the bridge design to be improved to give a more overall sustainable performance, but a high score does not mean that the bridge is poor in terms of sustainability. A score of zero corresponds to all points lying at the centre of the spider's web. A score of one corresponds to all points lying on the boundary. The contribution of each individual impact to the overall rating can be weighted. The sustainability index described here gives equal weighting to the five top-level headings of economy, society, environment, climate change and resources.

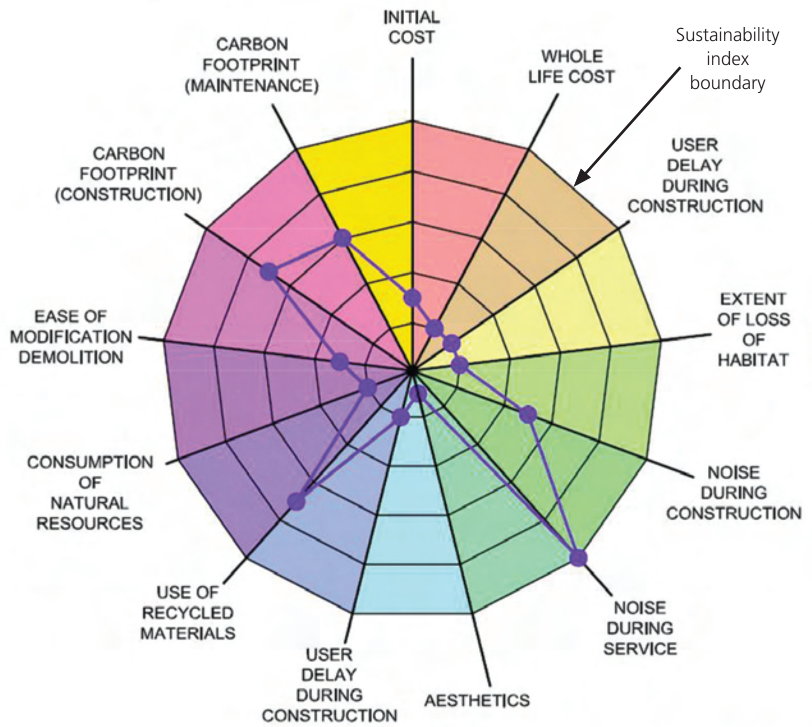
Application of the sustainability index

The index can be used at any stage of project, but it is intended to be completed at:

- Outline design stage
- Detailed design stage
- During construction and at completion

By updating the index at detailed design stage, this allows concrete and steel quantities to be accurately determined and refined, together with the carbon footprint. It is likely that a design driving these elements down will also provide a more economical structure, which in turn, will have an improved index rating.

During construction and upon completion, the sustainability index can be updated with site changes and as-built information to evaluate the impacts of these changes. This will highlight any lessons learnt, whether beneficial or adverse in terms of sustainability, which can then be communicated to other projects.



Overall sustainability rating = 0.427

The smaller and closer the boundary line is to the centre of the circle the greater optimisation to sustainable designed solution.

Figure 2 - Example graphical output of Sustainability Index for Bridges

Table 1 - Individual aim of sustainability attributes

No.	Attribute	Aim of attribute	Method of assessment
Economy			
1	Initial cost	To identify the direct and indirect costs associated with the initial design and construction and to encourage identification of areas where savings on this initial cost can be made.	Cost per square metre of deck area
2	Whole life cost	To identify the maintenance (and potentially demolition) requirements and costs and to facilitate consideration of design modifications which can be made to reduce these demands and achieve a more economical solution for the life of the structure.	Cost per square metre of deck area
3	User delay during construction	To identify the costs associated with significant disruption or diversion to traffic which impact on the local and national economy and to facilitate consideration of methods of construction and design alternatives which reduce the impact.	Structured questions covering length and consequence of delays
Environment			
4	Extent of loss/disruption to habitat	To assess and reduce the 'footprint' of the structure in terms of disruption to habitat, particularly when working adjacent to watercourses.	Structured questions covering loss of habitat and mitigation measures
5	Noise during construction	To assess designs and methods of construction in terms of their impact on the natural environment and local people with respect to noise, and to facilitate consideration of alternatives to minimise this impact.	Structured questions covering types of construction employed, working hours and proximity to residential and business premises
6	Noise during service	To identify mitigation techniques which can be employed to reduce the effect on the surrounding environment of the noise created by the completed bridge when in use.	Structured questions covering anticipated noise and mitigation measures employed
Society			
7	Aesthetics	To identify mitigation techniques which can be employed to reduce the effect on the surrounding environment of the noise created by the completed bridge when in use.	Structured questions covering the importance of aesthetics to the particular scheme and a survey of opinion on the design appearance
8	User delay during construction	To identify and quantify the extent of disruption to the travelling public and to facilitate consideration of methods of construction and design alternatives which reduce the impact.	Structured questions covering length and consequence of delays
Resources			
9	Consumption of natural resources	To assess the consumption of natural resources and facilitate consideration of design and construction amendments to reduce this consumption.	Tonnes of materials per square metre of deck area
10	Use of recycled material	To assess the extent to which recycled materials have been and can be specified. It is acknowledged that there is a potential energy offset with the production of recycled materials, but this is considered in the carbon footprint. It is considered that advocating the use of recycled materials is important in changing behaviour and attitudes towards sustainability.	Structured questions covering percentage of recycled steel, aggregates and cement replacement used, and the appropriateness on the particular project.
11	Ease of modification/demolition	To assess the impact of future modification or demolition of the bridge in terms of complexity and hence financial cost and carbon, and to facilitate consideration of making these activities easier in the future.	Structured questions covering whether specific provision has been made to facilitate modification or removal.
Climate change			
12	Carbon footprint - Construction	To assess the carbon attributable to the design and construction of the bridge and to facilitate identification of areas of the design that can be refined in order to reduce this impact.	Tonnes of CO_2 per square metre of deck area
13	Carbon footprint - Maintenance	To assess the carbon attributable to the whole life maintenance of the bridge and to facilitate identification of areas of the design that can be refined in order to reduce this impact.	Tonnes of CO_2 per square metre of deck area

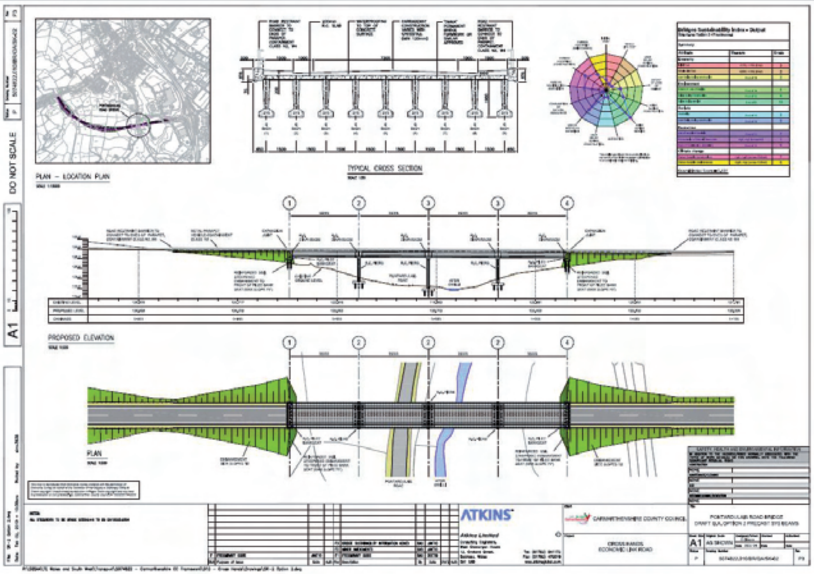


Figure 3 - Example Sustainability Index drawing

The sustainability index is intended to become an integral part of project delivery so that schemes can be compared against a benchmark performance for bridges in general. A template drawing can be created, Figure 3, where key results for each of the attributes are stated and the reasons for any changes noted on the as-built versions. Summarising the impacts on a drawing highlights the impacts of the projects to all concerned and facilitates discussion on what is really important to the project and what can be improved.

Case study - River Roding Crossing - Transport for London

In 2008 Transport for London (TfL) Major Projects commissioned Atkins to produce a structures options report to present the preferred structural form for a new River Roding Crossing – Figure 4 shows the location.

The River Roding structure was required to accommodate buses, cyclists, and pedestrians. The structural options developed had to address several key issues as identified by TfL. These were:

- Clearance and alignment constraints
- Health and Safety
- Aesthetics
- Buildability
- Cost (including whole life cost)
- Carbon footprint
- Future maintenance
- Environmental considerations.

The issues highlighted had a clear synergy with the attributes of the bridges sustainability index and therefore, an assessment of the proposed options for sustainability to demonstrate the preferred solution was undertaken using this tool.

River Roding Crossing design options

Five options were proposed with the final recommendation of a twin tied arch bridge being adopted. Two of the proposed options, and the way they were assessed using the sustainability index, are discussed below: a full width half through girder deck and a twin tied arch bridge as shown in Figures 5 and 6 respectively. The specific aspects discussed are the initial cost, whole life cost, aesthetics, consumption of natural resources and carbon footprint. All other sustainability attributes assessed showed little variation between the two proposed options or were not a key requirement of the client and therefore are not discussed further.

Initial cost and whole life cost

For any structure, the main choice of structural form will have a fundamental effect on the initial and whole life cost. Architectural features with unusual materials may increase the cost of the structure, but may be justified by the visual enhancements they bring. Similarly, some additional initial cost may make the structure more maintainable, thus reducing its whole life cost. The index determines a sustainability rating for both initial cost and whole life cost per square metre of deck area. The sustainability index output for the two options shows that the tied arch structure not only has a considerably higher initial cost than the through girder option, but also that this cost is very high compared to the benchmark from all bridges because the coordinate is right on the boundary of the diagram. This is due to the tied arch having more complex fabrication and erection requirements compared to through girder bridge construction or, indeed, typical plate girder solutions.

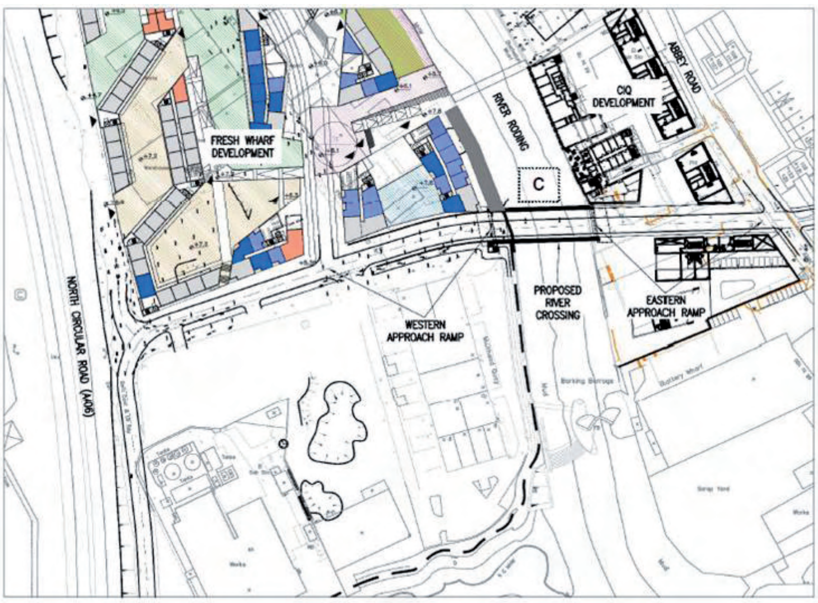


Figure 4 - Site location River Roding crossing

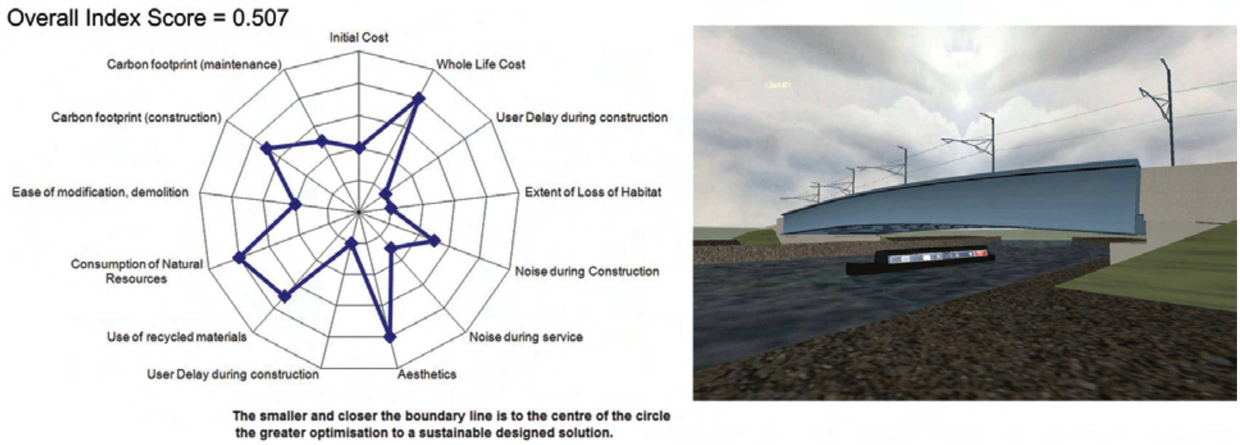


Figure 5 - Full width through girder deck and Sustainability Index Output

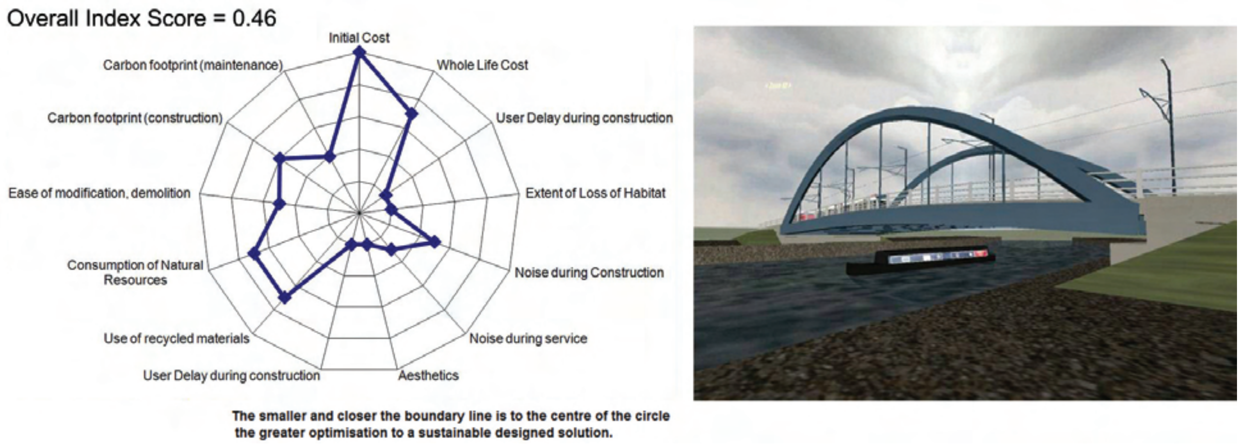


Figure 6 - Twin tied arch deck and Sustainability Index Output

Both options have bearings and expansion joints, requiring regular maintenance. The assessed whole life cost for the through girder option, however, was slightly higher than that for the tied arch due to maintenance requirements of the connections between the main longitudinal beams and the cross beams. These would be more difficult to inspect and maintain, due to confined access and lack of space between the parapets and the main beams, in comparison to the relatively open structural form of the tied arch. In addition, this connection zone would be particularly vulnerable to corrosion due to de-icing salts during its lifetime which increased the assessed potential maintenance costs of this option. Aspects of the tied arch design option, such as arch ribs and hangers, were, however, still considered to require significant maintenance. Difficult maintenance of hangers was mitigated by designing them to be replaceable during continued full operation of the bridge. The use of hangers has an adverse impact on the ease of modification and demolition, which is specifically reflected in the "Ease of modification, demolition".

Aesthetics

Aesthetics was a key aspect in influencing the form of structure for this new regeneration project. Structural choices were reviewed for suitability within the proposed developments, using appropriate materials for its urban location. The bridge needs to fit in with the river front redevelopment and should be appealing to those viewing it.

The sustainability index uses a combination of a quantitative and qualitative assessment to determine the aesthetic merits of the design. The considerations comprising the overall rating include whether aesthetic considerations are relevant at all, the ratio of solid side elevation area to plan area, the blending of the bridge with its surroundings and the views of a sample number of individuals shown illustrations of the bridge.

As shown in Figures 5 and 6, the tied arch had the better sustainability rating for aesthetics. The openness of the tied arch was preferred visually to the deep solid elevation of the through girder option. The latter also significantly reduces the bridge users' visual experience when crossing the bridge compared to the views obtained from the arch bridge.

Consumption of resources and carbon footprint

The index attribute for consumption of natural resources quantifies the tonnage of concrete and steel components of a structure per square metre of deck and benchmarks this against typical existing steel, concrete and steel-concrete composite structures. The through girder option actually contained the greater amount of materials, albeit marginal, and thus the sustainability index tool reports a greater adverse impact on natural resources for the through girder than for the arch.

The increased quantity of construction materials used in the through girder option also has a direct associated impact on the carbon footprint of the structure. However, as well as the materials themselves, the method of construction and plant and labour requirements are also included in the carbon calculation. The sustainability index attribute is then based on the tonnes of CO_2 embodied in the design and construction per square metre of deck area. The carbon footprint was found to be higher for the through girder than for the tie arch.

As noted above, the through girder has an increased whole life cost in comparison to the tied arch which, again, has a direct impact on the carbon footprint for the maintenance of the structure. The through girder has a greater adverse effect in comparison to the tied arch option.

It is worth noting here that the calculation of carbon footprint is open to much interpretation. The Environment Agency's carbon calculator was used for this project but different engineers can make different assumptions in its use, such as those relating to the transport distances of materials or the traffic disruption resulting from the bridge construction. It is therefore vital that a consistent set of assumptions is used when comparing options until such time as a more bridge-specific industry standard carbon calculator is produced.

Figure 7 - River Roding Crossing Tied Twin Arch

Summary

The sustainability index calculated an overall sustainability rating for the two River Roding Crossing options. The ratings are 0.507 and 0.460 for the through girder and tied arch options respectively. The sustainability index rating shows that the tied arch option is more sustainable in comparison to the through girder, given the evaluation criteria incorporated in the tool.

The sustainability index verified the extent to which each of the proposed options impacted on society, the environment and the economy and allowed the client to make a more informed choice on preferred option taking all of these criteria into account. The output demonstrated that initial cost, whole life cost, aesthetics, consumption of natural resources and carbon footprint were the key aspects driving the final choice of structural form and it also identified the areas which could be refined further to improve the overall impact on sustainability for the final design.

Figure 7 shows a virtual reality model of the final option selected for the River Roding Crossing.

Conclusions

Quantifying and assessing civil engineering projects in terms of sustainability and meeting carbon reduction targets is a new challenge for engineers and the civil engineering industry. Whilst the development of carbon accounting tools identifies areas of bridge design and construction that have the greatest contribution to carbon emissions, quantifying sustainability overall has been less well studied. The sustainability index for bridges is a significant step towards facilitating the systematic quantification of the sustainability of schemes through a simple and graphical tool.

The output facilitates identifying where improvements can be made on a particular design and it allows designers and clients to objectively compare alternative design solutions. Further, it can be used throughout the design process to monitor the strength of decisions and the success of changes to the design and to inform decisions on future projects with a view to improving the sustainability of designs.

The overall sustainability index rating provides a useful means of benchmarking designs. Targets can be set for the desired performance for a particular group of bridges on one project or for all the bridges produced by an organisation. The format of the tool is flexible enough to allow different projects to weight attributes differently; in such cases, the benchmarking can obviously only take place within that pool of structures for which the weighting has been altered. The methodology lends itself to adoption by Clients so that, once the key attributes are set, designs can be benchmarked across their whole asset pool of bridges.

Acknowledgements

The authors would like to thank the bridge engineers throughout Atkins who contributed to the development of the Sustainability Index for Bridges. The tool was developed collectively by the Atkins Bridge Engineering Working Group, which links all 41 Atkins offices engaged in bridge design and engineering.

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