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SNC · LAVALIN

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Foreword

Welcome to the seventh edition of our SNC-Lavalin Technical Journal, established to showcase the fantastic depth and breadth of our engineering expertise across a wide range of disciplines and domains and to demonstrate that technical excellence is at the heart of everything we do. This edition highlights the impressive work we have been doing planning and designing new infrastructure, and monitoring and managing existing assets to assure their safety and extend and optimise their service life.

In planning and design, we have performed the master planning and design of the Tilbury2 major port development, dedicated to unaccompanied Roll On-Roll Off (RoRo) freight and bulk aggregate handling which has played an important role in handling RoRo traffic to and from Europe in a post-Brexit UK. On High Speed Two (HS2) in the UK, we have put environmental and inclusivity considerations at the forefront of our designs. We have developed an ecological mitigation and compensation approach which has reduced the time and cost implications involved in licensing development works, and we have optimised the design of toilets on station platforms for wheelchair and other diverse users which is now mandated into the HS2 Inclusive Design Standard for all stations.

In asset management, we have carried out trial replacements and testing of existing hangers on the Humber Suspension Bridge in the UK, enabling the life of the remaining hangers to be extended prior to their eventual replacement. We have also carried out continuous monitoring of the existing hangers of the Great Belt Suspension Bridge in Denmark using digital image correlation (DIC) to determine vibration-induced fatigue loading and hence predict their residual life for optimal interventions. For nuclear reactor piping systems, we have developed and validated software for fitness-for-service assessment to identify risks of sudden fracture under crack propagation. For other assets, we have used a risk-based Targeted Asset Management (TAM) approach to maintain the safe operation of a railway on Chitts Hill embankment within limited available budgets, and we have developed an innovative method to simulate and map dam breach inundation, together with dam risk products to facilitate emergency management of dams throughout the U.S. state of North Carolina.

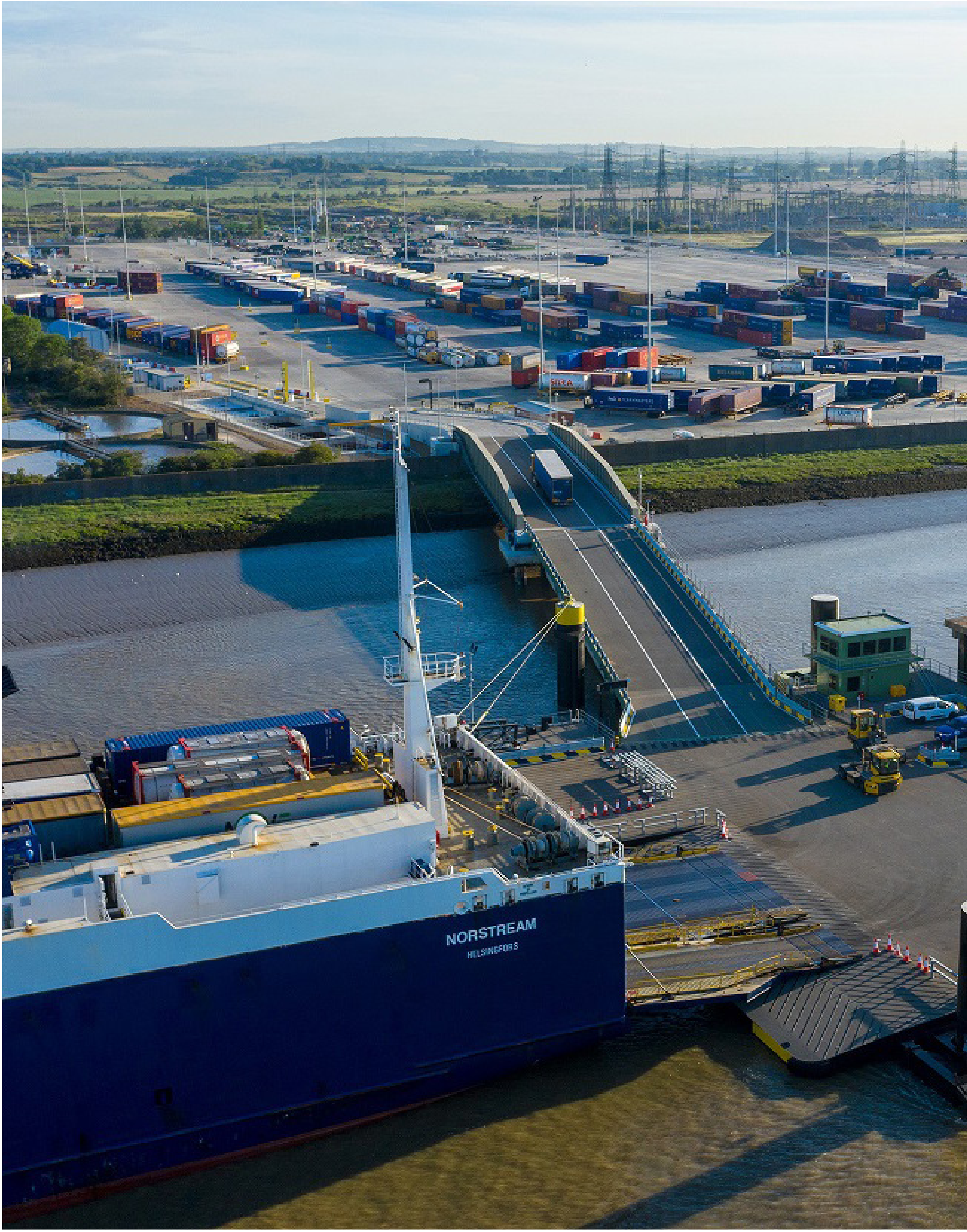
The above examples provide only a small insight into the wealth of innovation that SNC-Lavalin creates day to day.

I hope you enjoy the selection of technical papers included in this edition as much as we have enjoyed compiling them.

2022 Editorial Board Members

About the Cover

Phase One of HS2 will see a new high-speed railway line constructed from London to the West Midlands. Surveys revealed the presence of great crested newts (Triturus cristatus) throughout the Phase One route. Due to the protected status of these amphibians and the potential impact on their habitats, mitigation licensing was a legal requirement.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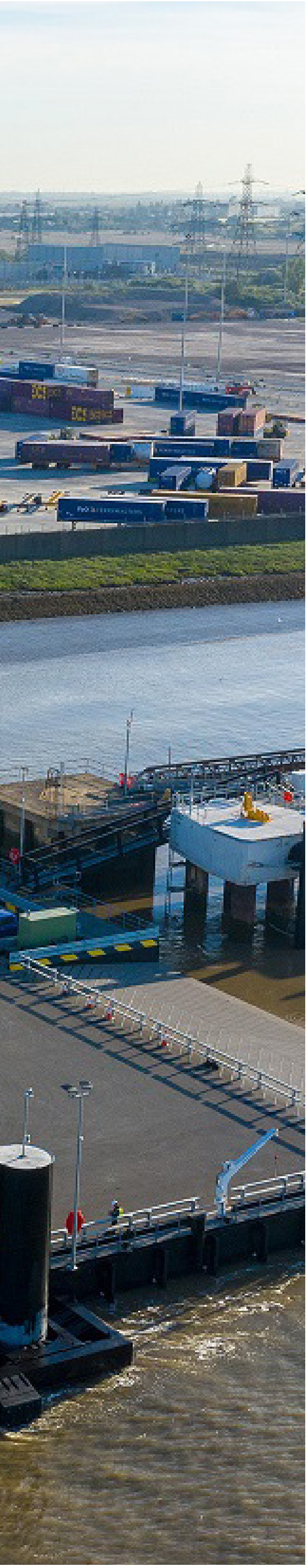

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Infrastructure Planning

01: Tilbury2 - Planning the UK's Newest Major Port

Abstract

Built on the site of the former Tilbury Power Station, Tilbury2 is a major port development dedicated to unaccompanied Roll On-Roll Off (RoRo) freight and bulk aggregate handling. This important facility was developed by Port of Tilbury London Ltd, part of the Forth Ports Group, to take advantage of:

- The opportunity presented by the demolition of Tilbury Power Station and
- the growth in RoRo traffic that was a significant trend in UK trade during the 2010s

The 61-hectare site presented many important challenges, from coordinating the plan alongside existing maritime infrastructure to developing an inland site that incorporates several underground features that needed careful treatment. It came into operation in May 2020 and has played an important role in handling RoRo traffic to and from Europe in a post-Brexit UK. This paper covers the planning to development stage of this important project, and covers the following range of issues:

- The importance of RoRo trade to the UK
- The commercial landscape that led to the Tilbury2 project coming to fruition
- Unaccompanied RoRo and containers on RoRo operations
- Aggregate handling operations
- How the existing maritime frontage at Tilbury was incorporated into the berth layout
- How the power station land area was developed to form a modern unaccompanied freight only RoRo and containers on RoRo terminal. It also included a construction materials and aggregate handling terminal
- How the planning included facilities for transferring a reasonable percentage of the container cargo and aggregates by rail

KEYWORDS
Maritime transport; Port freight; Roll On-Roll Off (RoRo) cargo; Project planning

1. Introduction

FIGURE 1-1

The completed Tilbury2 development

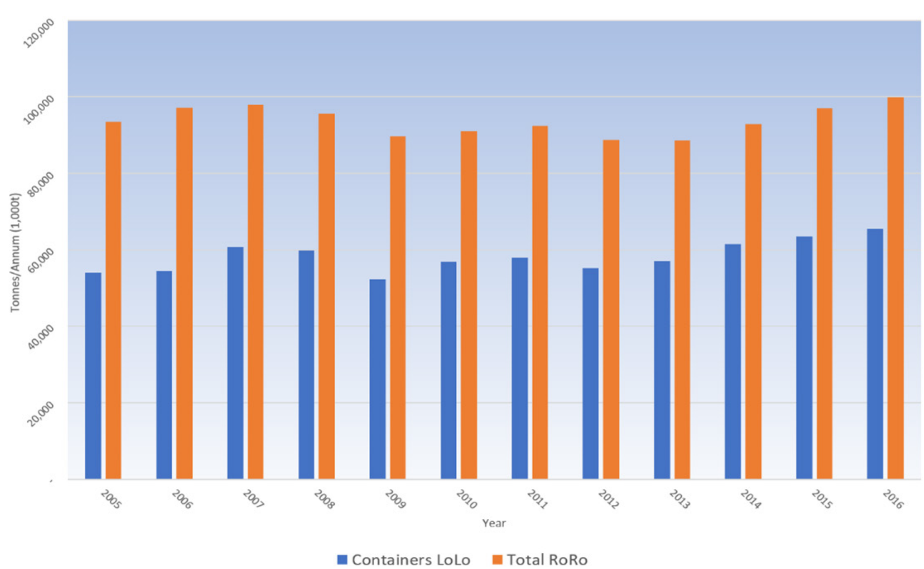
More than 80% of international freight is carried by ships all round the world. Apart from bulk trades such as oil, coal, and iron ore, most trade is containerised, so it is not surprising that the public's vision of a "port" features container ships and the familiar ship-to-shore gantry cranes.

However, there are other modes of maritime transport and one of the most important is Roll On-Roll Off (RoRo) cargo, such as that carried by ferries. The most notable example of that is the Dover/Calais crossing which handles some 23 million tonnes of goods, mostly by RoRo.

Figure 1-2 displays statistics from the UK Government Department for Transport on the amount of trade by tonnage handled by containers and RoRo freight (accompanied and unaccompanied). This shows that, in the UK, there is more than 40% more goods by tonnage being transported by the RoRo mode than by containers lifted on and lifted off ships.

FIGURE 1-2

Utilised cargo trade in the UK



2. The Opportunity of Tilbury2

2.1. OBTAINING THE SITE

In 2015, Port of Tilbury London Ltd (POTLL), which is a member of the Forth Ports Group, became aware that a 61-hectare site that formed part of the Tilbury Power Station was going to become available. This site was on the market along with a group of waterfront structures on the side of the River Thames. The Tilbury A power station was demolished in 2015 and the Tilbury B power station was set to be demolished when POTLL acquired the site. The location and site boundary of the acquired land is shown in Figure 2-1. The site is bounded by the yellow and red lines, the boundary between the two sites being a service corridor serving the Tilbury B power station site which was expected to become available in the future.

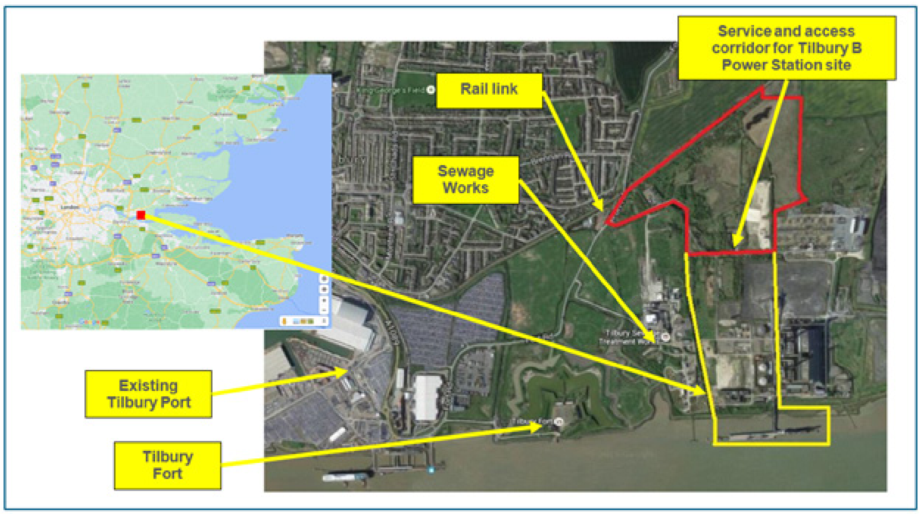


FIGURE 2-1

Location of Tilbury2 Terminal (Source: Google Maps & Google Earth)

POTLL judged that it was very much in their interests to obtain the site to enhance their trading capacity and expand their market share. The site was judged to be suitable for a RoRo terminal and a construction material and aggregate terminal (CMAT). The following sections give some background to the market scenario that confirmed their decision to proceed.

2.2. THE EXPANSION OF RoRo TRADE

The development of Tilbury2 is primarily targeted at the RoRo trade and the aspiration was for the site to be able to handle some 500,000 units per annum. Some perspective on this ambition can be given from the unaccompanied freight traffic statistics in 2014 from Port of London, Felixstowe, and Harwich (Table 2-1). After the significant slump in trade following the financial crash in 2008, there was a very significant growth trend (See Figure 2-2), so the opportunity presented a chance to attract market share in favourable trading circumstances. POTLL already had an unaccompanied RoRo trade which was served from the impounded dock, so there was a great attraction in expanding this. POTLL also traded in import/export trade vehicles, mostly from the pontoon linkspan near to the cruise berth. Tilbury2 presented an opportunity to expand this trade, as well. Figure 2-3 indicates that this trade was also experiencing significant growth.

When coupled with the very favourable location, a compelling argument for development was made.

TABLE 2-1:

2014 trade statistics for London, Felixstowe, and Harwich, 2014

		Tonnage (1000's)			Units (1000's)		
		Imports	Exports	All	Imports	Exports	All
London	Trailer	2,322	1,151	3,473	156	156	312
	Rail Wagon, shipborne port to port trailers and barges	2,931	694	3,625	112	109	221
Felixstowe	Trailer	1,784	1,043	2,828	100	98	198
	Rail Wagon, shipborne port to port trailers and barges	-	-	-	-	-	-
Harwich	Trailer	1,337	403	1,740	82	80	162
	Rail Wagon, shipborne port to port trailers and barges	209	34	243	11	8	19

Source: Atkins Ltd from UK Government DfT Maritime Statistics

FIGURE 2-2

UK major and minor ports all unitised freight traffic, 2000-2014

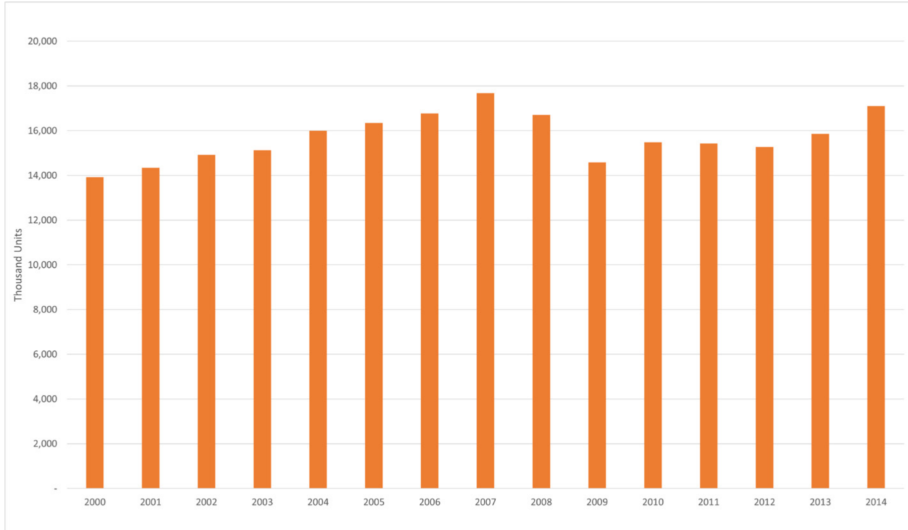
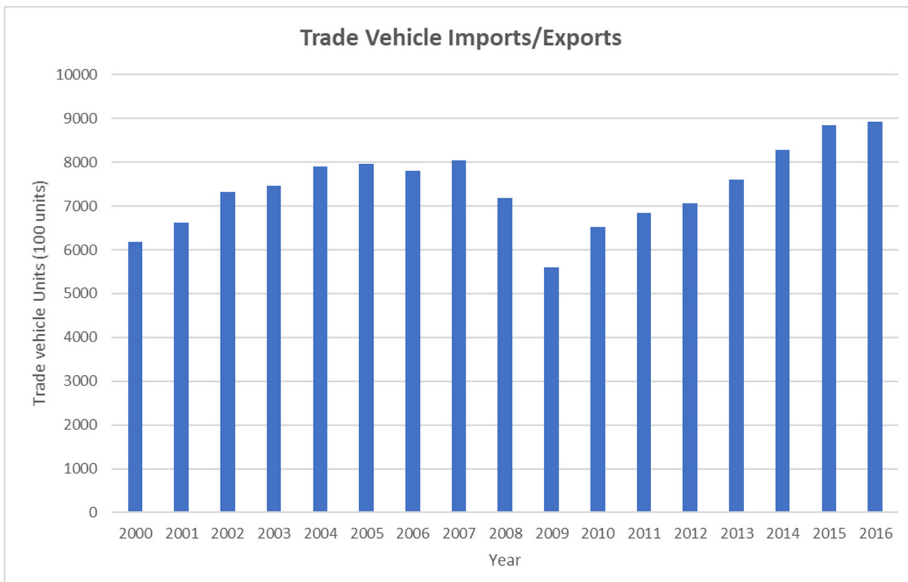


FIGURE 2-3

UK major and minor ports, import and export trade vehicles, 2000-2016



2.3. THE DEMAND FOR CONSTRUCTION AGGREGATES

POTLL aspired to achieve a throughput of 4.5 M tonnes of bulk material per annum with marine dredged aggregate considered to be the most likely material.

Marine dredged aggregates make an important contribution to the total supply of sand and gravel to the concrete producers and the UK construction industry. A large proportion of this aggregate makes its way to the Thames Estuary. Crown Estates had undertaken a study to assess the feasibility of developing one or two "hub wharves." These new hub wharves would reduce the delivered cost of aggregates/tonne through economies of scale at the wharf and the use of higher capacity dredgers. POTLL identified Tilbury2 as a potential Crown Estates Hub.

The Crown Estates Marine Aggregate Capability & Portfolio 2015 document stated that the hub wharf concept aimed to:

- Future-proof wharf locations by concentrating deliveries from numerous production facilities to a single safeguarded regional site
- Provide unrestricted shipping access to more modern and efficient wharf facilities
- Provide cost savings by optimising the dredging vessel cycle time and by increasing the economies of scale for processing and onward supply logistics
- Provide potential for increasing the supply of marine aggregates; and
- Reduce the number of vessels required and the capital investment required by industry

The hub wharf concept was applicable to a number of potential regional markets and was scalable. The first markets being investigated by the Crown Estate were the key London market and the English South Coast market.

POTLL identified that their aspiration could be achieved by becoming a Crown Estate hub wharf.

2.4. THE TILBURY2 SITE

The location of the Tilbury2 site is given in Figure 2-1.

The Tilbury site had been used for coal-fired power generation since 1951, when construction started on the "A" station and associated jetty. An aggregate plant, utilising pulverised fuel ash to make "Lytag" (a lightweight aggregate), was constructed to the north of the A station site and railway connections were also made. Land to the east of the power stations was used for ash disposal. In 1961, construction started on the larger "B" station and a second jetty was built as an extension to the existing one.

The A station closed and was decommissioned in 1983 with the turbine hall and other buildings demolished in 1999 before that part of the site was remediated. The Lytag plant was demolished, and railway lines decommissioned during the 1980s.

In 2011, Tilbury B was converted to operate on biomass and operated on 100% wood pellets and bio-oils until closure in 2013. Demolition is now complete.

During the operational life of the site, HGV traffic was comparatively light, although the site was then exporting around 25 loads per day of pulverised fuel ash for recycling.

The power station owner, RWE Generation UK plc, wished to retain sufficient land at Tilbury to provide an option to construct a gas-fired combined cycle gas turbine (CCGT) power station in the future and had undertaken an assessment of its future land requirements. This identified that approximately 61 hectares of land was surplus and therefore available to POTLL.

POTLL acquired an approximately 36-hectare brownfield site in 2016 (shown in red on Figure 2-1) to the east of its current port boundary. A further 25-hectare site (shown in yellow on Figure 2-1) was acquired by POTLL in January 2017. This land was formerly occupied by:

- The former A station
- the Lytag plant and
- the oil tanks for the A and B stations

2.4.1 RIVERFRONT STRUCTURES

During operation, the riverfront jetty handled up to 2.3 million tonnes of coal per annum. The jetty had a total length of 352m and was able to accommodate Panamax vessels of up to 68,000 tonnes. The berthing pocket was dredged to 13.8m below Chart Datum (CD).

The A jetty, originally built in the 1950s, ran west (upstream) from the access point. It was 213m in length with an "old part" 18.3m wide, plus 13.4m for a new jetty alongside the old one, which was built in 2003. All of the jetties have a deck height of 8.91m CD. The dredged depth of the A jetty was maintained to give a minimum draught of 13.8m. The new part of the jetty was designed for mobile cranes and HGVs up to 50 tonnes capacity.

The B jetty, built at the time of the B station construction, is 139 m long and 18.3m wide. This jetty had a dredged depth of around -7.2 CD when last surveyed but was typically dredged to 9.3m water depth. The arrangement of the various parts of the jetty is shown in Figure 2-4.

FIGURE 2-4

Tilbury2 - The jetty as it existed in 2016

POTLL's initial concepts envisaged providing a pontoon/link bridge type of linkspan between the two existing dolphins, D1 and D2, with the approach bridge crossing the river frontage at the extreme western end of the site.

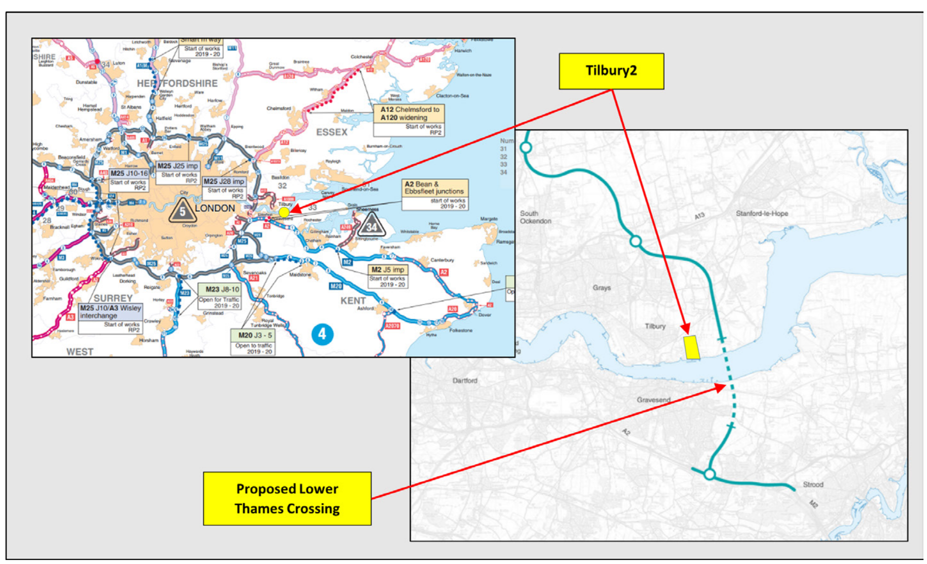
2.4.2 ROAD CONNECTION

Tilbury2's location is favourable for the handling of new freight traffic. It is located outside the London orbital motorway, M25, and is therefore clear of many of the most congested roads in the UK.

A new tunnel under the Thames is to be constructed alongside the Tilbury2 terminal. When complete, this would grant ready road access to the highway network both north and south of the Thames and therefore greatly enhance the desirability of Tilbury2 as a destination.

FIGURE 2-5

Road connections to Tilbury2



2.4.3 RAIL CONNECTIONS

There is an electrified rail route comprising a double rail line available at Tilbury, as indicated in Figure 2-1.

The planned expectations by POTLL for the new port terminal were for up to six trains per day each way (twelve movements in total) for aggregate and three trains per day (six movements in total) for containers. This forecast level of freight services was judged to be achievable, especially if some of the aggregate traffic could run at night.

For the container traffic, train lengths with up to 30 wagons carrying 45ft containers were envisaged (up to 460m long). For the aggregate wagons, it is assumed double headed trains with 18 wagons (14m long) would be used and would typically be 275m long.

Two principal routes exist between the Port of Tilbury and the rest of the rail network. All trains would use a length of line known as the Tilbury Loop as far as Barking in East London. From Barking, freight trains would have one of two possible routes across the north of London and into the rail network.

Outside peak hours, passenger services on the Tilbury Loop line were relatively light and running times for freight and passenger services are similar, meaning best use can be made of capacity. Although the line handles significant volumes of freight from other, it was judged that capacity would still exist for additional freight services.

2.4.4 CONSTRAINTS

There were several constraints on the way in which the combined Tilbury2 site could be developed. See Figure 2-1 which indicates most of them.

- Access needed to be maintained to a sewage works alongside the proposed terminal
- Space was needed to provide a suitably radiused railway loop from the Tilbury rail link into the terminal
- The service and access corridor across the site to Tilbury B Power Station site needed to be kept available
- The ground is generally very flat, presenting some significant drainage challenges
- An environmental area provided to mitigate for the construction of the old Lytag plant would need to be either respected or alternative habitat locations identified and provided
- Access needed to be arranged between the original Tilbury port and Tilbury2 to permit the ports to be suitably managed

2.5. THE SIGNIFICANCE OF TILBURY2 TO PORT OF TILBURY IN GENERAL

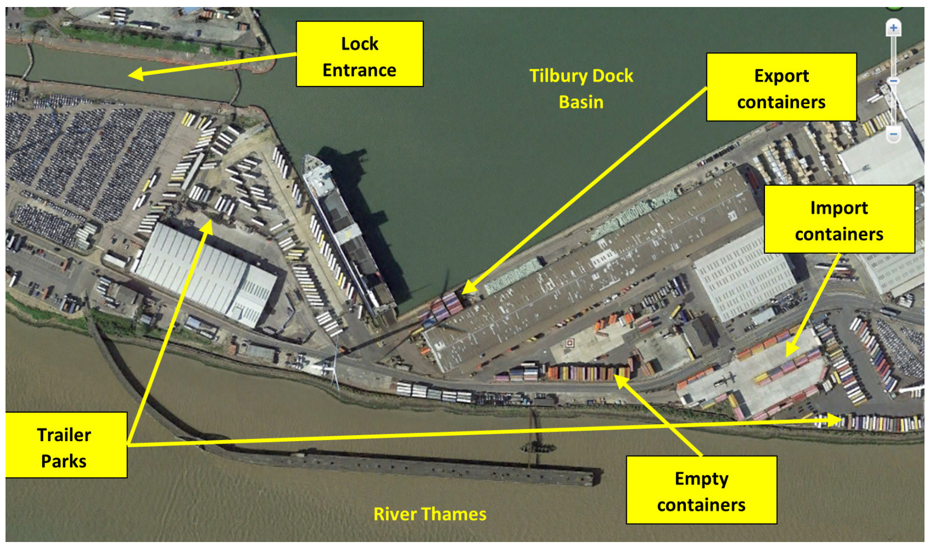
Tilbury already accommodated a modest unaccompanied freight terminal including containers in RoRo at a corner of the main Tilbury Dock basin. The shipping lines were P&O Ferries and Bore Line which operate unaccompanied freight only RoRo vessels between Tilbury and Zeebrugge.

In addition, several shipping lines brought trade cars and vehicles to the port via the floating pontoon terminal near to the cruise terminal.

The previous operation made imaginative use of a difficult shaped site (see Figure 2-6), but it is apparent that some of the parking and stacking areas are not conveniently spaced and, while appropriate for the trade that existed, allowed little room for expansion.

FIGURE 2-6

Existing unaccompanied RoRo terminal area



It can also be seen that the proposed availability of Tilbury2 with a hoped-for capacity of 500,000 units per annum representative a step change in operation for POTLL and a substantial expansion of market share.

Furthermore, the proposed provision of a riverside berth meant that there was considerable time saved in turning round the visiting vessels, allowing either more frequent services or else slower and more fuel-efficient voyages.

2.6. THE PLANNING-TO-DEVELOPMENT PROCESS

The organisations that took part in the development process are listed in Table 2-2.

Owing to the trade ambitions of the port, this was triggered as a nationally significant infrastructure project and as such had to go through the UK's Development Consent Order (DCO) process.

This process requires that POTLL present plans and performance specifications that suitably define the scope, concept, environmental, and social impacts of the development so that the government processes can be carried out efficiently. In order to facilitate its progress through the planning processes, it was essential that the environmental impact of any proposals were carefully evaluated and mitigated to the greatest possible extent. This required actions such as minimising the size and number of marine structures in order to minimise the impact on the river processes.

The timetable of the development process is given in Section 7.

TABLE 2-2:

Organisations taking part in the development process

Function	Organisation
Project promoters	Port of Tilbury (London) Ltd (a member of Forth Ports Group)
Master planning and front end engineering design	Atkins Ltd
Legal services	Pinsent Masons
Planning	Vincent & Gorbing
Environmental Consultants	Atkins Bioscan CgMs Consulting David Jarvis Associates i-Transport
Land agent	Ardent
Cost consultancy	Currie and Brown
Design and construct contractor	Graham Construction Ltd
Terrestrial work designers	HBPW
Marine designers	Doran Consultants

3. RoRo Operations

3.1. THE PROCESS AND IMPLICATION FOR UNACCOMPANIED FREIGHT

The flowchart for the unaccompanied freight operation is given in Figure 3-1. This describes the following processes.

- Bringing the truck (HGV tractor/trailer vehicle) to the terminal
- Separating the tractor from the trailer
- Handling the trailer onto the vessel using port tractor plant
- Lashing it securely onto the vessel

This is the export trailer operations in summary. The import trailer operations are reversed.

This requires that trailers are parked in the terminal in individual randomly accessed parking places, sometimes in square or angled back-to-back parking slots, sometimes in drive-through echelon parking places for greater speed or operation. Examples of such parking layouts are given in Figure 3-2.

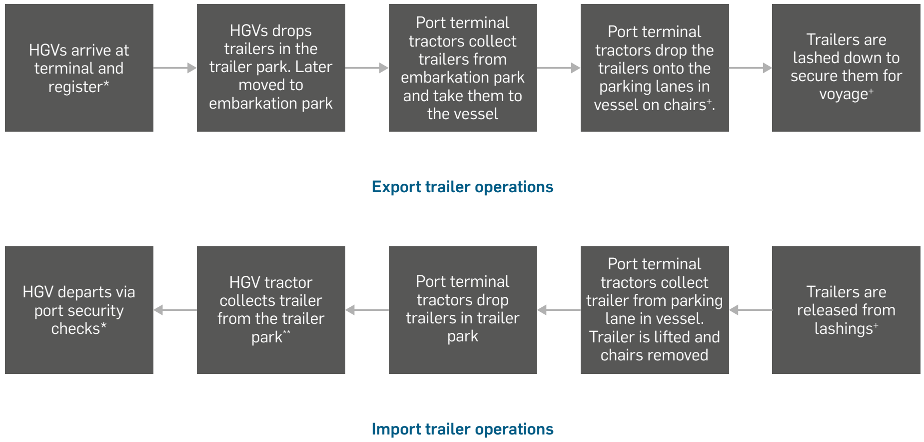
A large area is required to park each unit for such random access parking spaces, being typically 83m^2 per unit which is more than 30% greater than the equivalent parking space required for nose-to-tail driver-accompanied freight units.

Furthermore, unaccompanied import trailers can be expected to dwell in the terminal for an average of 1.5 days. So, in the case of a typical morning and evening ferry service, there would need to be parking spaces for three times the average carryings of the ship.

Since this loading is no more than a normal highway loading, the paving requirements are not severe.

FIGURE 3-1

Flowchart for unaccompanied RoRo trailer operations



HGV = heavy goods vehicle – Highway tractor trailer trucks

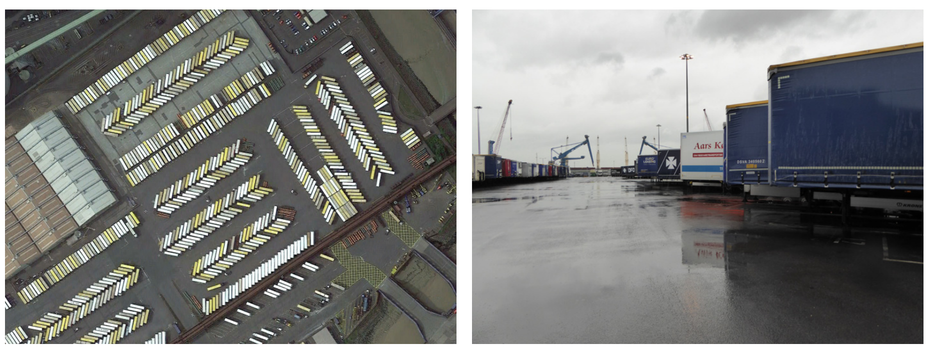
* *Increasingly an automated process*

** *Can involve waiting for customs processes*

\+ *Usually a steel support frame and chains. Lashings can be replaced by special automated rigid "chairs"*

FIGURE 3-2

Typical trailer parking layouts



Source - Google Earth Pro

3.2. THE PROCESS AND IMPLICATION FOR CONTAINERS ON RoRo

The flowchart for the containers on RoRo is given in Figure 3-3. This describes the following processes:

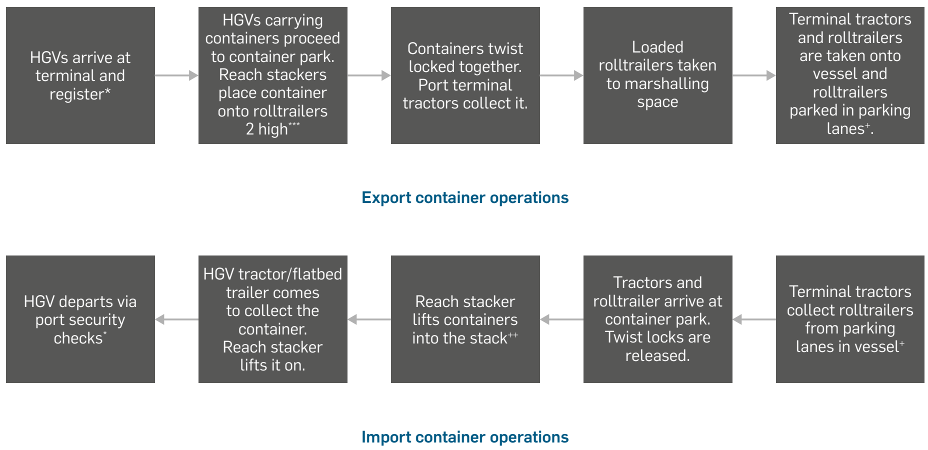
- Bringing the truck (HGV truck with flatbed trailer and container) to the terminal
- Placing the container onto roll trailers 2 high (see Figure 3-4)
- Taking to the marshalling area for embarkation
- Embarking the vessel and placing into parking lanes on the ship

This is the export trailer operations in summary. The import container operations are reversed but include an additional step of placing containers into the import container stack.

Import containers can be expected to dwell in the terminal for an average of 2 days. So, for a typical RoRo service arriving morning and evening, enough import container storage is required to accommodate 4 average shiploads of containers.

FIGURE 3-3

Flowchart for containers on RoRo operations



HGV= heavy goods vehicle – Highway tractor trailer trucks (units can arrive/depart by train)

* *Increasingly an automated process*

** *Can involve waiting for customs processes*

*** *Can be unloaded into a container stack first*

\+ *Rolltrailers are low enough not to require lashings*

\+\+ *Container stacks for RoRo are typically 2 high*

An additional feature of containers in RoRo operations in the North Sea is the use of 45ft containers instead of the more normal 40ft containers. 45ft containers have become popular because they are the same length as normal European highway trailers. Therefore, the Tilbury2 container parks were planned on the basis of 45ft containers forming the bulk of the trade.

FIGURE 3-4

Container on RoRo operations in Port of Tilbury (before Tilbury2)

A) 45' containers on rolltrailers (note extensions of containers)

B) Rolltrailers and containers marshalled for export

C) Containers disembarking

D) Import container stacks

Figure 3-4 shows a typical container on RoRo operations from the original Port of Tilbury (see also Figure 2-6).

The handling operations for containers are usually carried out using reach stackers. Because reach stackers exert enormous axle loads (front axles greater than 100 tonnes) and they are used repetitively, the paving for such container areas has to be exceptionally robust. Indeed, paving thicknesses in such terminals are often significantly greater than required for major RTG (rubber tyred gantry) 5-high container stack operations.

Also, because of the height of the double stacked containers on the comparatively narrow footprint of the roll trailers, it is necessary for the roadways to be very regular and the paving under the roll trailers to be fairly heavy duty. Furthermore, there is also an issue of wind on the side of the 2-high containers during embarkation/disembarkation operations. Wind does not usually cause problems but, in the case of the Tilbury2 scheme, the rolltrailers have to progress over a long and quite high approach bridge in order to reach the berth. This bridge was sideways on to the prevailing winds. For this reason, wind shielding formed part of the RoRo berth plans.

3.3. THE PROCESS FOR TRADE CAR HANDLING

Figure 3-5 shows the typical flowchart for trade car or trade vehicles (buses, coaches, delivered HGVs, and wheeled construction equipment). This type of operation is common worldwide:

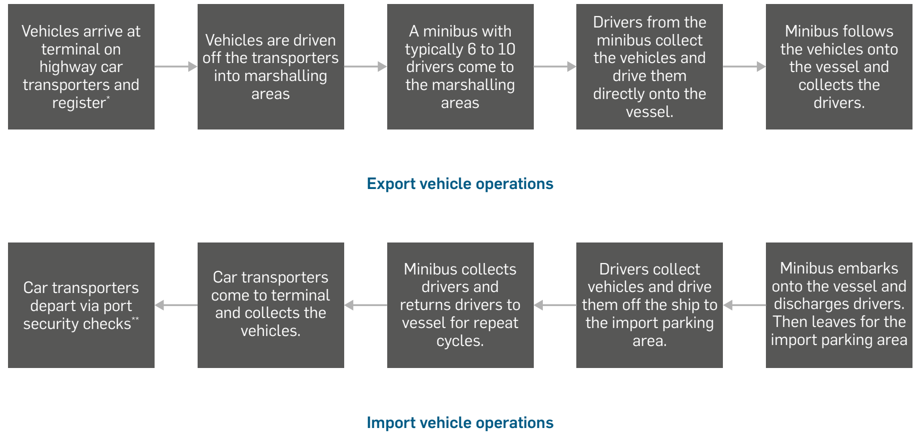
The flowchart shows export and import vehicle movements, but usually ports tend to specialise in one or the other.

Depending on the operation, trade car parking can be in nose to tail rows or random-access parking in the case that pre-delivery services are carried out in the port estate. Figure 3-6 shows a nose-to-tail block parking arrangement suitable for rapid operation. The area requirements are different for the different modes of parking, but because of the very small footprint of individual vehicles, this type of trade lends itself to being accommodated in awkward or restricted land areas and thus allows flexible use of a port estate.

Another feature of trade car terminals is that, because the loading is very light, the paving requirements are modest and such areas can be developed cheaply and on poor ground.

FIGURE 3-5

Flowchart for trade car (or other vehicle) operations



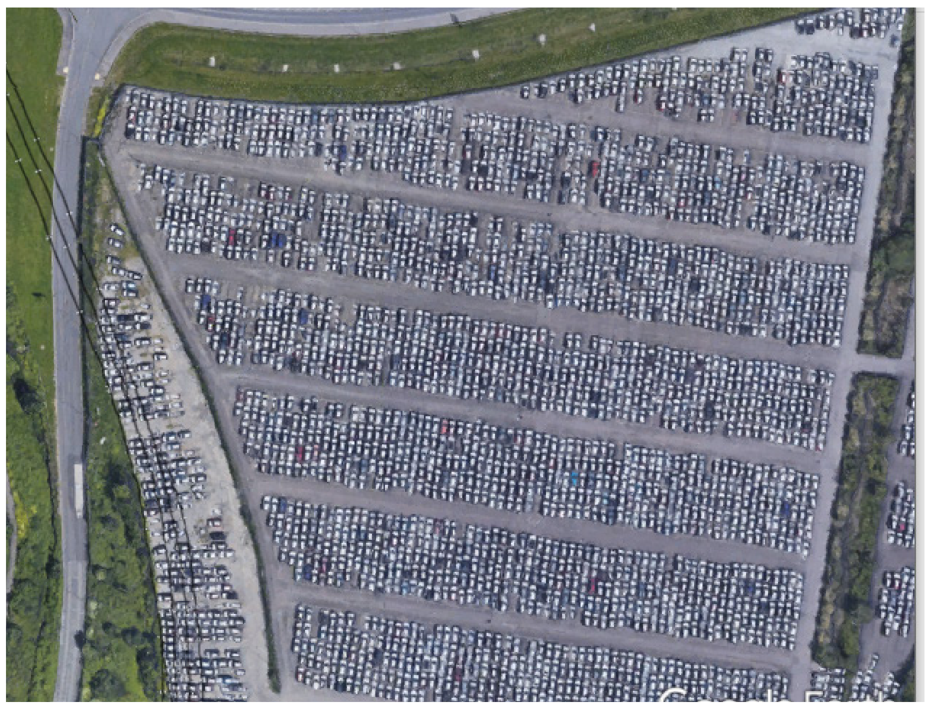
HGV= heavy goods vehicle – Highway tractor trailer trucks (units can arrive/depart by train)

* *Increasingly an automated process*

** *Can involve waiting for customs processes*

FIGURE 3-6

Typical trade car parking at Port of Tilbury



4. Aggregate Handling Requirements

A construction materials and aggregates handling area (CMAT) was intended to be included in the Tilbury2 scheme. POTLL had been in discussion with several aggregate handling companies on the basis that 4.5 million tonnes of aggregate and other bulk materials could be handled per year.

The advice from such operators defined that an area in the order of 10 hectares was required to handle this trade.

The plan was for the aggregate to be offloaded to the shore into hoppers over conveyor belts from the vessels' own unloading gear and transported from berth front to the ship via the existing conveyor bridge and then to the CMAT terminal area via conveyor belts.

The CMAT terminal would then comprise several stockpile areas fed by conveyors, typically in a circular layout plan. Conveyors from the CMAT area would then be able to deliver aggregates to the proposed rail siding for import into the UK hinterland.

5. Adapting the Berth Frontage

5.1. THE CONDITION OF THE EXITING JETTIES

The existing jetties are described in outline in Section 2.4.1 and illustrated in Figure 2-4, but in summary:

- Tilbury A Power Station was built in the 1950s. Tilbury B Power Station was built in the early 1960s
- The jetty has an approach bridge at the boundary between the sites for the Tilbury A Power Station and Tilbury B Power Station
- The section to the west of the approach bridge is known as Jetty A and was built in the 1950s. To the west, there is another marine structure which is over the shaft for the cooling water outfall
- The section to the east of the approach bridge is known as Jetty B and was built in the 1960s. There is an additional dolphin to the east of the jetty
- The berthing line of Jetty A was moved southwards in 2004 and an infill platform with raking piles was constructed. At the same time, 2 new mooring dolphins were constructed. See Figure 5-1

Condition surveys were carried out in 2015, which confirmed that, for the age of the structures, the jetties were in reasonable condition. The extension for Jetty A and the mooring dolphins were in good condition and therefore were expected to be able to effectively serve as the berth frontage for the proposed new RoRo berths.

FIGURE 5-1

Jetty A extension structure

Although the condition of Jetty B was considered reasonable, the aspirations were for the berth to handle larger vessels than originally used and so some upgrading work would be required.

5.2. VESSEL REQUIREMENTS

5.2.1 RoRo VESSELS

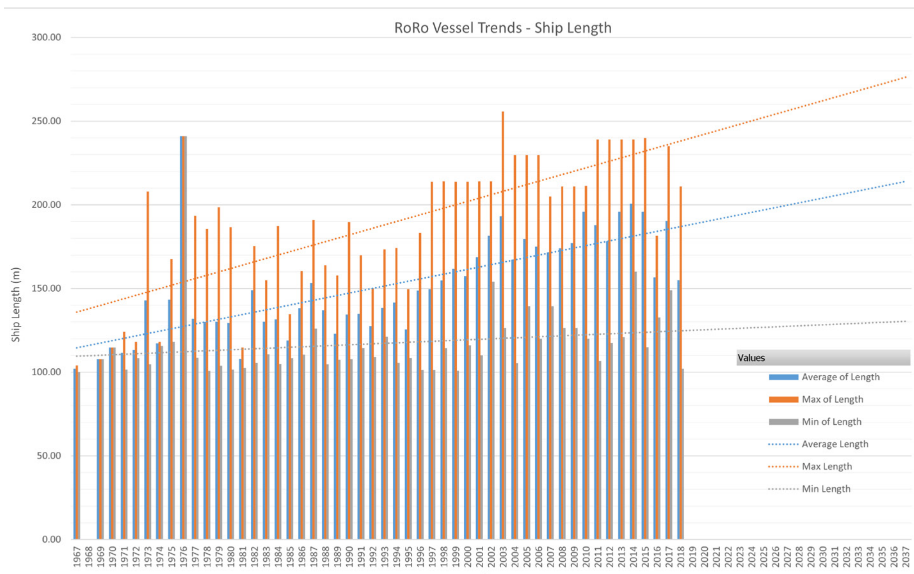
Tilbury2 was to be a state-of-the-art RoRo terminal and, as such, needed to be ready to accommodate a range of vessels that represented not only the existing trade but also the potential future trades.

The RoRo operations that existed when Tilbury2 was being planned were based on vessels such as MV Bore Sea and MV Norsky (See Table 5-1). However, while it is well known that container vessels have been increasing in size in a very dramatic way, it is not often appreciated that the same has been happening to RoRo vessel design and for the same reasons of greater fuel efficiency and capacity. The increased capacity also has the benefit of permitting the same trade to be handled with slower crossing speeds. Given that vessels that sail slower use less fuel, there is an important reduction in emissions per voyage.

The trends for freight RoRo vessel length and capacity (lane metres) are shown in Figure 5-2, from which it can be seen that over 50 years vessel lengths have, on average, increased in length by 65% and in capacity by 600%. The capacity increases are disproportionate and have generally occurred as a result of improving vessel design, for instance by allowing changes in allocation of buoyant space permitting additional vehicle decks to be introduced at below waterline locations without loss of stability.

FIGURE 5-2

RoRo vessel trends, 1967-2017



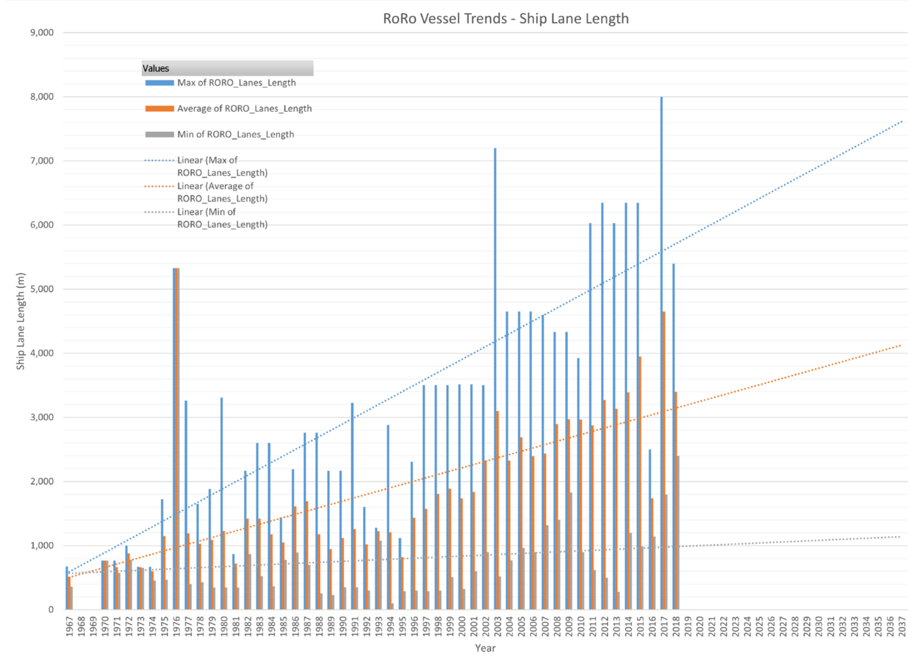
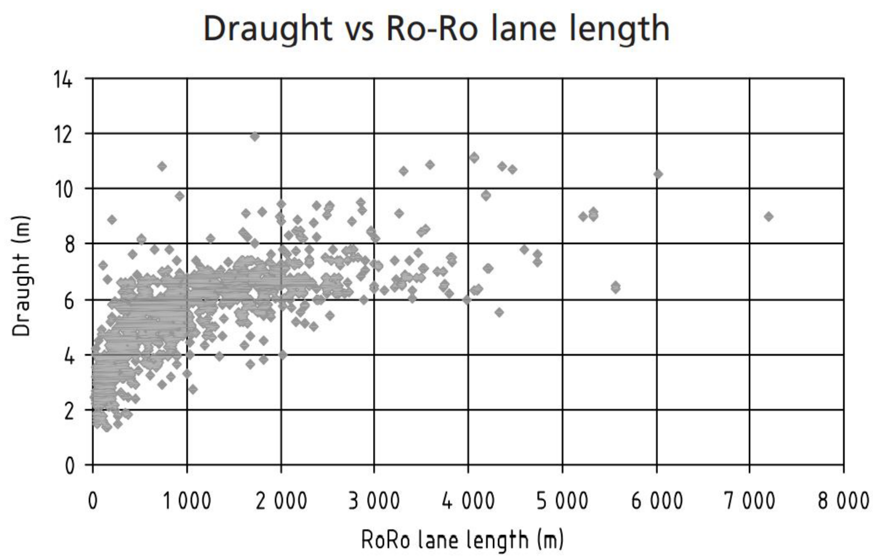


FIGURE 5-3

Extract from Figure D.9, BS 6349 Part 1.1 showing RoRo vessel draught



It is worth noting that, since the planning process began, vessels such as the freight only RoRo MV Celine (Length 234m, Beam 33.3m) have come into service in the North Sea area, justifying the need to make allowance for longer vessels than were presently serving the Tilbury trade.

There has also been an increase in beam of vessels, but it is generally noteworthy that the draught of freight-only RoRo vessels has not greatly increased with capacity. Figure 5-3 shows an extract from BS 6349 Part 1.1 showing the range of draughts on RoRo ferry vessels, from which it is apparent that the vast majority of ferry vessels are less than 8 m draught. This is born out in the selection of design vessels that was the basis of the planning for Tilbury2 and which are laid out in Table 5-1.

It will be noticed that the potential draught of the PCC and PCTC vessels is greater than 8 m, so the planning for the berth frontage would need to take account of that.

5.2.2 VESSELS FOR THE CMAT (BULK HANDLING)

As a result of consultations with aggregate supplying organisations, POTLL assembled a list of vessels that they wanted to accommodate on the berth frontage in order to supply the CMAT terminal (See Table 5-2). The range of sizes is considerable, but perhaps the most notable feature is that the largest vessels to be accommodated were significantly larger than those for which Jetty A and Jetty B were originally designed. Jetty A and Jetty B had been handling vessels up to 68,000 DWT, but the largest vessel that was planned for Tilbury2 is more than 96,000 DWT. This has implications for the waterfront design (See Section 5.3).

All of the vessels under consideration had their own discharging equipment, so the quayside equipment could be modest in nature.

TABLE 5-1:

Design vessels for the RoRo berths

Name of Ship	Length (m)	Ro-ro Lane Length (m)	Beam Breadth (m)	Draught (m)	Deadweight (tonnes)	Displacement (tonnes)
Stern Ramp RoRo Vessels						
MV Bore Sea	195.4	2,907	26.2	7.4	13,625	22,990
MV Norsky	180	2,630	25.5	6.5	11,400	19,900
New large RoRo ship class	240	TBC	29.4	7.9	TBC	TBC
Car Carrier (Pure Car Carrier (PCC), Pure Car and Truck Carrier (PCTC)) and Con-Ro Vessel						
Graceful Leader	199.98	6,658	32.26	10	21,000	43,200
Morning Linda	232.37	8,011	32.26	10	28,230	55,700
Otello	199	6,713	32.26	11.3	22,650	43,347
Grande Abidjan	236.32	5,772	36.190	9.75	30,801	53,000

TABLE 5-2:

Design vessels for the Bulk Handling (CMAT) Berth

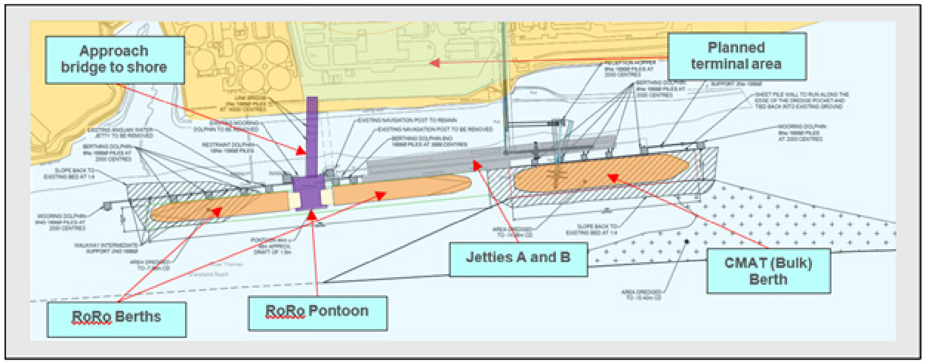
Name of Ship	Ship Type	Length OA (m)	Length BP (m)	Beam Breadth (m)	Draught (m)	Dead-weight (tonnes)	Displacement (tonnes)
Cemisle	Self Discharging Cement Carrier	119.95	113.33	16.60	6.7	5,183	7,200
NACC Poros	Self Discharging Cement Carrier	119.95	111.97	16.84	6.7	8,107	11,300
NACC NAPOLI (Previously MV Endeavour)	Self Discharging Cement Carrier	170.00	162.00	27.55	10.0	30,000	39,100
Sage Amazon (Previously JS Amazon)	Self Discharging Bulk Carrier	199.99	193.74	32.30	13.3	63,227	74,917
Yeoman Bridge	Self Discharging Bulk Carrier	249.90	239.00	38.07	15.0	96,772	115,473
Reimerswaal	Trailing Suction Hopper Dredger	130.25	119.75	22	7.95	12,525	-
Sand Heron	Trailing Suction Hopper Dredger	99	94.5	16.3	6.54	5,916	8,388
Alan Bennet	Aggregate Barge	76.5	-	10.96	3.1	1,740	-

5.3. UPGRADING THE BERTHING LINE

The berth frontage that was eventually selected for inclusion in the Development Consent Order (DCO) application is as shown in Figure 5-4.

FIGURE 5-4

Berth frontage planned for Development Consent Order



The plan included the following features:

- For the RoRo Berths
  - The berthing line was taken from that for Jetty A
  - 7 new dolphins (5 to the west of the RoRo pontoon and 2 to the east)
  - A dredged area to -7.88m CD for the west berth
  - No dredging was required for the east berth which was already dredged to a minimum draught of 13.8m

- For the CMAT Berth
  - The berthing line was also taken from Jetty A
  - Because Jetty B was designed for smaller vessels than planned for the CMAT berth, the DCO plan was based on new free-standing dolphins in front of the jetty to provide sufficient strength to accommodate the bulk vessels
  - A dredged pocket to -14.98CD to accommodate large bulk vessels, though the approach dredging areas were dredged to a lesser depth so that arrival at the berth would be tidally limited to some extent

The layouts were subjected to a full navigation simulation hosted by the Port of London Authority (PLA) to confirm the appropriateness of the scheme.

5.4. THE PONTOON LINKSPAN

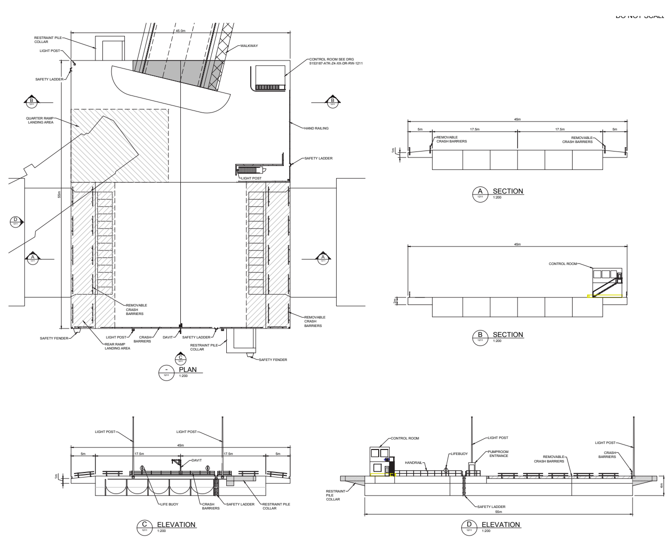
The concept of using a pontoon/link bridge type of linkspan was an early aspiration in the project and was clearly very suitable for this configuration. Fortunately, the layout of the site allowed such a linkspan to be located in the area to the west of Jetty A and also west of the cooling water outfall shaft. However, the geometry of the terminal area meant that the approach bridge was required to approach the shore at a small angle.

Although the main purpose of the RoRo berths was to accommodate trailer and containers on RoRo operations, POTLL also wished to accommodate PCC and PCTC (trade car) vessels on the berth, if possible. POTLL's trade car service was being served by the pontoon that is close to the cruise berth in the main port, but it was clearly desirable to have an alternative option.

Since PCC/PCTC vessels are usually equipped with quarter ramps on the starboard side, this could only be achieved by accommodating the vessels on the west RoRo berth. Furthermore, this required the pontoon surface to extended landward with the result that the pontoon needed to be dimensioned at 45m parallel to the berthing line and 55m perpendicular to the berthing line, making it one of the largest pontoons manufactured for such a purpose (See Figure 5-5).

FIGURE 5-5

Pontoon layout for the Development Consent Order

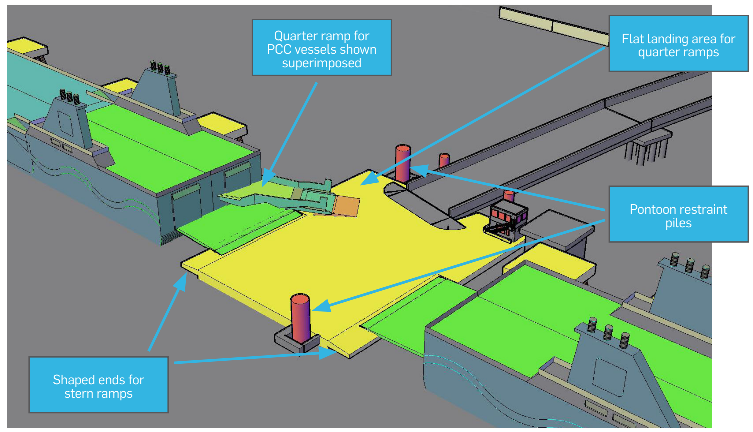


Pontoon restraints are usually located to the shore side of a pontoon to allow for easy installation. However, in this case, because the pontoon was longer in the direction perpendicular to the berthing line, this would not have provided a sufficiently secure restraint in the case of accidental impact on the pontoon face. Therefore, the pontoon restraint system was based on providing two restraint piles, one on the shore-facing side and one on the river-facing side of the pontoons.

The exact position of them had to be carefully considered so that it would be possible to manoeuvre the pontoon successfully into place (See Figure 5-6).

FIGURE 5-6

3D image of proposed pontoon



The performance specifications included with the DCO required the following:

- All geometry to comply with BS 6349 Part 8 (Code of practice for the design of RoRo Ramps, Linkspans and Walkways)
- Threshold heights for the vessels (which is the height from vessel vehicle deck to waterline) to be able to vary between 3.5m and 2.0m
- Where the energy absorption for vessel impact was to be taken at the fender piles, the forces caused by wave loads should not cause any fender units to buckle and therefore risk fatigue effects
- Any change of freeboard was to be achieved by a ballasting arrangement with the requirement that it could be altered to fit any ship within a 3 hour time period
- The deck loading of the pontoon and approach bridges should take account of the extremely heavy wheel loads exerted by roll trailers carrying 45ft containers 2 high

Another feature of the scheme was the need to cross the flood protection embankments at a comparatively high level. Figure 5-7 and Figure 5-8 shows this feature clearly. It also shows how, between planning and construction, the scheme was altered so that the two approach spans that were planned were eventually executed as a single span.

FIGURE 5-7

Cross section of RoRo approach as planned

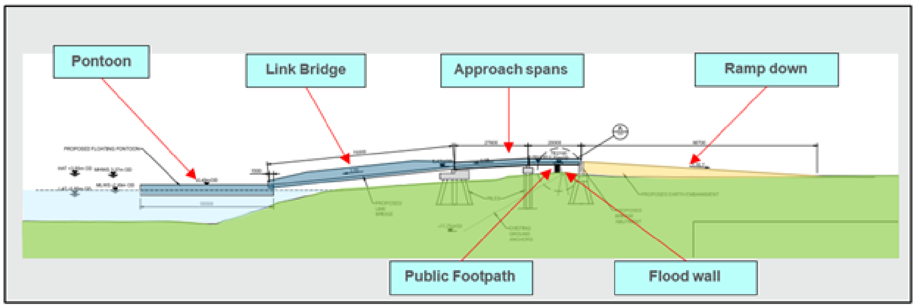
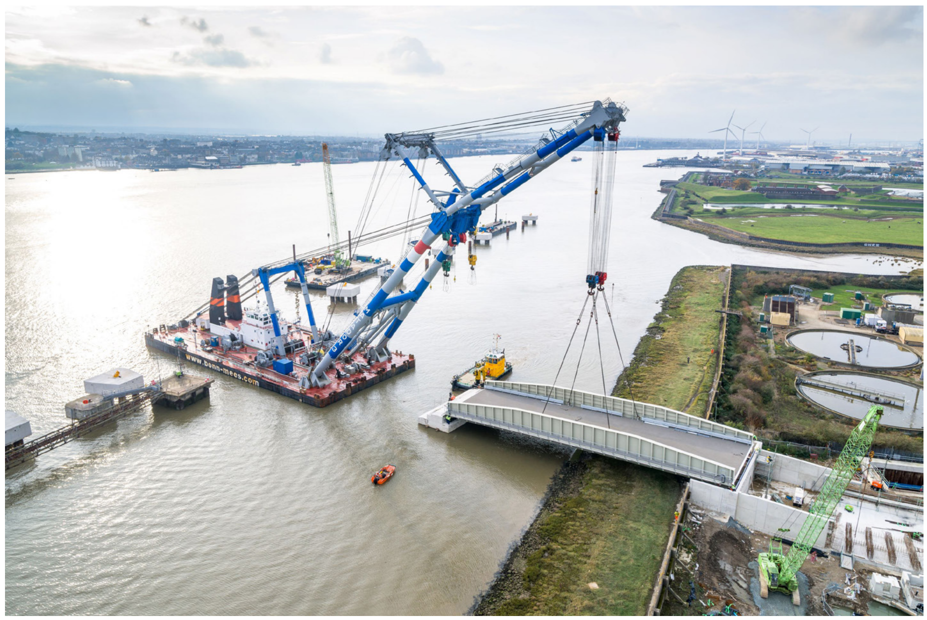


FIGURE 5-8

Approach bridge as constructed being lifted into position



6. Developing the Terminal Area

6.1. THE CHALLENGE OF THE EXISTING SITE

The Tilbury Power Stations were constructed on difficult ground conditions with large depths of alluvium with interspersed peat lenses overlying a chalk bedrock. See the typical borehole pattern shown in Figure 6-1. For this reason, the original power station was constructed on an array of piled foundations which supported the immense loads from the buildings, boilers and generating blocks (see the example in Figure 5-2).

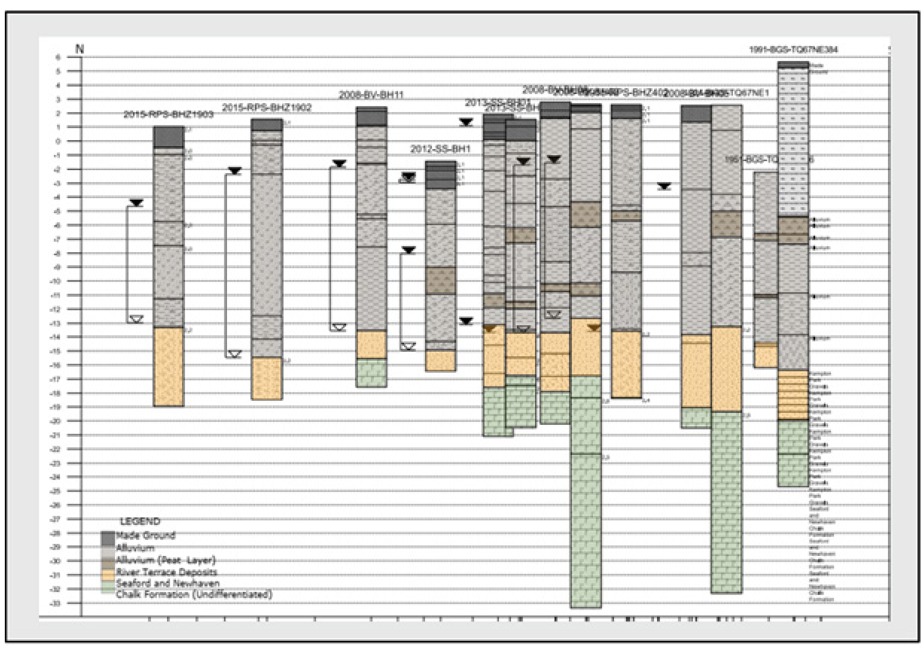
Furthermore, there were numerous culverts and other underground features that were left below ground level when the site was handed over to POTLL.

Further challenges could be expected due to the different ground usages that the site had experienced over the years. Figure 6-3 shows some of these. The development of the terminal area had to take account of the following:

- Large settlements could be expected in the parts of the site that had not previously had construction on them
- No settlement was expected in the areas of the piled foundations
- The areas of old coal yards had experienced substantial long-term surcharges and could be expected to settle less than other areas

FIGURE 6-1

Typical boreholes on the site



Fortunately, the land uses proposed for the Tilbury2 terminal are comparatively tolerant of settlements.

- Trailer parking can tolerate settlements, even reasonably irregular settlements without great difficulty
- The loads exerted by trailer parking are not great
- Parking of trade cars applies light loadings and is extremely tolerant of settlements

However, the container operation does require a reasonably regular surface and, because it is for a reach stacker operation, also requires an exceptionally robust pavement. Since the plan was for construction to take place under a design and construct contract, it was necessary to draw attention to all these issues so that the contractor's designer could take them into account during the detailed design.

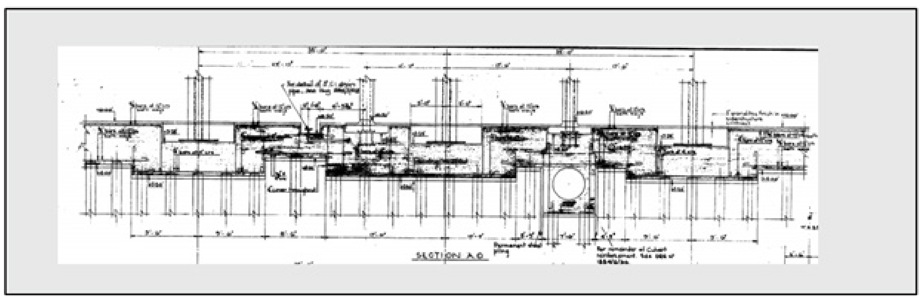


FIGURE 6-2

Piled foundations for Tilbury A Power Station (extract)

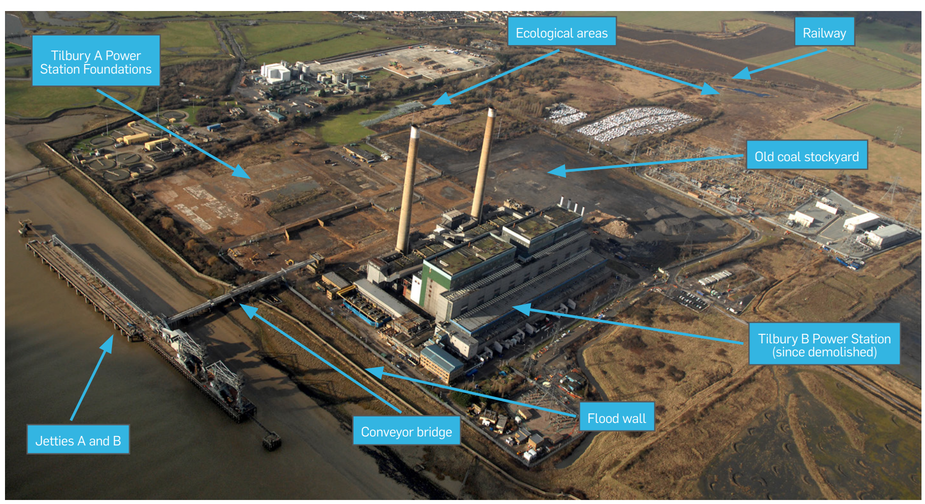


FIGURE 6-3

The site at the time of planning (2017)

6.2. AREA ALLOCATIONS

The objective of the plan was to accommodate the following functions:

- Freight RoRo operations
- Container on RoRo operations
- A rail intermodal terminal
- Possible warehouse storage buildings
- The CMAT (Construction material and aggregate terminal)
- Trade car storage
- Maintaining the service corridor to the Tilbury B Power Station site

The site area, while substantial, was nevertheless difficult to plan because it is comparatively narrow. Arranging for all the functions to be accommodated in the area and also provide space to introduce a railhead into this narrow site was particularly challenging.

In the end, the area zoning proposed in Figure 6-4 was adopted for the following reasons:

- The location of the rail siding needed to be on the east side of the site because otherwise it would cut off the access road arrangements
- The service corridor referred to in Section 2.4.4 needed to be kept operational and therefore divided the site in a particular way
- It was necessary for the CMAT terminal to have access to the railhead. This could be done using conveyors, but the conveyors would need to be routed to alongside the railhead, i.e., to the east. After considering locations near to the berth, it was eventually identified that placing it to the north of the services corridor gave it a sufficient area and also it fitted naturally into the curve of the rail track approach (see Figure 6-5)
- The unaccompanied RoRo operation is energy intensive, so locating the RoRo and container parking areas as close to the berth as possible reduces the carbon footprint of the terminal
- The long conveyor to the CMAT has a very modest land footprint and is typical of many similar CMAT operations elsewhere

FIGURE 6-4

Tilbury2 - Cargo Zoning

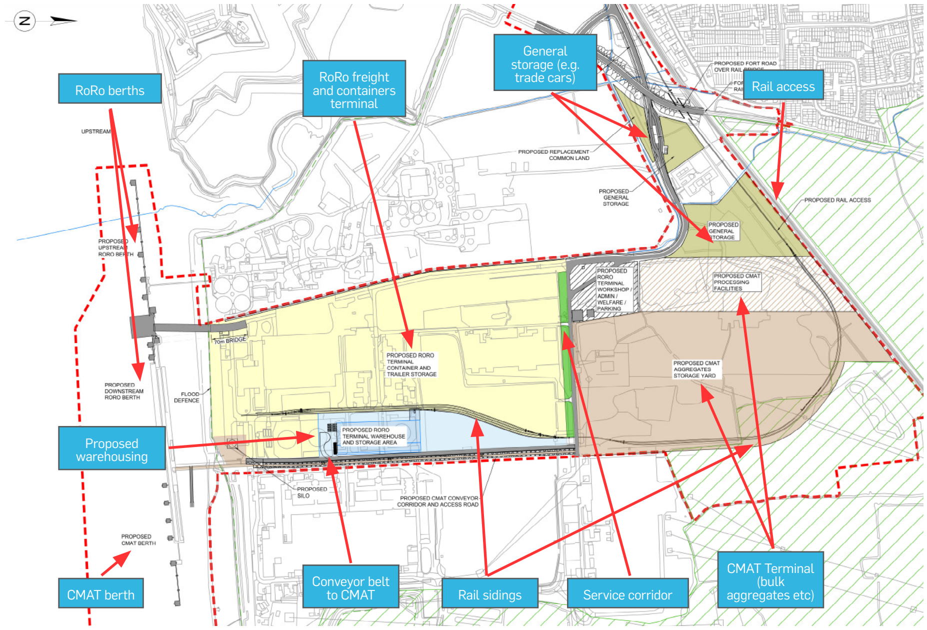
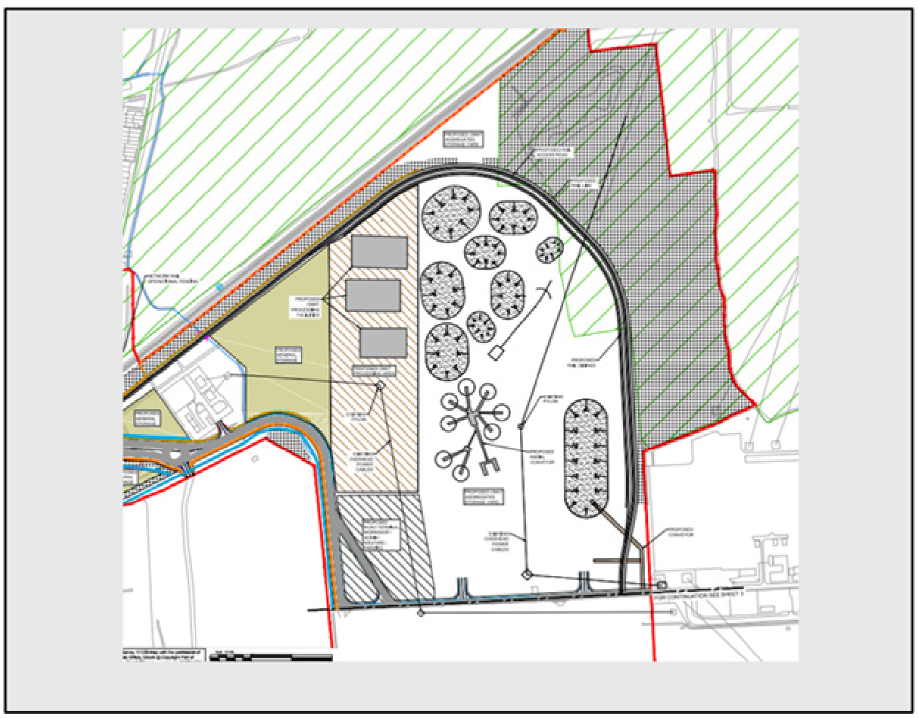


FIGURE 6-5

CMAT Terminal layout



This zoning has been largely carried through to implementation.

6.3. THE RoRo AND CONTAINER FREIGHT TERMINAL AREA

The site to the south of the Services Corridor and bound by the sewage works to the west and the Tilbury B Power Station to the east was dedicated as a freight and container RoRo terminal.

The constraints were:

- The aspiration to accommodate a warehouse operation into the area
- The need to facilitate rail import/export operations
- The access point to the berth which was fixed to the southwest corner of the site
- The need to provide the mix of RoRo freight and trailer parking arrangements, which have very different geometric requirements
- A requirement to accommodate reefer containers and refrigerated trailers

The latest layout considered prior to Development Consent Order application was as shown in Figure 6-6.

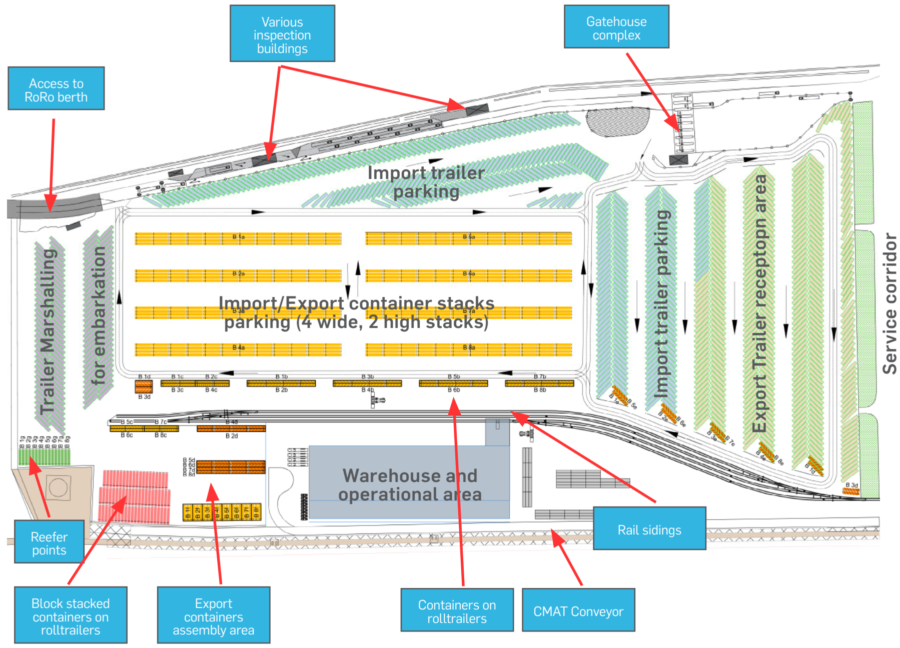
The specification for the terrestrial area for the paving works included the following six types:

- Type A1 – RoRo Terminal Pavement laid on existing ground - Container yard
- Type A2 – RoRo Terminal Pavement over existing sub-structures - Container yard
- Type B – RoRo Terminal Pavement - Roll trailer yard
- Type C – RoRo Terminal Pavement - Trailer yard
- Type D – Internal road/highway
- Type E – General storage/car parking areas

It was required that Types A1 and A2 would be jointed reinforced concrete pavement and the remaining types would be flexible paving comprising of hot rolled asphalt/dense bitumen Macadam.

FIGURE 6-6

The RoRo and Container Terminal



7. Conclusion

This paper has outlined the master planning stages of the Tilbury2 project for the years 2016 to 2019.. The timetable for development is as shown in Table 7-1.

TABLE 7-1:

Timetable for the development process

Activity	Date
Site purchased by Port of Tilbury (London) Ltd	2015-2016
Masterplanning	2016- 2017
Development Consent Order application	2017-2019
Development Consent Order Awarded	March 2019
Design and Construction Process	2019- 2020
Berthing Trials	June 2020
First Operations commence	May 2020

The major features of the plan have been taken through to implementation, except that the rail connected warehousing operation was eventually not developed because an alternative solution was found in the main Tilbury dock. This means that the area available for RoRo and container on RoRo operations could be significantly enhanced.

The terminal has come on stream at just the point that Brexit has become a reality. While Brexit is expected to have a negative effect on trade to Europe, this has not depressed demand for this new service. Indeed, since most existing RoRo services will be to some extent affected by the new and more controlled Brexit processes, Tilbury2 offers a welcome addition to the national ports capability that will undoubtably help to offset logistical problems at other ports.

Meanwhile, the Tilbury B Power Station site has become available and Port of Tilbury (London) Ltd were sufficiently confident in future prospects to purchase the additional land.

The UK's newest major port development evidently has a bright future.

FIGURE 7-1

The UK's newest major port commencing operations

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02: Rethinking Inclusive Public Toilets: Exemplar Standard and Best Practice to Meet Needs of Multiple User Groups

Abstract

HS2 believes that an inclusive design approach will make the network easier for everyone to use. The design will be based around the people using them and developing a better understanding of users and their needs is therefore key to defining the best solutions and informing new standards and best practice.

An opportunity for developing a new standard was identified to address a gap between the provision of 'Accessible Toilets' and 'Adult Changing Places' facilities in BS8300[1]. Bridging this gap will improve the usability of toilets on station platforms, both for wheelchair users and for people requiring specific features for a range of needs. This provision is additional and exceeds current best practice.

The EDP multi-disciplinary investigation began with collating information relating to a wide range of users and toilet facilities. It drew upon three toilet layouts from within BS8300[1]. An inclusion/exclusion matrix cross compared the features and configurations with the requirements of a range of users and enabled the identification of gaps in the design considerations.

New layouts were developed and refined through user engagement and stakeholder feedback, into a single square layout with the flexibility to be rotated in different contexts. This study has assisted HS2 towards achieving its strategic customer experience objectives and supports HS2's Inclusive Design commitments. The proposal is now mandated into the HS2 Inclusive Design Standard and will therefore be implemented at all stations and has potential to influence the development of national standards.

KEYWORDS
Inclusive design; Accessibility; Human factors; Architecture

1. Introduction and Purpose

If projects such as HS2 are to 'ambitiously change the structure of society'[2], then it is essential that, in completing these projects, new standards are established in how they are designed and delivered as this is one way in which change can be brought about.

This technical paper describes the process by which a new standard has been set for the design of inclusive toilet facilities in the platform areas of HS2 stations. Engagement with subject matter experts suggests that this standard can also be used as the catalyst for updating the applicable British Standard, BS8300-2:2018 Part 2, 'Design of an accessible and inclusive built environment'[1], and therefore has potential to influence the provision and design of inclusive toilet facilities nationally.

Compared to other aspects of HS2, this is a relatively small-scale element of design. However, it is one which can have significant impact on the quality of customer experience and the extent to which different user groups feel included. This is evidenced by the results of the National Rail Passenger Survey[3] where a significant proportion of station users rate facilities at existing stations as either poor or very poor.

The design of the accessible toilet has changed little since the layout was first standardised in BS 5810:1979, 'Code of practice for access for the disabled to buildings'[4]. This layout was first included in the Building Regulations in 1987. This design was updated and validated with ergonomic studies and included in the publication of BS 8300:2001, 'Design of buildings and their approaches to meet the needs of disabled people'[5], but the layout and contents remained essentially the same. Though subsequent revisions of the British Standards enlarged the footprint, the layout and design thinking has remained principally unchanged for over 40 years.

Work in developing the inclusive design approach to the London 2012 Olympics changed the approach to design thinking and development of inclusive design standards, including the British Standard BS 8300[2] and the RIBA Plan of Works 2020[6] through recognition of the need for critical design thinking about how the needs of different user groups are met. This is recognised in the HS2 Inclusive Design Strategy[7] and commitment to deliver inclusive solutions that meet the needs of its customers. This approach creates significant opportunities for HS2 to build on the inclusive design legacy of London 2012 and the output of this study represents one such opportunity.

In 2019, HS2 and Atkins, part of the joint venture acting as the Engineering Delivery Partner for Phase 1, identified that significant benefit could be achieved by providing enhanced accessible toilet facilities on HS2 station platforms that would serve a wide range of user groups, from people who use walking aids and wheelchairs to people who have neurological needs, or parents with children.

Between November 2019 and May 2020, a team from Atkins, including Human Factors Consultants, Access Consultants and Architects, worked collaboratively with HS2 to investigate requirements and develop a design for an enhanced facility that meets the needs and preferences of as many people as possible.

This paper provides an overview of the research and resulting proposal for a new standard design. This has now been formally adopted within the HS2 standards and project requirements and will be implemented across HS2 stations. This study should result with the provision of approximately 18 enhanced facilities across the four Phase 1 stations.

2. Methodology

The aim of this study was to go beyond the existing guidance and standards available. Therefore, a methodology was required that enabled analysis of current British Standards against research user requirements to identify where these needs are not being met.

1. Literature Review: Research into available standards and written resources
2. Challenging standards through use of an analytical matrix; checking the accessibility needs of users against provisions,
3. User and Stakeholder engagement,
4. Design development,
5. Ergonomic analysis,
6. Design refinement.

3. Literature Review

The study started with a comprehensive literature review to understand current guidance for the design of accessible public spaces and toilet facilities. This section outlines the information and understanding gained from the key documents that input into this study.

3.1. BS8300-2:2018 – 'DESIGN OF AN ACCESSIBLE AND INCLUSIVE BUILT ENVIRONMENT'[1]

This British Standard provides recommendations for the design of buildings to accommodate a range of characteristics and capabilities. This part of BS8300 refers to transport buildings, along with other public facilities. Most importantly, BS8300 provides recommendations for accessible toilet accommodation. These recommendations formed the basis of this study and analysis.

3.2. CHANGING PLACES: THE PRACTICAL GUIDE[8]

The Changing Places Consortium launched a campaign in 2006 for all large public spaces to have Changing Places facilities installed, enabling all disabled people, and particularly people requiring assistance to travel, with the same dignity as everyone else. This report focuses on the changing places facility. Whilst the enhanced platform facility is not intended to be a changing places facility, this guide provides detail around the features of the facility and outlines important considerations which should be included in the design of an accessible facility.

3.3. UNIVERSITY COLLEGE LONDON RESEARCH STUDY – ACCESSIBLE TOILET RESOURCE[9]

This resource was co-authored by Jo Bichard, who has conducted extensive research into the accessibility of public toilet facilities. It focusses on analysing the design of existing accessible toilet cubicles, stating that 'it is essential to provide these facilities so that disabled people can participate on equal terms.' The resource provides case studies and design recommendations related to the requirements of certain user groups. This was used as a basis for the design of the enhanced facility and informed the Inclusion/Exclusion Matrix analysis.

3.4. BICHARD (2004) – EXTENDING ARCHITECTURAL AFFORDANCE: THE CASE OF THE PUBLICLY ACCESSIBLE TOILET[10]

Jo Bichard completed a PhD qualification focussing on accessible toilet facilities. This report uses design guidance and user experiences to analyse existing facilities and demonstrate that current designs create barriers that prevent wider access to the city and engagement with public facilities, including education and work. The premise for this research is aligned with the enhanced platform toilet study and has informed the Inclusion/Exclusion Matrix analysis.

3.5. GOOD LOO DESIGN GUIDE (2004)[11]

This guide was published by the Centre for Accessible Environments (CAE) and the Royal Institute of British Architecture (RIBA) and takes an inclusive approach to the provision of toilets that are designed for a range of users. The guide gives guidance on several toilet layouts and advice on potential conflicts between the needs of different user groups.

3.6. CENTRE FOR ACCESSIBLE ENVIRONMENTS – MANAGING ACCESSIBLE TOILETS FACTSHEET (JAN 2017)[12]

The CAE has conducted research into a range of accessible spaces. This high-level report outlines the difference between an accessible and an ambulant accessible toilet. It includes recommendations such as providing more than one transfer from wheelchair to WC pan option where multiple facilities are available. The most valuable information obtained from this document is the list of management recommendations, the majority of which are applicable to the enhanced platform facility.

3.7. SPORTS ENGLAND: MAPPING DISABILITY – THE FACTS (2016)[13]

This Sport England fact file contains a wealth of information relating to the demographics of people and their requirements. The purpose of the document focuses on the inclusion of disabled people in sport; however, the demographics included provide a valuable representation of the proportions of the population with disabilities in support of this enhanced platform toilet study.

4. User Groups

Following the literature review, the next stage was to determine the user needs for an accessible and inclusive toilet facility. This required identification of user groups and key design elements that impact their accessibility to the facility.

The user groups considered within this study were identified through research and contact with authors of existing studies.

The individuals identified requiring consideration were individuals:

- with learning difficulties,
- who are neurodivergent,
- living with dementia,
- with mental health needs,
- who are blind and/or partially sighted,
- who are deaf or hard of hearing,
- with continence needs,
- with paruresis/shy bladder,
- with an ileostomy/colostomy,
- with stamina/health difficulties,
- with arthritis/dexterity difficulties,
- with limited upper body strength,
- with short or tall statures,
- who use wheelchairs/walking aids,
- travelling with children/babies,
- travelling with a carer/companion,
- travelling with luggage.

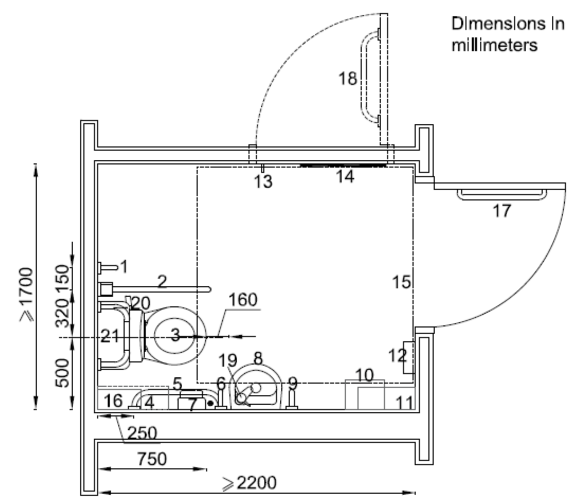
Having identified the user groups, an investigation was required to understand the needs of each user group and determine how these would be addressed in the design. This understanding was achieved through desktop research and communication with Jo Bichard, who has previously conducted research with University College London (UCL) into the accessibility of public toilets[9] which involved engagement with several charities and representative groups.

5. Summary of Inclusion/Exclusion Matrix

The three standard layouts for accessible toilet facilities given in B8300-2:2018[1] were analysed using an Inclusion/Exclusion Matrix, comparing the user requirements of the identified user groups against the provision and arrangement of features. The three layouts are demonstrated in Figures 1 through 3. (Figure 8 further in the paper presents a summary table of the matrix analysis for each layout formation and incorporates stakeholder feedback).

FIGURE 1

Unisex accessible toilet derived from Figure 40 in BS8300



1 - Vertical grab rail
2 - Drop down support rail
3 - Flat-topped close-coupled cistern
4 - Sanitary disposal unit
5 - Horizontal grab rail
6 - Vertical grab rail
7 - Toilet roll dispenser
8 - Hand rinse basin projecting not further than 250mm
9 - Vertical grab rail
10 - Disposal bin
11 - Shelf, 760mm above FFL
12 - Sanitary disposal bag holder

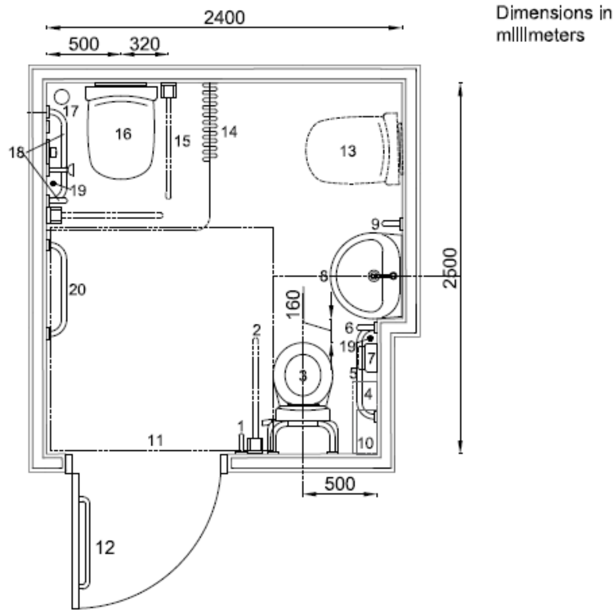
Note: Examples shown are for right - hand transfer to WC

13 - Two clothes hooks, one at 1050mm and one at 1400mm above floor
14 - Long mirror
15 - Wheelchair turning space, 1500mm x 1500mm
16 - Colostomy bag shelf
17 - Horizontal pull rail
18 - Alternative door position
19 - Tap on side of basin nearest to WC
20 - Flush mechanism on this side of WC pan
21 - Flat -topped close - coupled cistern providing a beck rest and a colostomy bag changing surface for standing user
22 - Alarm pull cord

The layout shown in Figure 1 is the standard WC facility provided in public buildings for people requiring an accessible WC. However, the exclusion matrix analysis has highlighted barriers to its use for some user groups. For example, the position of the WC pan only allows for one side handed transfer; therefore, wheelchair and scooter users with a preference for the opposite side are not accommodated and likely to experience difficulty when using the facility. In comparison to Figure 2, the wash basin provided in this arrangement is smaller and positioned at a lower height, which may be awkward for some users. The smaller size basin also reduces the washing ability, particularly for individuals who require these facilities for changing a colostomy or ileostomy bag and those with limited dexterity.

FIGURE 2

Corner WC layout with shower facility derived from Figure 30 in BS8300



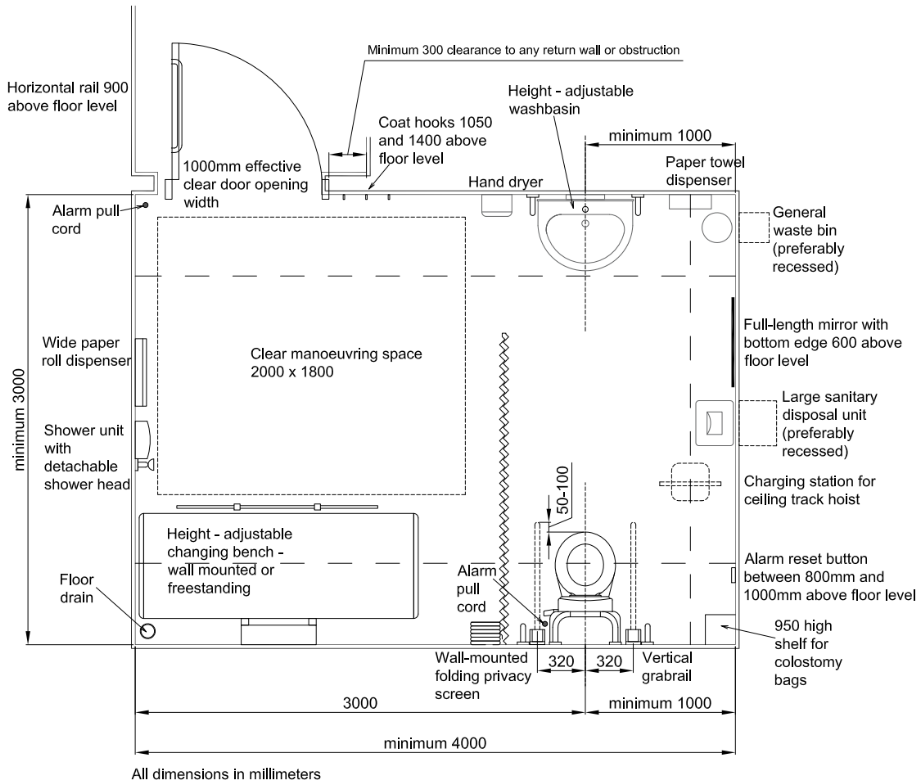
1 - Vertical grab rail
2 - Drop down support rail
3 - Flat-topped close-coupled cistern
4 - Sanitary disposal unit
5 - Horizontal grab rail
6 - Vertical grab rail
7 - Toilet roll dispenser
8 - Hand rinse basin projecting not further than 250mm
9 - Vertical grab rail
10 - Colostomy bag shelf
11 - Wheelchair turning space, 1500mm x 1500mm
12 - Horizontal pull rail to help close the door from a wheel chair
13 - Optional tip-up seat for use when drying
14 - Shower Curtain
15 - Drop down support rail
16 - Tip - up shower seat
17 - Floor Drain
18 - Vertical and horizontal grab rails
19 - Alarm pull cord
20 - Towel Rail

Note: Examples shown are for right - hand transfer to shower seat and WC

The facility configuration in Figure 2 includes a full shower facility with screen, corner WC and tip up seat. The key barriers for use for this layout include the position of the shelf which is required for changing pads and bags. This position is not within reach of wheelchair users or individuals who are short in stature. The size of the facility limits the ability for larger features to be included, such as baby changing facilities, resulting in the need for individuals travelling with children and babies to continue to the concourse for other facilities. The position of the wash basin is too low for tall users and individuals using walking aids such as crutches who require access from a standing position. The matrix analysis also highlighted the provision of washing facilities as particularly beneficial if required but it was noted that these could cause confusion for some users with neurological difficulties.

FIGURE 3

Changing Places facility layout derived from Figure 48 in BS8300



The key purpose of the Changing Places facility shown in Figure 3 is to provide space provision for assisted use; therefore, the facility is much larger than a standard accessible toilet. This enables access for even the largest wheelchair and scooter models, but it is not advisable to use this facility without assistance due to the positioning of the features. The provision of a single wash hand basin opposite the WC means that hand rinsing cannot be done whilst on the toilet pan, which presents a hygiene problem.

6. Stakeholder Engagement

After initial design layout formulation based on the BS8300-2:2018[1] layouts and analysis output from the matrix, it was essential that further development incorporated feedback from end-user input. In total, six telephone interviews were conducted with a range of representative users with varying requirements and experience travelling across the UK rail network.

The interviewer provided an overview of the study and the key design decision and features of the possible layouts. The participant was then invited to consider a sequence of use for the facility and consider whether their specific requirements had been met by the configurations.

The comments from the participants were noted during the interviews (see Table 1 below) and subsequently discussed and applied to each configuration. The comments that are specifically relevant to the context of this facility are highlighted in bold.

TABLE 1:

Summary of User Engagement Feedback

Participant	Requirement	Comments
A	Luggage/children/ elderly	› **Position of luggage storage space next to door** › Travelling with children – unisex facilities beneficial › Level entrance preferable for elderly users › Consider complexity of locking mechanism for people living with dementia › Provision of luggage space is optimal
B	Baby changing facility	› Shelf of suitable size to put down nappies › **Hook closer to baby changing table would be useful** › **Babies may find noise of handdrier uncomfortable** – quiet model would be preferable › Consider potential for people to queue at facility – in the path of other passengers? Signage to other facilities required

Participant	Requirement	Comments
C	Travelling with children and babies	› Importance of maintenance – hygiene › Coat hook close to changing mat would be helpful › Is there access to the bins when the changing table is down? › Button mechanism locks could be a problem – children playing with locks whilst parent using WC › Retractable child seat where parents can put child so they can use WC would be beneficial › Paper sheets for changing table – nice but not essential › Provision of nappies/wet wipes would be beneficial; most users carry them but always risk of running out. This would need to be controlled – potential for abuse › Option of smaller toilet seat for children › If feeding room is provided elsewhere in the station, then signage to facility useful at accessible toilet
D	Travelling with luggage	› Proximity of facility to lift is important › **Consider accidental activation when positioning the hand dryer** › Other immediate requirements – coeliac may also require emergency use of facility
E	Children/baby changing/ pushchairs	› **Sink close to baby changing table** – ability to stay within one step of table › Noise of hand dryers scary for children – provision of paper towels is preferable › Somewhere to put both dirty and clean items when changing baby › Something to distract/grab child's attention whilst adult uses facility – perhaps artwork or puzzle › Provision of nappy bags would be helpful and help hygiene too

Participant	Requirement	Comments
F	Wheelchair user (left-handed)	› Hygiene and cleanliness are of upmost importance - wheelchair users are more susceptible to bladder and kidney infections › Preferred transfer is from side and would avoid using WC pan to support › Position of pull-down seat for changing would be beneficial in instance of accidents › Position of soap, paper towels and hand dryer close to wash basin reduces manoeuvring requirements › Provision of sanitary wipes for cleanliness is beneficial › **Peninsular layout is preference – opposite transfer side can be stressful. Peninsular WCs allow transfer from either side.** › Provision of facilities with alternate transfer provision would reduce waiting time and accommodate more users. It would reduce the stress of travelling to other locations within the station › Recommendation of foil cover of toilet seat which can be changed, as is practice in USA or full cubicle wash/dry mechanism in parts of Europe › Enhanced size accommodates family use with children which removes requirement to use facility twice and saves time

7. Design Development

The design development stage allowed progressive design decision making to refine the layout whilst taking user requirements and accessibility into consideration. This process enabled the identification of preferable feature groups and key layout trade-offs. The design progressed with additional provisions to improve inclusivity.

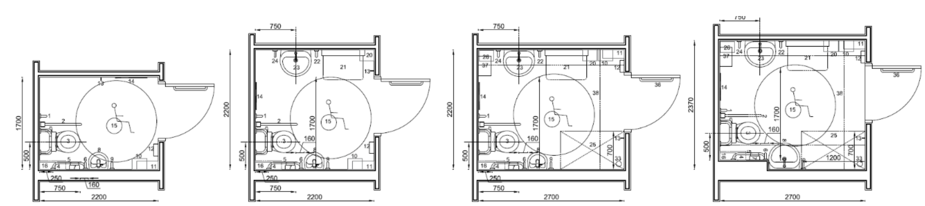
This development was captured through production and review of plan and elevation layouts, Figure 4, Figure 5 and Figure 6, a 3D model (Figure 7) and the Inclusion/Exclusion Matrix in Figure 8. This information was also used for stakeholder and end user engagement.

As shown in Figure 4, initial design development focussed on the standard accessible toilet corner layout, which is suitable for self-propelled wheelchair and wheelchair users with limited dexterity. This configuration lacks the additional space requirements for users travelling with children and infants or luggage.

The wash basin in front of the WC pan is smaller, lower and within reach from the WC, allowing users to wash their hands before touching mobility aids and wheelchairs to transfer. This design feature has remained consistent throughout the design development process for this hygiene related reason.

FIGURE 4

Design development sketches

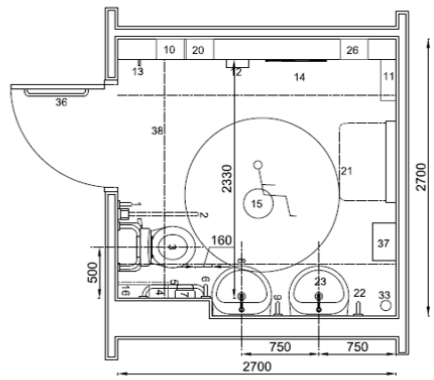


The most notable design development is the gradual increase in the size of the facility (as demonstrated in Figure 4). Throughout this process, it was found that the larger the cubicle, the greater the ability to include larger features such as the baby change bench and second, larger wash basin. The increase in size also provides the capability to better position the shelf required for changing pads, colostomy bags or changing clothes. Accommodation of these provisions provides facilities required by a wider range of user groups. Following dialogue with HS2 stakeholders, a further development resulted in designing to a square layout as this would enable the layout to be rotated to suit different locations without needing to make changing to the interior. This has benefits including consistency of arrangement across all locations and therefore consistency of customer experience. The next design development exercise sought to determine the optimum configuration within a square facility.

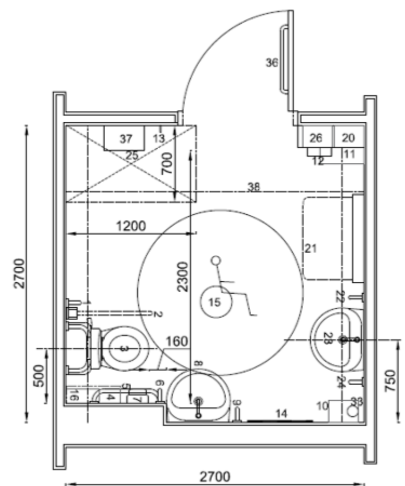
FIGURE 5

Square facility configuration development

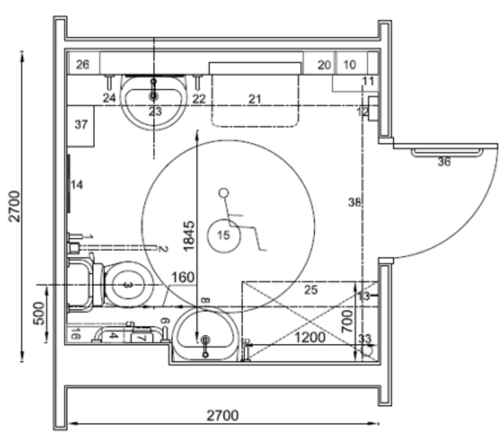
A



B



C



The three layouts presented in Figure 5 are all the same size. However, the key differentiator is the position of the door in relation to the WC pan, which influences the arrangement of the interior features. The configuration shown in 'A' has both wash basins adjacent to one another. This is not deemed optimum as it may cause confusion for some users. Layout 'B' presents manoeuvrability challenges as the door position in relation to the WC pan narrows the turning space for wheelchair and scooter users. Of the three layouts, 'C' is considered the most accessible, with provision for luggage and pushchair storage as well as ample manoeuvrability space and inclusion of both a smaller and larger wash basin at opposite sides of the facility. The positioning of the mirror in layout 'C' raised concern due to the risk of damage from contact with wheelchairs and scooters as users position themselves alongside the WC pan, and this influenced further refinement of the layout.

The study identified the following design considerations which are paramount for consideration during the detailed design stage:

- A lightweight and easily opened door is preferable for limited dexterity and upper body strength users and for wheelchair and scooter users who will require control from a seated position;
- Transfer may be limited by the handing of the layout. Where constraint allows, it would be preferable to provide more than one facility within close proximity, enabling right- and left-handed transfer preferences to be catered for;
- Consider design specifications for soap and paper towel dispensers. Sensor mechanisms are preferable as they remove the requirement for strength and dexterity. They also assist with maintaining the hygiene of the facility;
- Similar strength and dexterity considerations should be given for flushing mechanism. Consider provision of two flush operation options – paddle flush and sensor;

8. Ergonomic Analysis of Square Layout Options

An ergonomic assessment was conducted to ensure the manoeuvrability space was adequate for a range of wheelchair and scooter specifications.

PD ISO TR 13750-2: 2014[14] provides four principal groups for wheelchairs and scooters.

Discussion with HS2 enabled definition of the wheelchair type anticipated to access the HS2 service. It was agreed that the enhanced inclusive toilet facility should be required to cater for the type 2 mobility scooter as this type is suitable for both indoor and outdoor environments and is the maximum size likely to be used on the HS2 route.

This assumption was used as the basis for the ergonomics manoeuvrability assessment, along with consideration of users who are accompanied by carers or those who carry personal items, such as luggage and pushchairs, that will impact the space requirements.

8.1. FIGURE 5A – MANOEUVRABILITY ANALYSIS

Use of this facility layout by an accompanied wheelchair user would result in reduced ability to manoeuvre. It would be possible for a type 2 wheelchair or scooter user to access this facility; however, if baby changing was also required, they would likely experience difficulty completing a 180-degree turn, therefore requiring entry or exit in reverse.

8.2. FIGURE 5B – MANOEUVRABILITY ANALYSIS

The position of the door has resulted in the reconfiguration of some of the interior features. The pushchair space encroaches on the entrance space; however, space has been allowed alongside the WC for transfer, which combined with the pushchair space, provides adequate provision for stowing luggage and accessing the WC. This facility layout design provides space to accommodate a wheelchair user and carer, if required, although the purpose of the facility is primarily for independent use.

8.3. FIGURE 5C– MANOEUVRABILITY ANALYSIS

The manoeuvrability for wheelchair and scooter users with this design layout is more comfortable, with additional capability for turning provided by the provision of the dedicated space for pushchairs/luggage. Furthermore, space is only reduced if use of the baby changing bench is also required. The transfer side of the WC should be clearly indicated on the signage outside the facility as right-hand transfer handlings will not be preferable for all users.

This manoeuvrability assessment has highlighted the critical factors that should be considered in the final design to provide manoeuvrability space within the facility is adequate. Table 2 provides the manoeuvrability considerations and a description of the requirement for each.

TABLE 2:

Critical factors that influence manoeuvrability

Manoeuvrability Considerations	Detail
Size of wheelchair/ electric scooter	The larger the wheelchair/electric scooter, the greater the manoeuvrability space required. Electric scooters also have larger turning circles due to their wheel mechanisms.
Is the individual accompanied?	Whilst the cubicle is intended for independent use, it may be that some users require assistance and therefore will be accompanied when using the facility.
Position of door	The position of the door on the side of the cubicle provides larger turning space in the centre of the cubicle.
Stowable features	The position of features influences the manoeuvrability space available. For example, the grab rails protrude the most and therefore have the largest impact.
Inset features into walls	Features such as bins, shelves and paper towel dispensers that could be inset into walls would further reduce infringement on cubicle space, increase manoeuvrability space and make features easier to use.

9. Proposed Standard Design for HS2

The proposed standard design and the research approach have been presented to the Public Toilets Research Unit, part of the Helen Hamlyn Centre for Design at the Royal College of Art, in June 2020. The design received very positive feedback and it is hoped that HS2 will continue to work with the research unit and feed findings into the British Standards Institute to further the impact of the research.

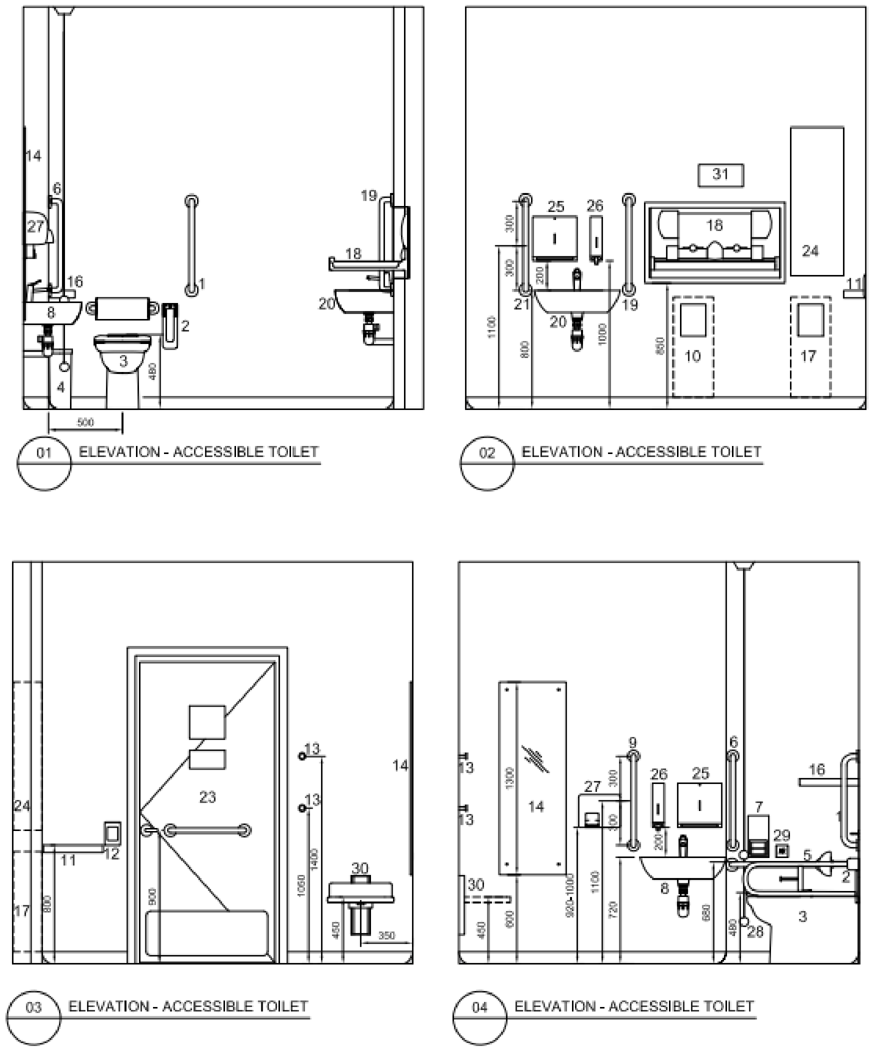
Following the design development process and end user input, a proposed standard design was produced (presented in Figure 6). The position of some features was adjusted in line with user engagement feedback and refined feature grouping. The previously noted concern regarding the position of the full-length mirror was reconsidered and subsequently the mirror was repositioned to overlap the space provision for luggage and push chairs. This position reduces the risk of damage by contact with wheelchairs and removes the confusion that can be caused by its position directly opposite the entrance. Additionally, a hand dryer was included due to the comparative ease of use for individuals with limited dexterity in comparison to paper towels which require grip strength. The hand dryer has been positioned alongside the mirror to minimise the risk of unintended activation which could be scary for people who find the sound distressing. The inclusion of the cavity wall has been maintained as the ability to inset features such as the bins and paper towel dispenser increases manoeuvrability space.

The square facility allows for rotation of the facility without interference with the interior arrangement, enabling installation across all HS2 station platforms in a consistent manner.

Figure 8 presents the Inclusion/Exclusion Matrix and provides analysis of the proposed standard design. This allows for a direct comparison between the proposed standard HS2 design and layouts currently included in the BS8300 standard. This analysis indicates that the requirements of the majority of users have been taken into consideration, although exclusion of the hoist mechanism has excluded individuals with complex needs and assisted use. Facilities that cater for these requirements are located elsewhere on the station concourses. On the basis that users requiring such a facility are likely to have planned their journey in advance and have knowledge of where these facilities will be located in stations, this exclusion is considered appropriate.

FIGURE 6

Elevation and plan drawings of proposed standard design



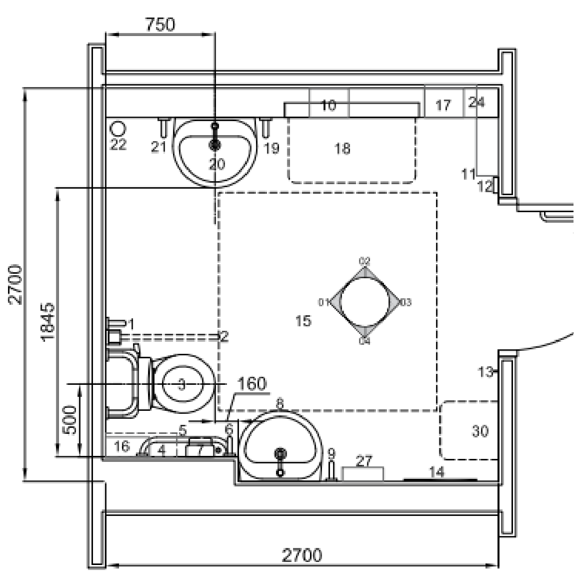


FIGURE 7

View of 3D model of proposed standard design illustrating position of features

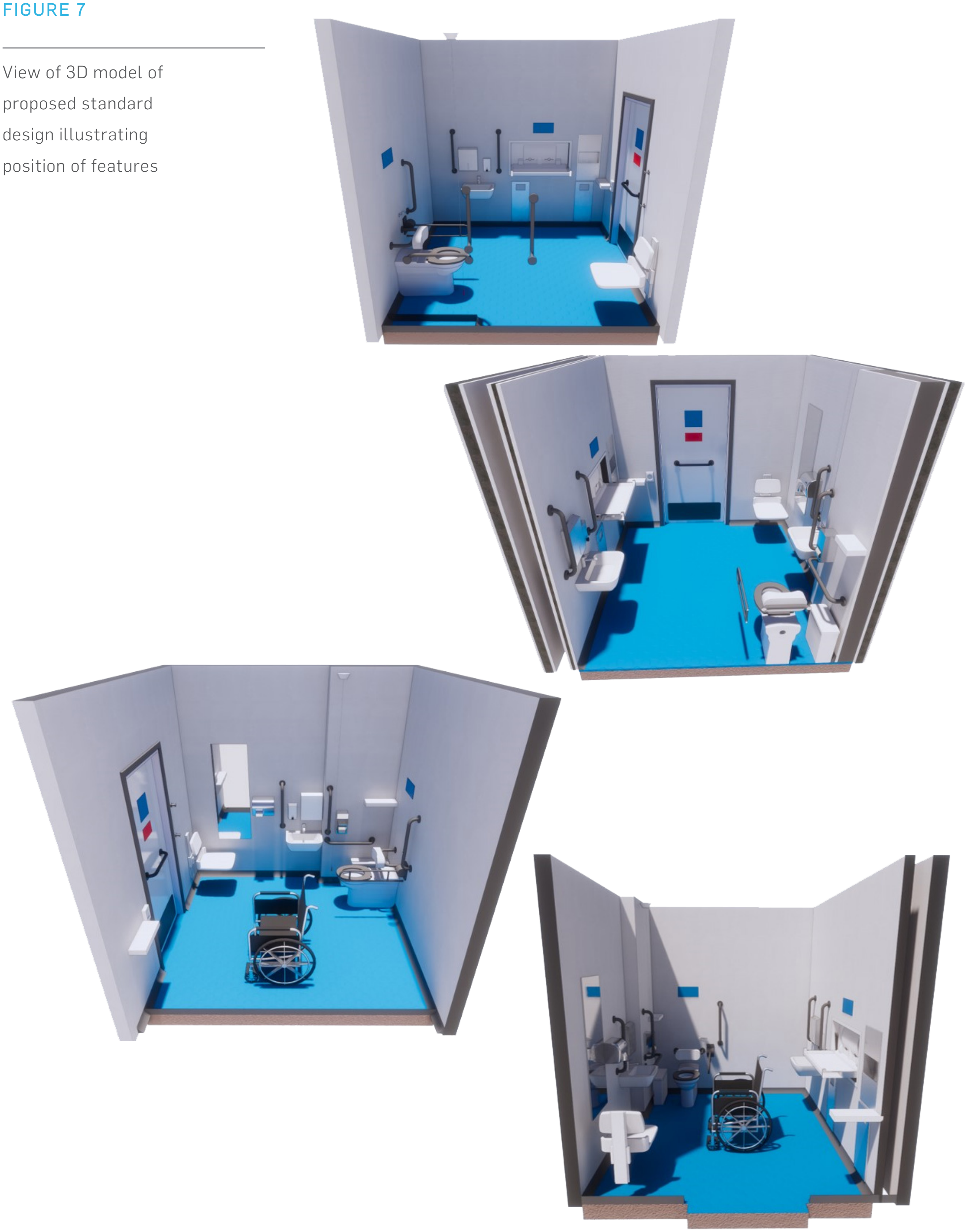


FIGURE 8

Inclusion/Exclusion

Matrix summary table

		Layouts given in BS8300			
		Figure 30 - Ensuite shower room with corner WC	**Figure 40 Unisex wheelchair toilet with corner WC**	**Figure 48 Changing Places Toilet**	**Enhanced Platform Toilet Facility**
Visual	**Blind/Partially Sighted**	Contrast Tactile	Contrast Tactile	Contrast Tactile	Contrast Tactile
Mobility	**Wheelchair User with limited coordination**	Transfer may be limited by handing of layout	Transfer may be limited by handing of layout	Not advised for independent use	Transfer may be limited by handing of layout
	Wheelchair user with limited dexterity	Transfer may be limited by handing of layout	Transfer may be limited by handing of layout	Not advised for independent use	Transfer may be limited by handing of layout
	User of small manual wheelchair	Toilet seat height may be an issue	Toilet seat height may be an issue	Not advised for independent use	Manoeuvrability space provided
	User of small manual wheelchair with carer	Transfer may be limited by handing of layout	Limited space for companion	Companion present	
	Person with muscular, skeletal or spinal injury (wheelchair or crutches)	Transfer may be limited by handing of layout and there is a low hand basin	Transfer may be limited by handing of layout and there is a low hand basin	Not advised for independent use	Transfer may be limited by handing of layout
	Person who is tall or of heavy build	Low hand basin	Low hand basin	May cause problems for independent use	
	Person who is of short stature	Toilet seat height may be an issue	Toilet seat height may be an issue	May cause problems for independent use	
	Person who has limited upper body strength	Transfer may be limited by handing of layout	Transfer may be limited by handing of layout	Not advised for independent use	Transfer may be limited by handing of layout
	Person who has arthritis and is a scooter user		Limited space for scooter	Not advised for independent use	
	Person who has arthritis and is a walking aid user	Low hand basin	Low hand basin	Not advised for independent use	
	Person who has complex needs and/or requires hoist	Limited facilities for companion to assist	Limited space for companion	Companion present	Signage provided for alternative facility if required
Auditory	**Deaf and hard of hearing**				
Neurological	**Person who has shy bladder syndrome**				Enclosed cubicle
	Person with a learning disability		Limited space if requiring assistance	Particular complexity is without assistance	More space assistance
	Person who is on the autistic spectrum		Limited space if requiring assistance	Particular complexity is without assistance	More space assistance
	Person who is living with dementia		Limited space if requiring assistance	Particular complexity is without assistance	More space assistance
Health Condition	**Person with an urostomy**		Limited facilities for user		
	Person with irritable bowel syndrome		Limited facilities for user		
	Person with a colostomy bag		Limited facilities for user		
	Person with incontinence		Limited facilities for user		
Other	**Person with carer/companion**	Limited facilities for companion to assist	Limited space for companion	Companion present	
	Person with particular faith or culture				
	Parent with child/baby	Limited facilities for user	Limited facilities for user	Parent present	

Key

No significant restrictions to users
Some restrictions to users
Significant restrictions to users

10. Conclusions and Recommendations

This study has demonstrated the value of critical design thinking in developing solutions that can meet the needs of a wide range of users for either minimal or no extra capital cost. This has been made possible through a combination of willingness on the part of HS2 to challenge existing standards and best practice, and the range of multi-discipline expertise made available through the HS2 Engineering Delivery Partner (Atkins). The study is an example of how standards and best practice can be redefined through collaboration in delivery on a major project.

Since the completion of this study, the proposed standard design has been adopted as the standard layout for public platform toilet facilities on HS2 stations. This has been mandated through the update of the HS2 Phase 1 Project Requirement Specification[15] and the HS2 Inclusive Design Technical Standard[16]. The incorporation of this standard layout into station design requires consideration of the following recommendations to ensure successful design development.

- Toilet facilities at HS2 stations are one of a number of customer-facing Station Common Design Elements. These are designed elements which HS2 require to have varying degrees of commonality across HS2 stations. To help ensure ease of use and maintenance, toilet facilities are required to have a high degree of commonality. At the time of writing, detail design for these elements is yet to commence, and it is intended that the detailed design for the enhanced toilet facility will be developed as part of this workstream to ensure consistency in delivery across the stations. It is also important that the design approach taken in this study is shared with the relevant parts of the HS2 supply chain as a practical demonstration of the HS2 commitment to inclusive design.
- Although it has received positive feedback from subject matter experts, the new standard design is yet to be user tested as a physical prototype. It is therefore recommended that further user testing is carried out by means of a prototype to further refine detail design aspects.

- During the course of this study, a number of design considerations have been identified which will be important in ensuring that the detailed design of the enhanced toilet facility meets the intent of the new standard arrangement. These considerations include acoustics, lighting, door and ironmongery specifications and detailed specification of fixtures and fittings. It has also been identified that these considerations can be applied to the design of other types of customer toilet facilities to enhance inclusion overall. A recommendation arising from this study is that HS2 and EDP consider how these considerations can be addressed at the next design stage.
- 'Look and feel' is a key consideration in developing HS2 station architecture. Stakeholder feedback received during this study has highlighted the importance of a softer approach to design of accessible facilities to prevent them being differentiated from other customer facilities as institutional and of lower quality design. It is recommended that this is considered in further design development.
- It is recommended that the output of this study is shared with relevant academic research units, industry bodies and the British Standards Institute to promote development of industry and national inclusive design standards.
- During the course of this study, a number of design considerations have been identified which will be important in ensuring that the detailed design of the enhanced toilet facility meets the intent of the new standard arrangement. It has also been identified that these considerations can also be applied to the design of other types of customer toilet facilities to enhance inclusion overall. A recommendation arising from this study is that HS2 and EDP consider how these considerations can be addressed at the next design stage.

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03: Great Crested Newt Licensing for Major Infrastructure Projects: Shifting the Paradigm

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Abstract

Protected species constraints are an early consideration for nearly every development project, and where impacts cannot be avoided, mitigation licensing is a legal requirement. HS2 Ltd are the licensee for a great crested newt organisational licence, as part of the Phase One works; the largest such licence ever issued by the regulator for a single development.

This paper identifies how the standard paradigm for protected species licences and their management had to be completely re-designed and new structures, processes and templates developed and implemented to ensure effective use of a limited resource and cost-effectiveness in delivery.

The measures and processes that have been established, through careful design, an understanding of lessons learnt and working closely with stakeholders, are providing successful management of a complex organisational licence and its delivery. The approach has applications to the other phases of HS2 as well as further application to other development projects, particularly those involving linear infrastructure.

KEYWORDS

Great crested newts; Licensing policies; Method statement; On-boarding; Organisational licence

1. Introduction

Detailed surveys undertaken between 2012 and 2016 identified the presence (or potential presence) of great crested newts *Triturus cristatus* (Figure 1a), hereafter GCN, throughout the HS2 Phase One (London to West Midlands) route (Figure 1b shows an example of GCN presence within Sector N1, Southam to Bickenhill). The surveys identified 105 GCN meta-populations (a group of connected and interacting populations) within 250m of the route; meta-populations are referred to in Figure 1b as AMPs (assumed meta-populations).

GCN are legally protected by the Conservation of Habitats and Species Regulations 2017 (as amended)[1] and the Wildlife and Countryside Act 1981 (as amended)[2] and undertaking any actions, which would otherwise be illegal, requires licensing by the relevant licensing body Natural England (NE). NE only licence for specific purposes as set out in Regulation 55 of the Conservation of Habitats and Species Regulations 2017[3] and when the licensing tests (commonly referred to as the 'three tests') are satisfied. Due to the potential impacts to GCN and their associated habitats, the Phase One works required a mitigation licence to legally undertake the works.

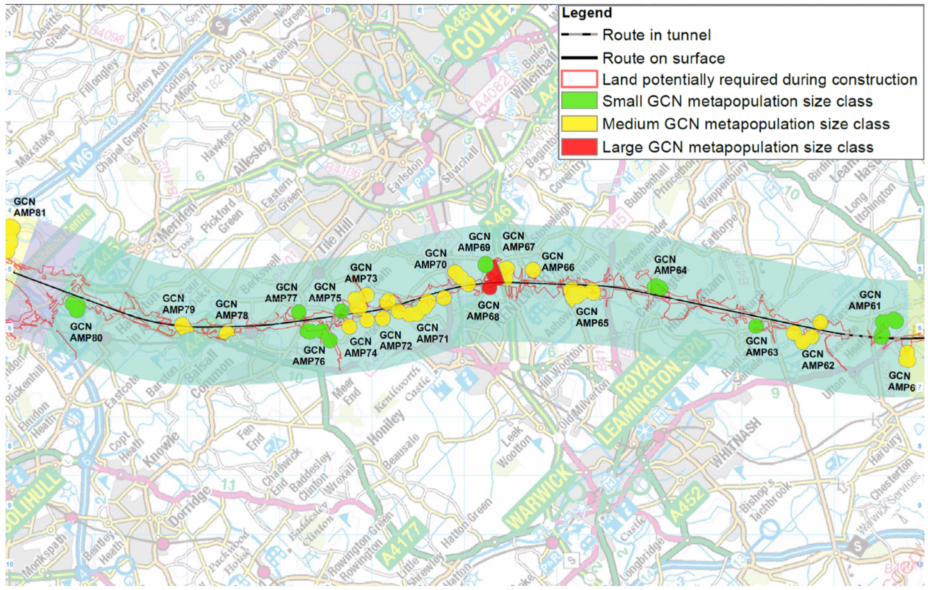
FIGURE 1A

The amphibious great crested newt

Credit: Martin Fowler (Alamy Stock Photo)

FIGURE 1B

Example of GCN assumed meta-populations (AMPs) across Phase One



2. Options for Licensing

The standard GCN licensing approach requires that each work site has a licence. The applications, including all the proposed mitigation and compensation measures to legally allow the work to proceed, would be prepared for each work site and submitted to NE. A standard licence takes several weeks to compile and approximately 30 working days for NE to review. In early discussions with NE it was determined that, due to the number of work sites (defined as works interacting with known AMPs) along the route, a standard licensing approach would be time-consuming, costly and difficult for NE to resource. For example, NE issued 223 great crested newt mitigation licences for England in 2019, with HS2 Ltd (HS2) GCN Method Statements totalling 47 in that year; if submitted to NE via the standard licensing process, this would have resulted in a 21% increase in workload for the NE licensing team.

In addition to the standard licence, at the time that the licence options were being assessed, NE was developing the District Level Licensing (DLL) approach for GCN. At the time of writing, DLL are available in Kent and Medway; Cheshire East, Cheshire West and Chester; Essex; Shropshire; Swindon and Wiltshire; Somerset; Greater Manchester and North Somerset and South Gloucestershire. The DLL approach was not an option for HS2 Phase One due to the locations affected and because the project is permitted by a hybrid Bill rather than subject to a Town and Country Planning Act consent.

3. A Fresh Approach - The Organisational Licence

Organisational licensing[4] is an approach offered by NE for routine activities affecting protected species, where the organisation has consistently met the conditions of other types of wildlife licences in the past. However, an organisational licence had not previously been issued to a development organisation, such as HS2, covering such a wide geographical area and where the impacts, mitigation and compensation measures required were so extensive.

Working together, HS2 and NE developed the existing organisational licence format to be fit-for-purpose for a major infrastructure project. This involved considering GCN at a landscape-scale, rather than an individual population or meta-population approach typical of a standard licence. This approach resulted in the production of two important documents that supported the licence application: 1) Great Crested Newt: Populations and Habitats Assessment, Phase One Route Wide; and 2) Great Crested Newt Habitat Management, Maintenance and Population Monitoring Plan (HMMPMP)[5].

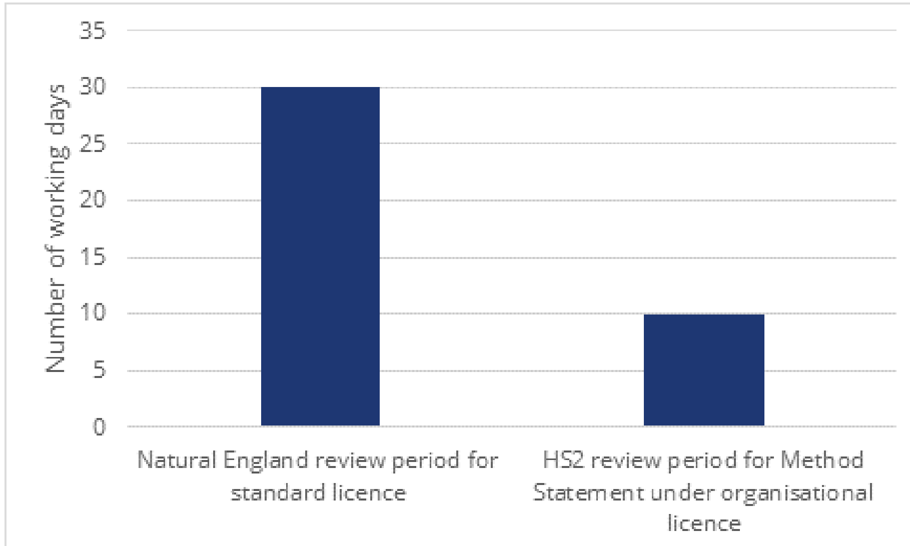
The Populations and Habitats Assessment reviewed and assessed GCN populations and habitat at the route-wide scale in order to inform the Favourable Conservation Status legal test. The Conservation Status is considered ‘favourable’ when the population and range of the species is healthy and will be maintained in the long-term[6]. The HMMPMP adopted a habitat quality approach as the basis for determining success of any GCN mitigation. The commitments made in the HMMPMP are a major step-change from the standard approach of determining success, which has historically always been focussed on numbers of GCN; the disadvantage of the standard approach being that without a long-term, strategic approach to the population and to their habitat and landscape management, GCN populations would always remain at risk from habitat change and stochastic events.

The licence application for HS2 was developed as a novel approach to an organisational licence and in March 2017 the licence was issued by NE to HS2.

This issued licence provides the most efficient approach for NE, as it is a single project licence that is renewed every two years and requires reduced resources to manage compared to a series of standard licences, as the licensee is responsible for reviewing and approving ongoing deliverables, such as Method Statements. The resource input and responsibilities for HS2 (as the licensee) are significantly greater than for the standard licence approach, however, due to having a dedicated ecological resource, Method Statements can be reviewed within two weeks. Therefore, using an organisational licence has substituted the reviewing time from 30 working days (for an individual licence) to 10 working days for a Method Statement (Figure 2) and will have saved approximately 2,000 working days over the lifetime of the HS2 organisational licence.

FIGURE 2

Time taken for review of a standard licence and an organisational licence Method Statement

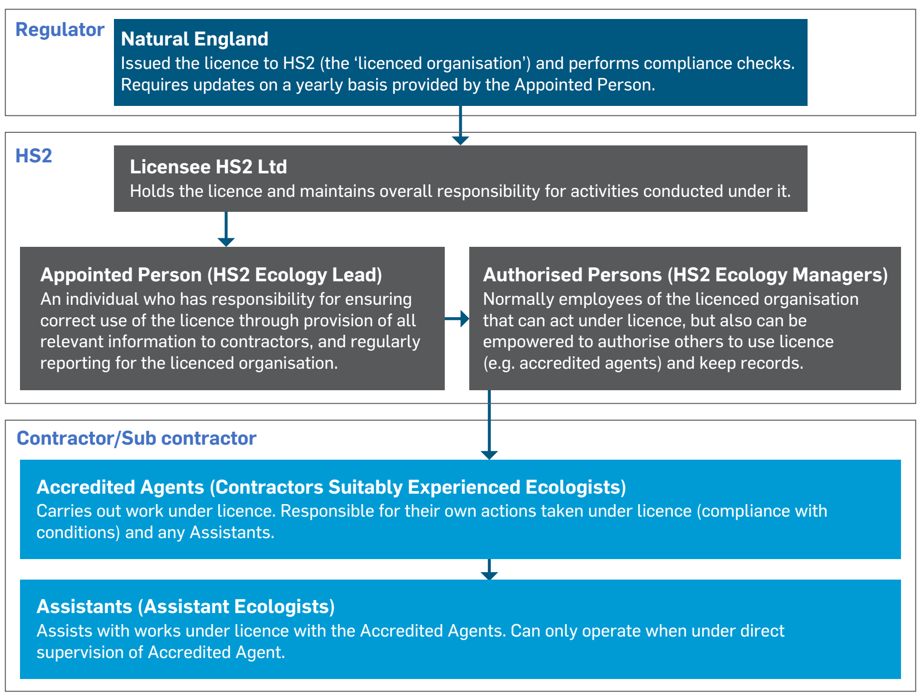


4. Managing the Organisational licence

The licensee (HS2) is responsible for the day-to-day management of the licence and an Appointed Person (Phase One Ecology Lead) acts as point of contact (Figure 3). The Appointed Person can delegate certain responsibilities to Authorised Persons within HS2 (the Ecology Managers). There are currently three Ecology Managers within HS2 all of whom are approved as Authorised Persons. The roles of the Appointed Person and Authorised Persons are novel and nuanced as they are delivery focused (being employed by the licensee and working very closely with Contractors who are undertaking the GCN impact assessments and developing and implementing mitigation), but also involve providing a regulatory function by approving the Contractors Method Statements and monitoring compliance of their implementation.

FIGURE 3

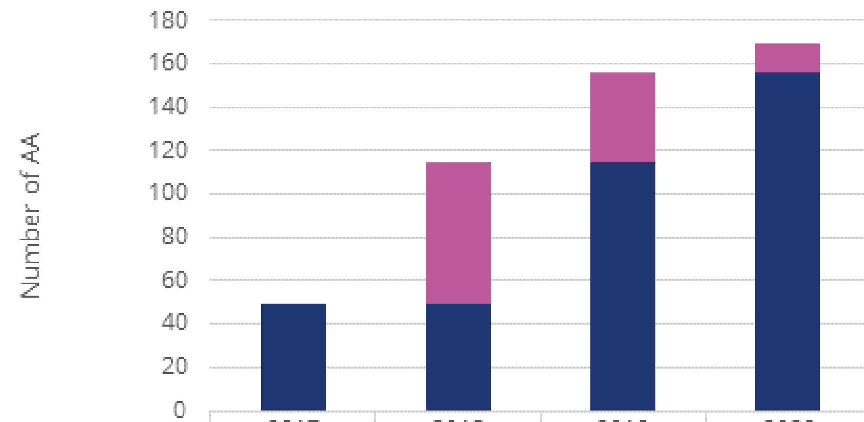
Organisational licence roles and responsibilities



Implementation of the mitigation and compensation measures is the responsibility of the Accredited Agents (AA), who are HS2 Contractors ecologists approved to work under the licence by the Authorised Persons. In 2017 there were 49 successful applications for AA under the HS2 organisational licence and this has been increasing each year where there are currently 169 approved (June 2020) as AA (Figure 4).

FIGURE 4

Showing the number of Accredited Agents approved each year



	2017	2018	2019	2020
Additional AA	0	66	41	13
Number of AA	49	49	115	156

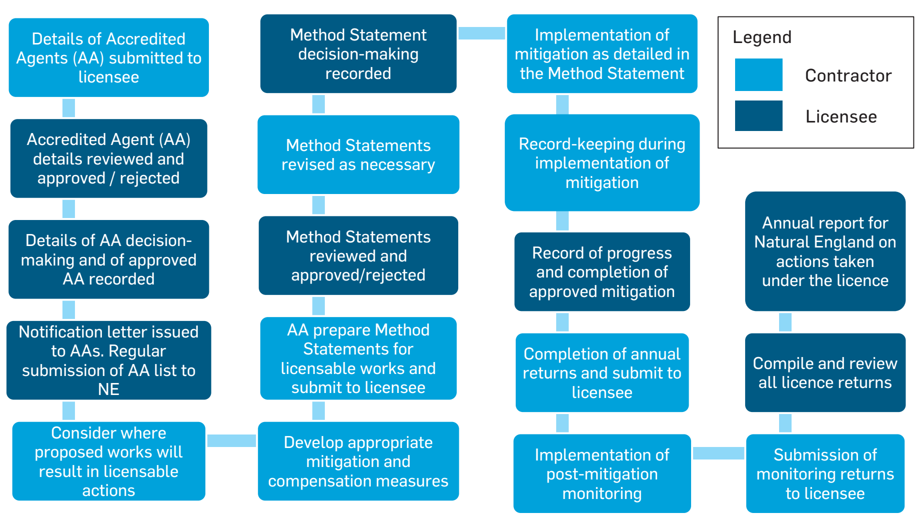
Responsibilities for delivery of the organisational licence was, at commencement, identified with three main steps:

- HS2 to approve Contractors Method Statements setting out the detailed design of impacts and mitigation for each population;
- Contractors undertake the works in accordance with an HS2-approved detailed design Method Statement; and
- HS2 to ensure compliance with the licence conditions.

These proved to be large individual steps, with the implementation of each requiring development and refinement, as significant resource demands were realised (see Figure 5). There were no existing similar processes in place at HS2 and all licence support and monitoring documentation and processes required developing and implementing.

FIGURE 5

Licensee and Contractor responsibilities for licence delivery



Each step was reviewed, and relevant processes, guidance and instructions put in place to ensure effective and efficient delivery. This involved developing:

- A compliance process and recording system.
- A bespoke Method Statement template.
- Guidance on the use of the new Defra Licensing Policies.
- An innovative GIS-based decision-making tool; and
- A process for effective engagement between all stakeholders.

5. Compliance Recording

Monitoring compliance at all stages of licence delivery is a core requirement for regulator audit purposes. This has been captured through the development of the following:

- **Compliance checklist:** The main compliance recording document for documenting the details of Authorised Persons; AA compliance with required minimum experience; that Method Statements provide all information required to comply with the licence and supporting documents; and evidence of approval (or rejection) of a submitted Method Statement. The GCN compliance checklist is reviewed and revised, where improvements are identified, to ensure it meets its intended function. For example, new fields provided to capture where compliance checks are needed.
- **On-boarding:** AA are invited to an on-boarding session organised and led by the Appointed Person and also attended by NE. The on-boarding involves a half-day session with presentations and Q&As with the aim of:

  - Providing clarity on roles and responsibilities of all those working under the licence.
  - Engendering a sense of shared responsibility for delivery of the mitigation under the licence.
  - Promoting a collaborative and integrated approach.
  - Stressing the need to consider a route-wide approach to mitigation and licence delivery.
  - Providing guidance on the application of the Defra Licensing Policies.
  - Providing clarity on compliance requirements and processes; and
  - Striving to ensure clear, consistent and comprehensive communication and engagement between all those involved.

- **Assurance Plan:** A plan detailing how HS2, as the licensee, will manage its assurance responsibilities necessary for legal compliance purposes and to provide confidence, and engender a high level of trust, with NE and other stakeholders. The Assurance Plan provides criteria to identify Method

Statements where impacts of works have been assessed as high, where Licensing Policies have been used, where there are increased stakeholder interests and where there are raised concerns of non-compliance. From this HS2 is able to identify where targeted, focussed or random compliance checks need to be undertaken. These can either be site-based, virtual or desk-based. HS2's assurance involves checking the following information:

» AA possession of their notification letter from HS2.
» The approved Method Statement is available on site and in the correct version.
» Evidence that the AA has supervised the works on site.
» The works undertaken accord with the approved Method Statement.
» Evidence of site inductions/toolbox talks.
» Evidence of suitability/maturation of receptor site terrestrial and/or aquatic habitat.

Where non-compliance issues are identified, they are raised with the AA and Contractor, and shared with NE as appropriate.

6. Method Statements

The organisational licence application did not contain the level of detail that would normally be expected at assessment stage for individual licence applications, due to the absence of scheme and impact detailed design at the time of application. Therefore, the licence conditions require a Method Statement (MS) to be produced per affected AMP, that clearly sets out impacts and mitigation and compensation proposals. Each MS is approved by an Ecology Manager in their role as Authorised Person.

A MS template was developed, with guidance on the level of information required, at the commencement of the licence to provide a consistent approach. The template was a 15-page Word document for AA to input all the relevant impact and mitigation information to submit to HS2 for approval.

Following feedback and lessons learnt from use of the template, improvements were made to try to provide greater clarity on the level of information required and to make the MS more concise. It was found that despite what were considered clear instructions in the template, the completed MS were consistently overly long, difficult to read/follow and, therefore, difficult to approve without additional iterations. Using feedback from MS authors it was identified that due to: 1) previous experience and expectations with standard licences; and 2) a risk-averse approach due to the very high profile of the project and the licence (with the regulator, general public and the industry as a whole), that they felt unable to reduce the content. Consequently, most MS involved several iterations before being suitable for approval, putting significant pressure on the programmed works, as well as on those reviewing and approving the MS.

Key template issues:

- Time taken to write and review the document. The MS could be 40-80 pages long and take up to six hours to review.
- Duplicate information in each completed MS.
- Each author has their own writing style so each MS could vary considerably in quality and content.
- No standardisation in the plans provided; and
- Many reviews required prior to approval (Figure 6), with 1% of submitted MS going through 11 iterations

FIGURE 6

Percentage of MS approved by version

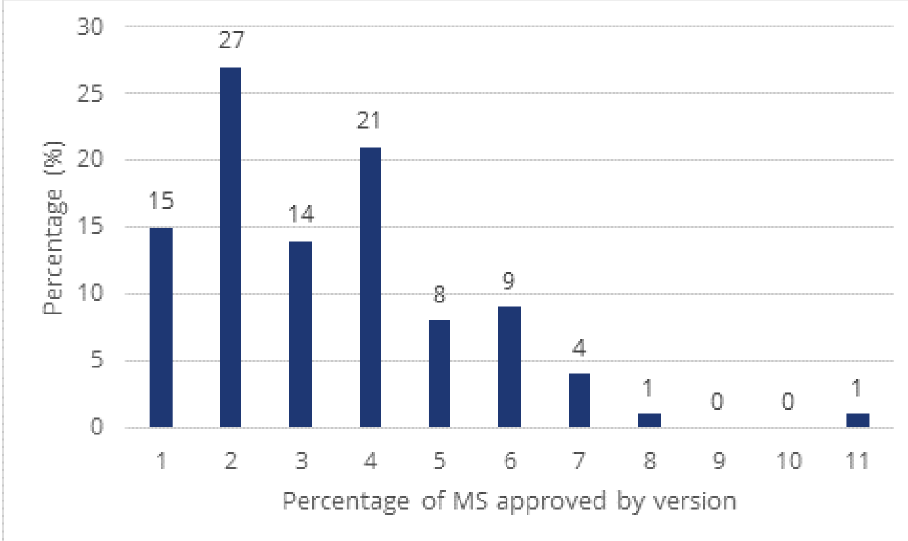


Figure 6 illustrates that with the Word MS template more than 50% of submitted MS took more than two versions to get approval and over 20% took five or more versions to get approval.

Due to the issues encountered with the MS template it was decided to take a novel approach and reproduce the template in a format, whereby: 1) most of the decision-making content was taken from the author (to avoid wordy explanations and unnecessary justifications); and 2) content was limited to what was required for compliance purposes (to avoid the inclusion of unnecessary additional information or data). Workshop sessions with the Appointed Person, Authorised Persons (who work directly with ecologists and AA) and senior HS2 environment team members (who work directly with Contractor project management teams), resulted in an improved MS template in Excel spreadsheet format.

The revised template approach asks licence compliance questions and provides drop-down menu responses wherever possible, and provides different sheets within the template for different compliance requirements, as follows:

- Site information
- Survey baseline
- Impact assessment
- Mitigation
- Habitat creation
- Site management
- Monitoring; and
- Work schedule and plans.

The Excel template is tailored to meet the specific need of the licensee, to provide only the information required to enable compliance with the licence. For example, a standard licence would require detailed information on the maintenance and monitoring of the mitigation site and GCN population, which is captured in the Excel template by providing all relevant options from the HMMPMP in drop-down menus covering up to 10 years post-impact monitoring. Early success of the Excel MS template is very positive:

- Review times are on average 2 hours, providing approximately a 33% reduction in reviewing time; and
- 100% of MS have been approved at either version one or two

7. Licensing Policies

Defra introduced four innovative new Licensing Policies (LPs) for European Protected Species in December 2016; shortly before HS2 applied for and was granted the organisational licence.

A summary of the LPs is provided in Figure 7 below.

FIGURE 7

Summary of the LPs

LP1	LP2	LP3	LP4
Reduction (or some cases removal) of exclusion and relocation effort where there are other alternatives which deliver **greater benefit to the local population**	**Off-site, or out of population**, compensatory habitat provision could have **greater benefits** for the local population	**Temporary habitat** that will be developed at a later date **without the need for full mitigation/ compensation**	The consideration of **less survey** effort

The LPs encourage thinking on a wider, landscape scale, and thereby channelling investment into bigger, better, more joined-up habitat for wildlife (The Lawton Report)[7]. The LPs lend themselves very well to a landscape-scale project such as HS2, and it is a requirement within the MS template to evidence that they have been considered. Where the application of the LPs is being considered, engagement with NE is required to ensure all relevant issues are considered and for NE's own records.

As the LPs are a relatively new approach to GCN mitigation, the AA on-boarding sessions include a dedicated briefing on LPs, which covers the rationale for the LPs, an overview of each LP in turn, and examples/ case studies.

Despite the support provided, use of the LPs was hesitant, with reliance on the traditional approaches that ecologists are familiar with. Therefore, to improve the uptake of LPs, NE and HS2 developed a guidance note on the implementation of LP1 and are developing a guidance note on LP2 (LP1, 2 and 4 are most relevant to the HS2 Project, and LP1 and 2 are the least understood by ecologists). At the time of writing, LPs have been applied within 24 MS (LP1 used 12 times and LP4 used 12 times), and uptake in the use of LPs increased between 2017/18 and 2019/20 (LP1 use increased from five to seven, and LP4 use increased from zero to 12).

8. Decision-Making Tool

Any proposals for the use of LPs must be proposed, agreed upon, and reported within the organisational licence conditions, however, use of LPs is in its infancy and so there are few case studies and no existing guidance to help guide ecologists. Therefore, a geospatial decision-making tool was designed to provide a level of guidance and to ensure consistency of approach across Phase One. The decision-making tool is designed to provide:

- Guidance on a Phase One-wide method for considering suitability of individual AMPs for LP use.
- Guidance on a Phase One-wide process for proposing and agreeing upon use of LPs by Contractors within the execution of the organisational licence; and
- An analysis of likely opportunities for LP use using currently available data.

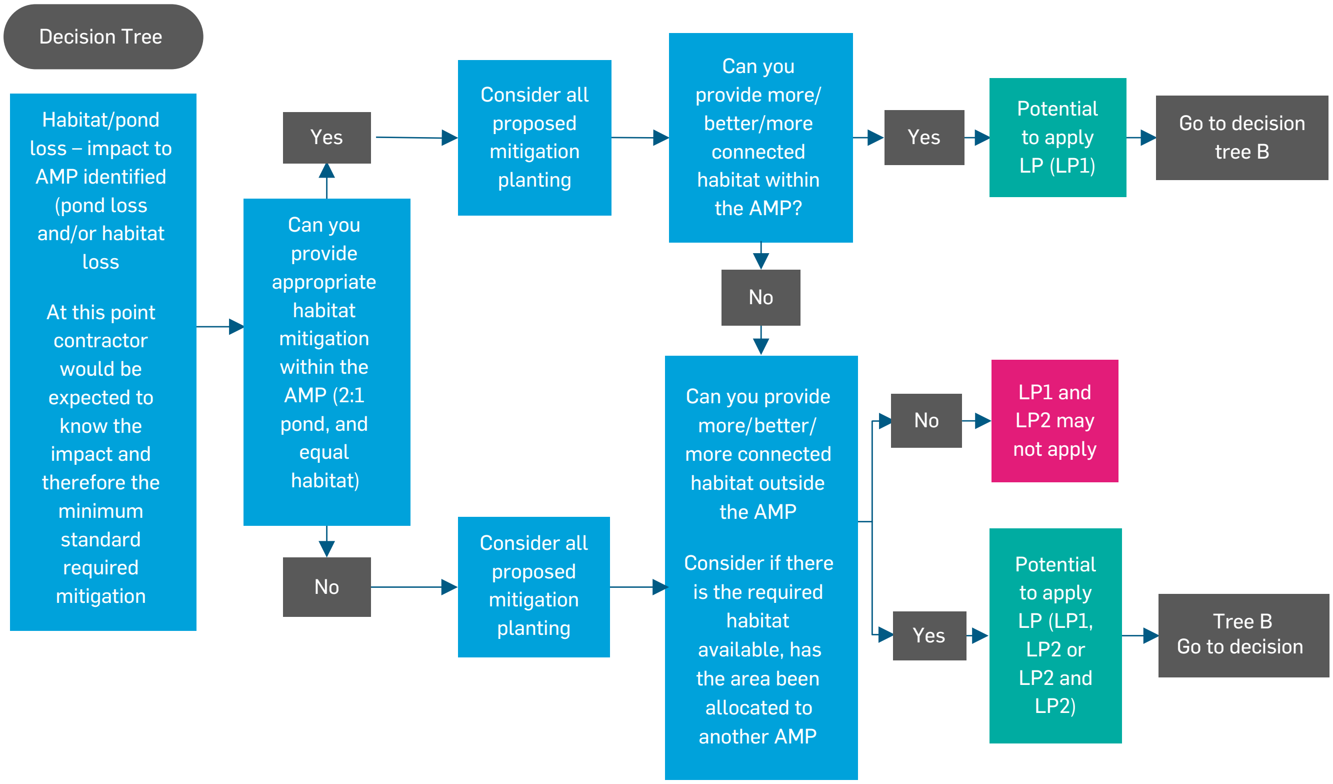
The GCN mitigation approach is based on the principle of ensuring sufficient suitable habitat available to each of the GCN meta-populations following construction, so not to affect their Favourable Conservation Status.

To provide an analysis of likely suitability of LP use using Phase One-wide data, the decision-making tool uses existing habitat data and available mitigation and landscape planting data to provide 'before' and 'after' ranges and extents of Suitable Terrestrial Habitat (STH) and ponds. This enables the calculation of change in availability within each AMP and within a migration zone outside each AMP. This data is also used for a calculation of connectivity between AMPs (before and after construction).

The analysis of likely opportunities is calculated using decision trees (Figure 8) and identifies the potential of each AMP for use of LPs 1 and 2 as a Red/Amber/Green (RAG) rating. An opportunities mapping analysis is based upon the limited design information at that time and showed that opportunities to use LPs are likely in 59 AMPs (amber rating) and highly likely in six (green rating) with 36 AMPs unlikely to have opportunities to use LPs (red rating).

FIGURE 8

Decision tree example



At the time of writing, there is no feedback on the usefulness and effectiveness of the decision-making tool or the need for consideration of revisions, but these will be addressed as required once there is sufficient evidence base from discussions with the practitioners. However, such a tool is a significant step-change in the application of the LPs, with potential for wider application to ecologists nationally.

9. Engagement

HS2 has on-boarded AA (approximately annually) to ensure that the responsibilities under the organisational licence were clear. The most recent on-boarding session was recorded for the benefit of those who could not attend in person and made available for all AA. Sessions were also arranged to explain the Excel MS template, the fields within it and how to populate it.

In order to achieve timely and effective mitigation on site (for which an approved MS is required), fortnightly Ecology Consents Meetings are available for the AA, with the Authorised Persons and NE, to discuss issues around data, assessment and mitigation prior to formal submission of the MS. Typical items for discussion are potential application of an LP, discussing a novel approach, or a particularly complex issue for which they might seek approval from NE or the Authorised Person.

The Authorised Persons have initiated weekly meetings where AA are encouraged to discuss site specific MS prior to formal submission to HS2 (via the document control system). This engagement provides an opportunity to explain any issues to the Authorised Person and to refine the MS content to increase the likelihood of approval at first submission.

Where input is required for approaches not covered in the organisational licence, the AA submits a Task Request Form directly to NE to seek advice or approval via email.

10. Recommendations for Future HS2 Phases and Projects

The HS2 organisational licence approach has demonstrable benefits, in terms of cost and time savings and the positive development of processes and documentation, to advance its implementation and in securing the Favourable Conservation Status of GCN. Therefore, a similar approach could be applicable for other infrastructure projects, including Phase Two of HS2, other rail projects, major linear utility works and highways schemes.

However, the following points would need further consideration:

- The in-house capabilities of each organisation will differ and may require further training to ensure that there are suitable Appointed Person and Authorised Persons to manage the delivery of the licence
- Providing a suitably high level of survey coverage prior to licence submission may not be possible for some linear schemes; and
- It may not be possible to accommodate all mitigation within the scheme boundary and, therefore, off-site mitigation may be more appropriate (including consideration of DLL)

Assuming the above are not prohibitive, the following measures are recommended for successful implementation of an organisational GCN licence:

- Provision at the outset of a clear process
- Ensuring required documentation is all in place and designed specifically around delivery of the licence mitigation
- On-boarding of all AA is undertaken as soon as possible and prior to any Method Statements being written, to ensure that the licence specifics and the method of delivery are fully understood.
- A simple, focused MS template

- Early engagement sessions between licensee, NE and AA to be undertaken to discuss the MS prior to (or during) it being written; and
- Where it is an expectation that LPs could provide a significant part of the mitigation approach, ensure that all available tools are used to aid the decision-making process and that AA fully understand how to implement them successfully

A future improvement for the Excel MS template would be to include a GIS schema to enable the site boundary and all proposed works, including mitigation, to be included in a GIS format, which would reduce the number of documents and plans to be reviewed.

11. Conclusion

The specific and, quite possibly, unique circumstances of delivering HS2 has necessitated the development of novel approaches and writing (or re-writing) the standards. Ecological mitigation and compensation are central to the success of HS2 and, through the development and implementation of the Phase One GCN organisational licence, this setting of new standards and a new paradigm for established processes is clearly demonstrable. The developed approach has reduced the time and cost implications involved in licensing development works for HS2 Phase One and put in place a process to ensure that delivering Favourable Conservation Status for GCN and compliance with the licence conditions can be implemented successfully.

Key to effective implementation of such an organisational licence are the development of processes for:

- Early engagement between stakeholders.
- Appropriate compliance and assurance measures and recording of those measures
- Understanding the requirement for bespoke deliverables and putting in place processes and documentation to manage them effectively
- An impact assessment and mitigation approach focusing on Favourable Conservation Status and ongoing commitments to habitat management; and
- Allowance for an iterative approach to ensure lessons learnt can be captured and applied to realise continuing improvements

The measures and processes that have been established are providing successful management of a complex organisational licence and its delivery, with obvious applications to the other phases of HS2. In addition, these measures and processes can have further application to other development projects, particularly those involving linear infrastructure. Transferability of the developed measures for HS2's organisational licence may not be wholesale for other projects and every scheme must consider its own specific circumstances when considering a GCN organisational licence and how it will be managed and delivered.

Acknowledgements

Rebecca Lee, Senior Advisor – HS2 Protected Species Licensing, Natural England.

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SAFE WORKING LOAD

04: Humber Bridge Hanger Replacements and Testing in the UK

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Abstract

The Humber Bridge is a Grade 1 Listed structure which remains one of the longest single-span suspension bridges in the world, connecting the East Riding of Yorkshire with North Lincolnshire. The suspension bridge carries the A15 dual carriageway over the River Estuary. The bridge has a main span of 1,410m and two side spans of 280m and 530m. The bridge deck is connected to the main suspension cables by spiral wire rope hangers with socketed end connections. The bridge opened to traffic in 1981. The two next longest-spanning UK suspension bridges, the Severn Bridge (opened in 1966) and the Forth Road Bridge (opened in 1964), had both already had their hangers replaced because of concerns over their deterioration. As a result, additional visual inspections were carried out on the Humber Bridge hangers, which suggested that the condition remained generally good. However, hanger internal environmental or fatigue deterioration to the internal wires could not be detected solely from visual inspection, so three hangers were selected for trial replacement and testing. The hangers were selected based on those with the greatest peak stress, greatest fatigue stress, and greatest apparent corrosion. This paper discusses the global analysis of the bridge to identify the predicted stresses in the hangers, which was complicated by the triangulation of the hangers and uncertainties in the actual original construction sequence and methodology. It then covers the selection of the three trial hangers to be replaced, the specification of the replacement hangers, and the testing specified to determine the residual life of the original hangers. The design, fabrication, and installation of the works involved in replacing the hangers is described and the paper concludes with a discussion of the results of the hanger testing and how this has informed the strategy for management of the remaining hangers.

KEYWORDS

Suspension bridge; Hangers; Fatigue

1. Introduction

1.1. THE BRIDGE DETAILS

The Humber Bridge is a Grade 1 Listed structure which remains one of the longest single-span suspension bridges in the world, connecting the East Riding of Yorkshire with North Lincolnshire. The suspension bridge carries the A15 dual carriageway over the Humber Estuary, spanning between Barton on the south bank and Hessle on the north bank. The bridge has a main span of 1,410m and, unusually, two differing side spans of 280m and 530m on Hessle and Barton sides, respectively. The towers are formed from two tapered rein-forced concrete hollow section legs with horizontal concrete cross beams to form a ladder arrangement (Figure 1).

The bridge superstructure comprises a stiffened steel box girder with an orthotropic deck roadway and internal diaphragms at 4.525m c/c. The box sections were erected in 18.1m long sections and are 22m wide and 4.5m deep, with 3m wide cantilever panels incorporated along each side of the boxes to carry the walkways and cycle tracks.

FIGURE 1

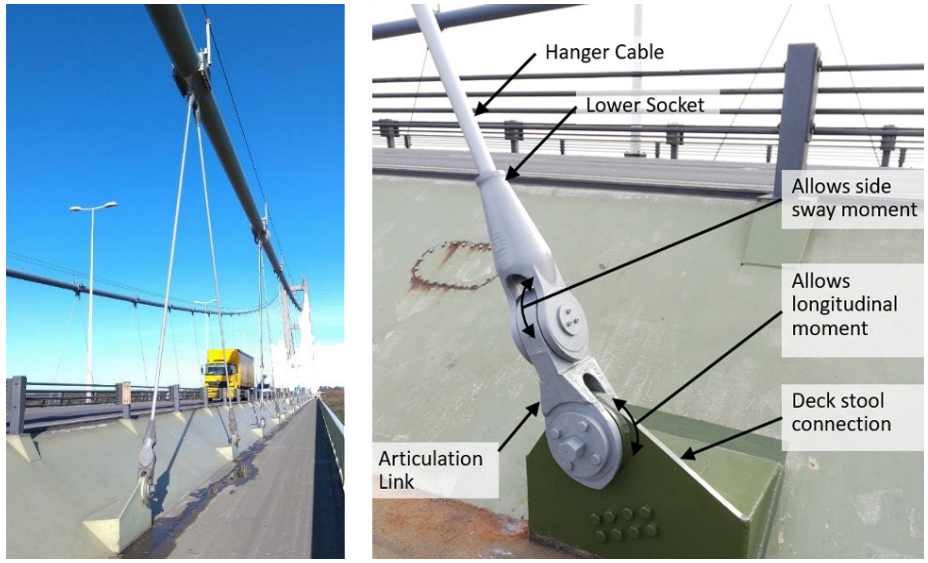
Humber Bridge

The bridge deck is connected to the main suspension cables by 484 spiral wire rope hangers (sometimes referred to as suspenders) with socketed end connections (Figure 2). The hangers are inclined rather than vertical. The hangers were manufactured by Bruntons of Musselburgh Limited from 62.3mm diameter spiral strand bridge rope. The hanger cables are attached to the deck via a welded stool with an articulated socket.

The shorter cables have double articulation to permit rotation of the cable both longitudinally and transversely, which may be caused by thermal and wind-induced movement of deck and main cables, together with vibration and misalignment of the hangers themselves.

FIGURE 2

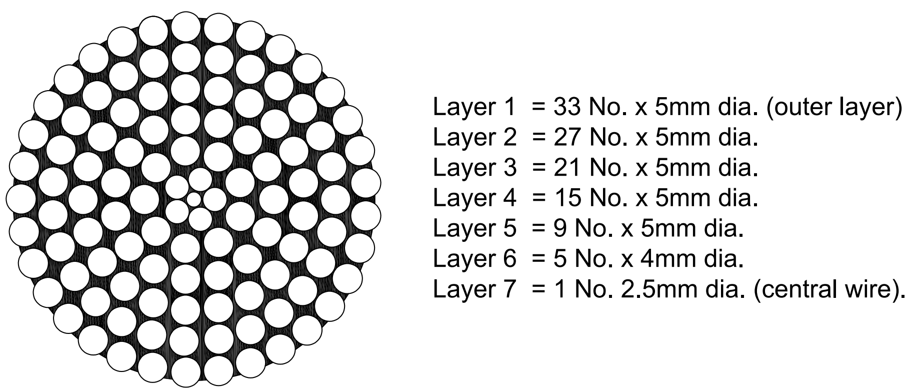
Hanger deck connection



Each rope is composed of 105No x 5mm Ø wire, 5No x 4mm Ø wire and 1No x 2.5mm Ø wires (Figure 3).

FIGURE 3

Spiral wire rope



The hanger ropes are internally lubricated with Bruntons “BM” compound and externally coated with Metal-coat, supplied by Bridon of Doncaster. The individual wires are galvanised. The minimum guaranteed breaking strength of each spiral rope hanger is 2,900kN and the Young’s Modulus of the hanger based on its total cross-sectional area is 140kN/mm^2, the low value compared to an individual wire being due to the spiral arrangement of the wires in the rope.

1.2. THE AIMS OF THE HANGER REPLACEMENT PROJECT

The two next longest-spanning UK suspension bridges, the Severn Bridge (opened in 1966) and the Forth Road Bridge (opened in 1964), had all of their hangers replaced after 23 to 24 years and 33 to 36 years of service life, respectively. This was driven by concerns over their deterioration and a wider industry belief that the life of hangers was typically limited to post-opening service by 2018 with all but two of the original hangers still present, hence it was decided that a programme of hanger investigation should be undertaken.

Additional detailed visual inspections (Figure 4) were carried out on the hangers which suggested that the condition remained generally good. A few wires were seen out of lay, which could indicate broken wires within a hanger, there was some water seepage, and there were a number of cracks in the Metalcoat paint system, but little evidence of significant corrosion on the outside of the hangers. However, corrosion, fatigue damage, wire embrittlement or wire breaks in the interior of the hangers could not be detected through visual inspection and hence more detailed investigation was required to create a management plan for the hangers and to inform decisions on when and how many hangers should be replaced.

FIGURE 4

Typical hanger defects

To gather more detailed information about the condition of the hangers and their residual life and hence produce a hanger management plan, it was decided that three hangers should be selected for trial replacement and testing.

2. Global Analysis and Selection of Trial Hangers to Replace and Test

2.1. GLOBAL ANALYSIS FOR DEAD LOADS

In order to enable the identification of the most highly stressed hangers and those with the greatest fatigue demand, and also to prepare specifications for testing existing hangers and procuring new replacement hangers, a 3D finite element model of the bridge was created in the LUSAS modelling software as shown in Figure 5.

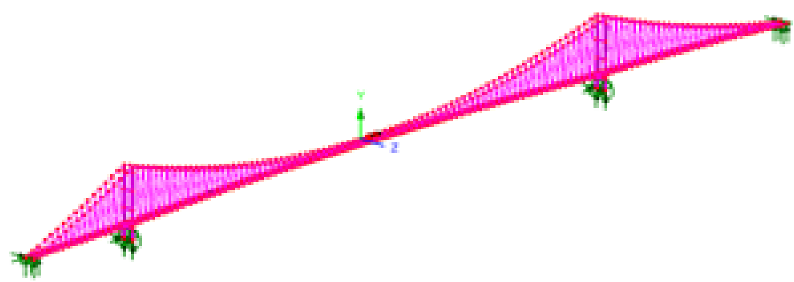


FIGURE 5

Global analysis idealisation of bridge

The inclination of the hangers sets up a truss action in the bridge under load, tending to put the deck into global tension and reducing the tensile force in the main suspension cables slightly. Adjacent hangers carry significantly different loads due to the inclination and truss action, one tending to increase its tension and one tending to reduce its tension. For the complete bridge, this distribution of forces between hangers under a given load case can be modelled with confidence. However, the distribution of forces generated in the hangers during construction is less certain and very sensitive to the assumed construction sequence. The bridge records show that the deck unit erection in the main span progressed from the centre of the span back towards the towers. In the side spans, deck erection started at the abutments and progressed to the towers. The sections of deck were first connected with rotational pins, allowing for movement during the construction, with those units later fully fixed once the catenary shape of the main cable had been achieved. In order to model the construction sequence, geometrically non-linear staged-construction analysis was undertaken to derive the forces in the hanger cables.

The construction sequence and the inclined cables had a very significant effect on the calculated cable forces, as can be seen in Table 1, which compares the original designers' estimate for the cable dead load force, the dead load force predicted by the current analysis, and the actual measured jacking force ultimately required to release the pins connecting the cable end sockets to the deck gusset plates during replacement. The locations of the replaced cables are shown in Figure 6.

FIGURE 6

Locations of selected hangers

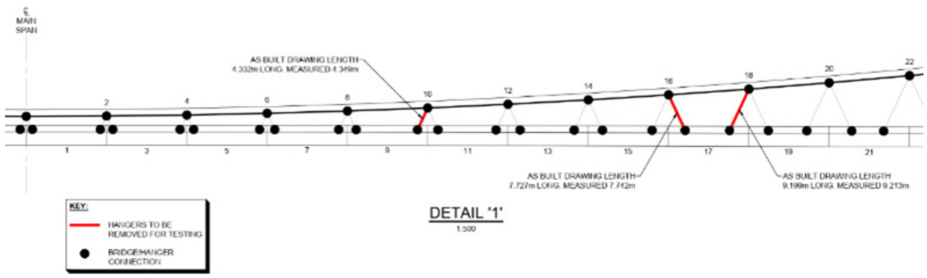


TABLE 1:

Predicted and actual dead loads in the three replaced hangers

Hanger reference	Freeman Fox & Partners Dead Load (kN)	Predicted Nominal Dead Loads (kN)	Actual Jacking Loads (kN)
10-9	410	793	950
16-17	410	137	290
18-17	410	774	1,100

Table 1 shows that the original design did not consider the effects of the diagonal hanger arrangement as the dead load forces are all the same. The predicted dead loads vary significantly due to the push-pull nature of the forces generated by the inclined hangers, whilst the actual measured jacking loads followed the same pat-tern, but the loads were all higher. The higher jacking loads measured onsite were expected because of the temporary works design which gripped the cable above the pin connection and therefore required a greater force than the actual cable force to reduce the reaction on the pin to zero (see section 5). For hangers 10-9 and 16-17, this accounted for the majority of the difference to the predicted loads and the rest was attributed to variations in surfacing thickness and a small effect of the traffic running on the bridge during jacking. How-ever, hanger 18-17 was found to be significantly higher loaded than predicted which was caused by the original fabrication length of the hanger being shorter than the adjacent hanger which was verified by detailed pin to pin site measurements.

2.2. LIVE LOAD ANALYSIS

To determine the hangers with greatest maximum service stress, both Eurocode design live loading to EN 1991-2 and Bridge Specific Assessment Live Loading, derived from weigh in motion equipment, were applied to the bridge. Hanger 10-9 was identified as having the greatest maximum service stress.

Hanger 16-17, pre-selected based on its condition, was noted to theoretically just be able to go slack under maximum live load and so certainly had the lowest minimum service load of all the hangers.

2.3. FATIGUE ANALYSIS

To determine the hangers with the greatest fatigue demand, simplified fatigue loading based on Eurocode BS EN 1991-2 (BSI 2003) fatigue load model FLM3, together with the damage equivalent factors in BS EN, 1993-2 (BSI 2006) were applied. This resulted in the determination of an applied fatigue stress range which, if applied over two million cycles, would cause the same damage as the real traffic spectrum. This approach is particularly useful in producing a fatigue testing specification (see section 4). The maximum service stress expected under normal operation was also determined because fatigue testing would be carried out by cycling the stress down from the maximum value by an amount equal to the calculated fatigue stress range. Hanger 18-17 was identified as having the greatest fatigue stress range which, based on the application of FLM3, was 78MPa. New cables/tension components are tested to $\Delta\sigma = 150$MPa in accordance with BS EN 1993-1-11 (BSI 2006) with an additional 1.25 additional safety factor, so this indicated that the actual fatigue demand on the bridge cables was much less than that required to be demonstrated for a new cable.

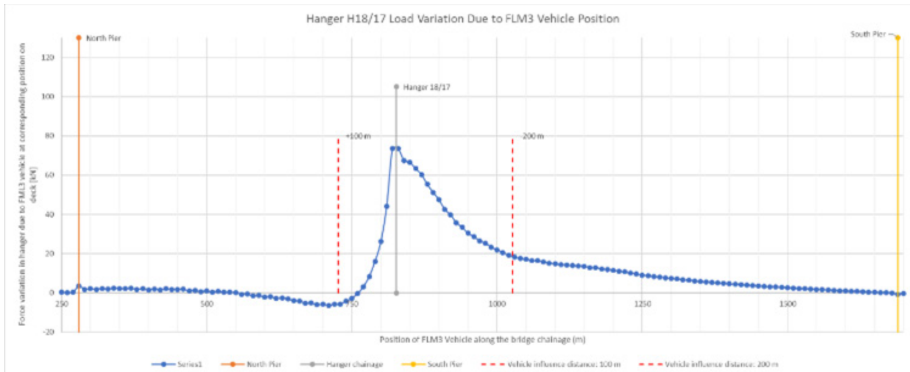
There is no obtainable detailed background to the λ_1 damage equivalent factor for fatigue load model FLM3 provided in BS EN 1993-2, and the fatigue assessment produced is generally considered to be conservative.

There is also some uncertainty around the application to a cable element. EN 1993-2 proposes the use of a critical length of the influence line equal to twice the hanger spacing for arch bridges, but this assumes that the majority of the load comes from the two adjacent bays between hangers. The influence line for the Humber hangers is very different to this because of the triangulation of the hanger system; Figure 7 shows the variation of a typical hanger force as the FLM3 vehicle traverses the bridge, where it is clear that the influence line is much more extensive than the hanger spacing.

Therefore, an additional assessment was carried out using fatigue load model FLM4, which uses a traffic spectrum defined in the UK National Annex to BS EN 1991-2 (BSI 2020) for comparison to the fatigue stress ranges derived from FLM3. This approach showed that the damage from FLM3 was a little greater than that from FLM4, hence the stress range from FLM3 was suitable for defining the fatigue testing requirements.

FIGURE 7

Hanger force variation due to FLM3 vehicle position



2.4. SELECTION OF TRIAL HANGERS TO REPLACE AND TEST

The choice of hangers to be replaced and tested were selected based on identifying cables with the greatest maximum service stress, the greatest fatigue loading and the worst physical condition. In addition, hangers were all chosen on the same side (west side) for construction purposes and hangers with attached structural health monitoring were avoided. The selected hangers and their reason for selection are summarised in Table 2.

Table 2: Selected hangers and outline testing proposed.

TABLE 2:

Selected hangers and outline testing proposed

Hanger Reference	Testing Selection Criteria
H10-9	High fatigue load range amongst central span hangers but with the greatest maximum hanger load under permanent + fatigue loading. › 4.3m long › No visible signs of corrosion but appears to have wires out of lay › Hanger tested for its residual fatigue life and ultimate tensile strength.
H16-17	Hanger with the lowest permanent load and subjected to theoretical decompression under live loading of the entire structure. › 7.7m long › In the worst physical condition, whereby it has corrosion and wires out of lay › Hanger proposed to be tested to destruction › Fatigue testing was not initially proposed for this hanger as it is in the worst physical condition and it may have had a limited fatigue life. Failure under fatigue testing alone would provide no information on the minimum tensile strength of all cables.
H18-17	Greatest fatigue load range. › 9.2m long › Hanger tested for its residual fatigue life and ultimate tensile strength (if sufficient fatigue life).

3. Digital Image Correlation (DIC) Measurement of Hanger Service Stress Range

As part of the project, Digital Image Correlation (DIC) measurements were undertaken on the cable with the highest predicted stress range to check whether the predictions made by the analysis model reflected the true behaviour. The outer layer of strands was exposed and a camera was used to track the movement of pixels and thereby measure strain, which could be converted to stress. This was done both during the day with unrestricted traffic and at night with an 11 T vehicle, for which the results were within +/-10% of those generated by the analysis model. The technique is well-established and has been used to determine live load stresses in a variety of bridges and also specifically to monitor hanger vibrations on the Great Belt Bridges (Winkler and Hendy (2018)).

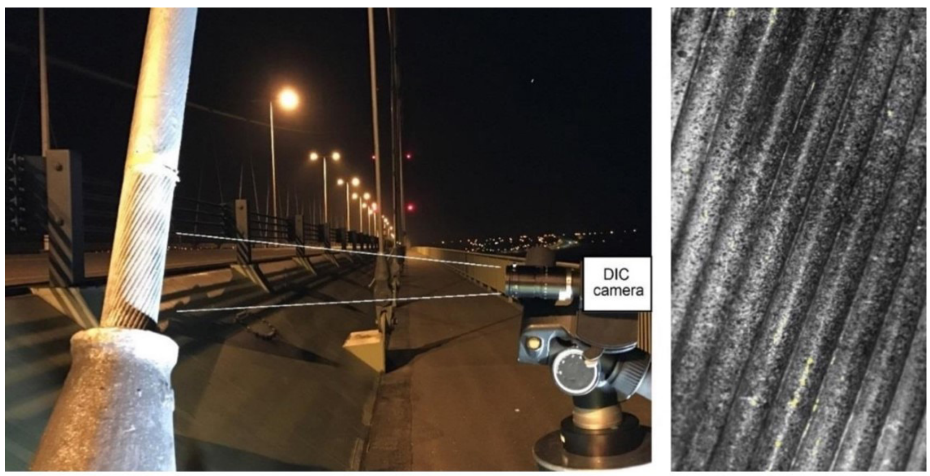


FIGURE 8

Hanger stress measurement using DIC (left) and close-up of hanger strand pattern as captured through DIC (right)

4. Hanger Testing Specification

4.1. OVERVIEW OF TESTING REQUIREMENTS

A wide range of testing, both non-destructive and destructive, was specified to be carried out on the three extracted hangers in order to provide as much information as possible on their condition and potential future life expectancy to facilitate producing the optimal management strategy. The testing schedule is shown in Table 3.

TABLE 3:

Hanger testing schedule

Hanger information		Test to be conducted						
Reference number	Length pin to pin (mm)	Visual Examination	Fatigue test	Ultimate tensile strength	Frac-ture surface inspec-tion	Examination of individual wires	Individual wire ultimate strength	Chemical analysis
9-10	4,332	y	y	y	y	y	n	y
16-17	7,727	y	n	y	y	y	n	y
17-18	9,199	y	y	y	y	y	n	y

The properties of a hanger and its constituent wires are listed in Table 4 below.

TABLE 4:

Hanger properties

Type of Steel	Diameter [mm]	Tensile strength [MPa]	Cross-section [mm²]	Minimum breaking force FMUTS [kN]	Nominal breaking force [kN]	95% of Minimum breaking force [kN]
Suspender Rope	62.3	1,416	2,271 (effective)	2,900	3,216 (328000kgf)	2,755
5mm wire	5	To be determined by testing	19.63	-	To be determined by testing	-
4mm wire	4		12.56	-		-
2.5mm wire	2.5		4.90	-		-

4.2. RESIDUAL FATIGUE LIFE TESTING AND TENSILE TESTING

Two original complete fully assembled hanger specimens were identified for fatigue testing as specified in Table 5. Each specimen was fully representative of the actual system and the conditions of use, including the actual anchorage systems of the hangers with the pin connection and corrosion protection. The tests were per-formed at ambient temperature between 10°C and 35°C, with a test frequency of 0.67 Hz, which was the fastest test speed practical.

Each specimen was subjected to two million cycles of the following coexisting effects:

- Sinusoidal variation of the axial stress between σ_{sup} and σ_{inf} as given in Table 5.
- Coexisting flexural loading induced by wedge-shaped shim plates introducing a 0.6° or 10mrad misalignment of the terminations with the axis of the hanger. Hanger reference 9-10 was tested under con-figuration 2 and hanger 17-18 was tested under configuration 1 as shown in Figure 9.

TABLE 5:

Fatigue axial test stress range per hanger

Hanger reference	Upper load limit [kN]	Upper stress limit σ_{sup}* [MPa]	Lower load limit [kN]	Lower stress limit σ_{inf}* [MPa]	Stress range (MPa)
9-10	1,118 (39% of minimum breaking load)	493	849	374	3,216 (328000kgf)
17-18	883 (30% of minimum breaking load)	388	705	310	119

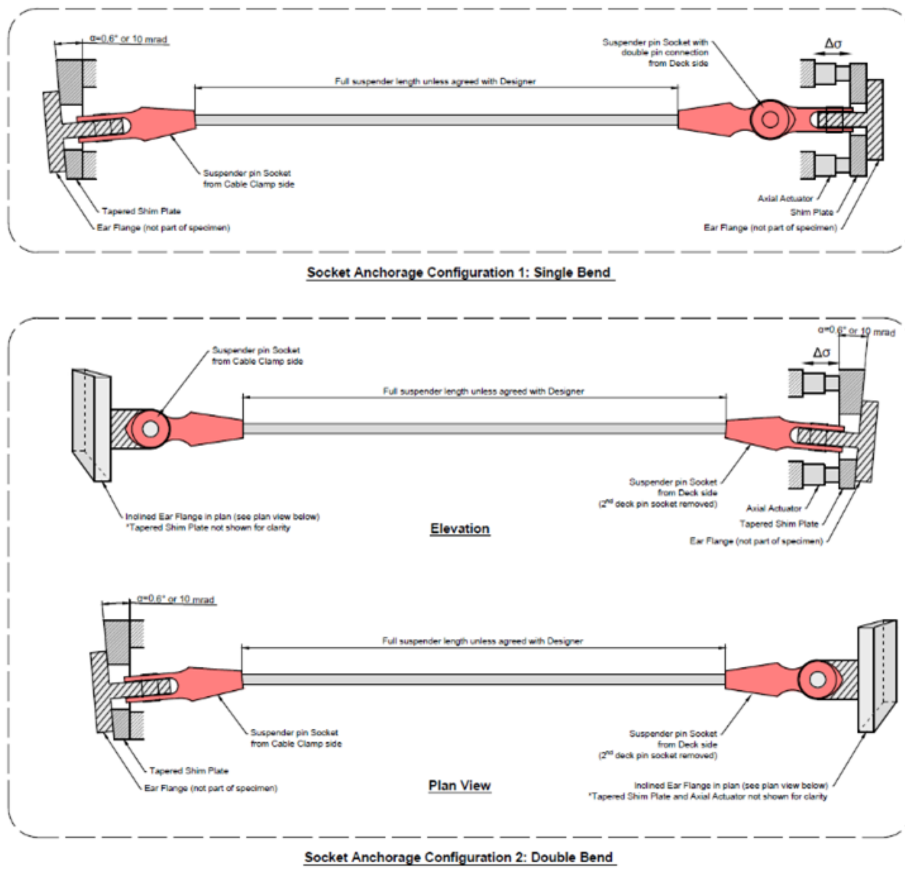
**Derived from calculated wire rope effective area of 2,271mm²*

The stress range of 78MPa specified for hanger 17-18 was that derived from the use of FLM3 as described in section 2.3. It was considered unlikely that the cables would be able to withstand anywhere near the same levels of stress as required for a new cable (typically 150MPa) because of the fatigue damage likely to have occurred in service to date and due to corrosion. Due to this concern, the 1.25 factor mentioned in section 2.3 was not applied to the 78MPa stress range. The specified stress range for hanger 9-10 was initially derived and specified the same way.

However, as discussed in section 7.2, the fatigue test for hanger 17-18 easily demonstrated adequate fatigue performance at the specified stress level, so the opportunity was taken to in-crease the stress range for the test of hanger 9-10 so that the test would give information at a different point on the S-N curve.

FIGURE 9

Fatigue test set-up



The fatigue test result was deemed to be satisfactory if:

- The specimen survived two million cycles of fatigue loading without detected breakage of more than 2% of the wires of which the hanger is made.
- No failure occurred in the anchorage material or in any component of the anchorage during the fatigue test.

An ultimate tensile-strength (UTS) test was also carried out after the fatigue test on each specimen. It was carried out in accordance with BS ISO 2408:2017 method 1 with the existing pin sockets used to support the hangers during the testing.

As per BS EN 1993-1-11 Appendix A4, the ultimate tensile test results of fatigue tested hanger was deemed to be satisfactory if it reached:

- a force of at least 95% of the minimum ultimate tensile strength FMUTS of the cable, defined from the characteristic strength of the strand and its nominal cross-sectional area from Table 4.
- a strain under the maximum load not less than 1.5%, allowing for deformation inherent to the operation of the anchorages (such as working-in of jaws).

For hanger 16-17, which was not first subjected to fatigue testing, the test was deemed satisfactory if the measured breaking force, Fm, reached or exceeded the minimum breaking force, FMUTS given in Table 4. UTS testing of the complete hanger assembly was followed by examination of the fracture surface to deter-mine type of failure (i.e., brittle or ductile).

5. Temporary Works for Replacing the Hangers

The hangers were removed and replaced without installation of temporary works to support the deck, under two lane traffic management. This required verification of the existing structure, deck, and adjacent hangers to accommodate the additional loads during the removal process.

Hanger releasing equipment was provided by the Contractor, Spencer, jacking up the deck to release the load from the hanger locking pin. The pin was removed and the deck was lowered gradually to remove the load from the hangers.

Load monitoring was conducted using proprietary vibrating wire strain gauges attached to adjacent hangers to ensure that the loads were not exceeded and to ensure that when the hangers were replaced this was at the correct load.

Due to the nature of the hanger replacement, the tolerance on hanger length was tighter than industry standards for new construction and therefore allowances for wedge draw-in were required during fabrication.

6. Results of the Testing for the Removed Hangers

6.1. OVERVIEW AND VISUAL INSPECTION

After removing from the bridge, the hangers, including their sockets, were transported to Norway where inspection and testing was undertaken by DNV.GL.

Hanger 16-17 was generally in good condition with a few wires out of lay and breaks in the paint coating at several locations.

Hanger 17-18 was in good condition. It was noted that the average thickness of the Metalcoat paint was thicker in the lower section of the cable at around 600 microns, probably because more maintenance painting had been conducted at the lower level of the hanger. The coating was cracked at one location but the wires below the Metalcoat appeared undamaged and free from any corrosion. The sockets appeared to be in good condition with no visible damage observed.

Hanger 9-10 was visually in a worse condition than hanger 17-18 with areas of exposed wires and corrosion, as shown in Figure 10. In addition, individual wires could be seen out of lay in several locations.

FIGURE 10

Hanger 9-10

6.2. FATIGUE TESTING

Hanger 17-18 was tested first in accordance with the stress range of 78MPa and set-up described in section 4.2. The rig is shown in Figure 11. A fabricated adaptor tool with a threaded bar was used to maintain the required misalignment of the upper and lower hanger sockets such that this force was not transmitted to the hydraulic system. The test rig was set to a nominal 58kN (2% of minimum breaking load) to allow final inspection and measurement prior to the test starting and acoustic monitoring was installed to detect wire breaks occurring during the test, which took 35 days to complete. One possible wire break was detected by the monitoring system after approximately 89,000 cycles, but no wire break was evident on visual inspection after the test until subsequent dismantling as described in section 6.3.

FIGURE 11

Fatigue test rig
(hanger 17-18)

Hanger 9-10 was tested second with the greater stress range of 119MPa, greater maximum stress in the cycle and set-up described in section 4.2. The test was completed to two million cycles with no wire breaks detected.

6.3. ULTIMATE TENSILE TESTING

All hangers were tested in a 2,900 T tensile test machine as shown in Figure 12 before the protective mat was placed over the hanger.

FIGURE 12

Tensile test rig
(hanger 16-17)

Hanger 16-17 was tested first and had not undergone prior fatigue testing. Failure occurred adjacent to the cable top anchorage (see Figure 13 and Table 6) at 3.4% average strain, which was greater than the minimum value specified of 2%. The measured breaking load also exceeded the 95% minimum breaking load as required.

FIGURE 13

Tensile test failure in hanger 16-17 (other hanger failures similar)

All wire failures were ductile and no other wire failures were visible away from the anchorage. Upon dismantling hanger 16-17, the socket was cut through by laser and the condition of the socket and wires inside were studied. There was no visible corrosion, and the condition of individual wire strands was very good. There were some minor air pockets within the zinc filler in the socket, but these had clearly not adversely affected the corrosion protection or the structural adequacy of the connection. The free length of the cable itself had isolated corrosion in wire layers 2 and 3 only, with two wire breaks noted away from the location of failure in the ultimate tensile test. These wire breaks were assumed to have occurred in service. It was also noted that the lubrication introduced when the cable was manufactured was absent in the three outer layers, but present in the inner four.

Hanger 17-18 was tested after fatigue testing. Failure again occurred adjacent to the cable top anchorage at 3.2% average strain which was greater than the minimum value specified of 2%. The measured breaking load also exceeded the 95% minimum breaking load as required. All wire failures were ductile and no other wire failures were visible away from the anchorage, but on dismantling, five wire breaks were found in layer 3. These wire breaks were subject to fracture surface investigation as discussed in section 6.5. Only one potential wire break was detected in the fatigue test, so it was concluded that the other breaks had either occurred in service or had occurred during the tensile test.

Hanger 9-10 was also tested after fatigue testing. Failure once more occurred adjacent to the cable top anchorage at 2.9% average strain (greater than the minimum value specified of 2%) and the measured breaking load exceeded 95% of the minimum breaking load as required. All wire failures were ductile and no other wire failures were visible away from the anchorage.

A summary of all the tensile test results is shown in Table 6. All failures occurred near the top anchorage and it was suggested that this was due to bending stresses during service near the upper anchorage for bending in the plane of the pin combined with a lack of lubrication at the top of the cable since it had flowed down to the bottom of the cable. There is also a trend indicating that greater fatigue cycling may reduce ductility as measured by the average strain at failure, but this is not pronounced.

TABLE 6:

Hanger tensile test results

Hanger	Nominal Breaking Force (kN)	Minimum Breaking Load (MBL) (kN)	Actual Breaking Load (kN)	Failure location	Failure strain
16-17	3,216	2,900	3,016	Close to upper socket	3.4%
17-18	3,216	2,900	3,070	Close to upper socket	3.2%
9-10	3,216	2,900	3,025	Close to upper socket	2.9%

6.4. EXAMINATION OF WIRE SURFACES UPON COMPLETION OF TESTING

Following completion of the ultimate tensile testing of the complete hanger system in section 6.3, the ropes were dismantled into their constituent wires, keeping a record of from which layer the wires came. The wires were categorized visually by corrosion stage (as defined in the NCHRP report 534 (2004)) for which the four corrosion stages are characterized by the presence of the following:

1. New / spots of zinc oxidation on the wires
2. Zinc oxidation on the entire wire surface
3. Spots of brown rust covering up to 30% of the surface of a 75mm to 150mm length of wire
4. 4. Brown rust covering more than 30% of the surface of a 75mm to 150mm length of wire

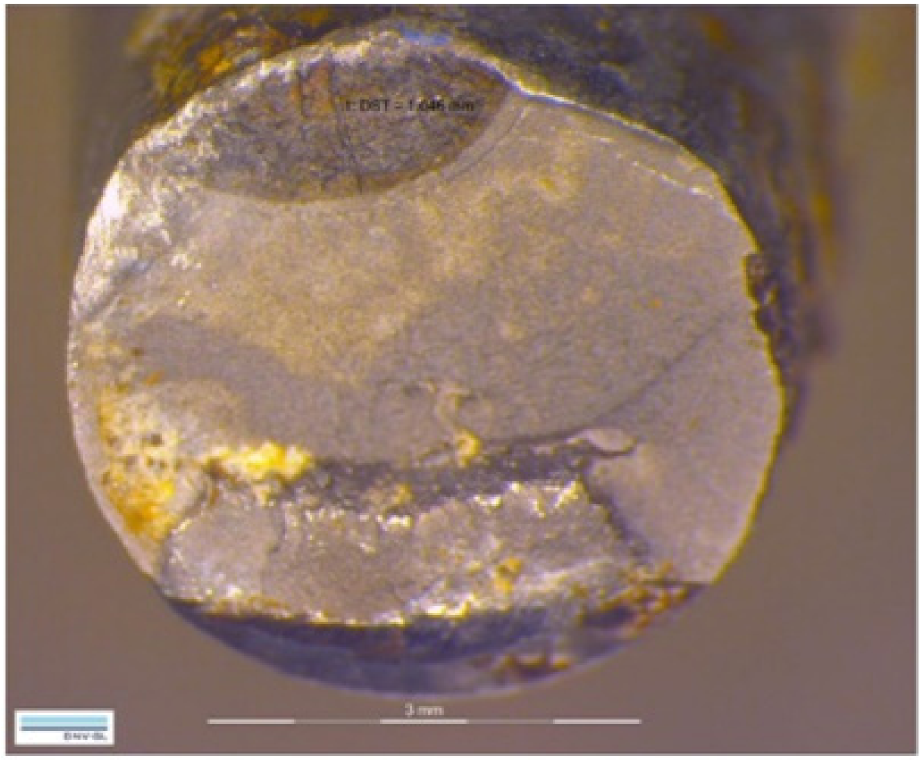
Previous experience on the main cable assessments of Humber, Severn (Hendy C., Mundell C., and Bishop D. (2014)) and Forth bridges has shown that the likelihood of brittle fractures of wire increases significantly once stage 3 and stage 4 corrosion conditions are present as crack propagation is initiated at corrosion pits which behave as crack-like defects.

6.5. FRACTURE SURFACE INVESTIGATION

After the tensile test, five single wire breaks were found in the third layer of the hanger cable 17-18. These single wire failures are remote from the main breakage, which occurred close to the upper socket. These individual wire failure surfaces were subjected to a fracture surface investigation which suggested that, in three out of five wires, the breaks had occurred due to brittle fracture following fatigue crack propagation from a corrosion pit that generates a crack like defect. A typical wire surface is shown in Figure 14. As mentioned in section 6.4, this phenomenon is usually linked with wires in corrosion stage 3 and 4.

FIGURE 14

Typical fracture surface of broken wire from hanger 17-18, away from tensile test failure zone



7. Conclusion

The Humber hanger replacement and testing scheme has developed the detailed and repeatable methodology for hanger replacement through an increased understanding of the bridge behaviour and has justified extending the life of the existing hangers through adoption of a detailed management strategy. The minimal structural intervention adopted is both environmentally and economically advantageous. The following conclusions can be drawn:

a. The original construction sequence has resulted in significant variations of permanent load in all hangers throughout the bridge. This results in hangers that are significantly more highly loaded, and which are at greater risk of failure from fatigue loading combined with internal corrosion.
b. All existing hanger cables were found to be in reasonable to relatively good condition, given the 38 years of in-service life, both internally and externally. However, corrosion stage 4 wires have been found internally, showing ongoing deterioration.
c. Fatigue testing has been carried out on two hangers based on the derived stress ranges. Both hangers completed two million cycles with only a small number of wire breaks found after the fatigue testing, suggesting that the cables are able to withstand future fatigue loading in the actual bridge for many years provided that corrosion does not continue.
d. This study has highlighted the lack of clarity and applicability provided by the current version of the Eurocode BS EN 1991-2 and the UK National Annex in applying fatigue load model FLM4 and has provided recommendations for future changes required for the National Annex.
e. Ultimate tensile testing of the hangers tested has exceeded the minimum breaking load originally specified and exceeded the minimum ductility requirements.
f. Existing hanger testing has demonstrated that there is additional service life of the existing hangers provided corrosion and further deterioration is appropriately managed.

g. Given the now known condition of the hangers' internal and external condition, it is recommended to extend the service life of the existing hangers through a regime of targeted mainte-nance, painting, possible targeted replacement and enhanced inspection.
h. A regime of targeted maintenance has a significant cost benefit when compared to full hanger replacement. Assuming a net present value of 4%, the comparable whole-life costs over a period of 20 years for carrying out the replacement now is £20.5 million, compared to delaying the re-placement by 10 years and implementing maintenance at a whole-life cost £14.5 million.

This paper provides guidance to other suspension bridge owners whose hangers are approaching the end of their original design life, who wish to pursue a similar reduction in structural intervention.

Acknowledgements

Adapted from the original publication: A Arundel, D Bishop, CR Hendy, C Mundell. Humber Bridge hanger replacements and testing. Proceedings of the Institution of Civil Engineers - Bridge Engineering 2021 174:4, 241-253

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05: Targeted Asset Management Approach to Mitigating Railway Earthwork Instability - Chitts Hill Embankment, Colchester, Essex

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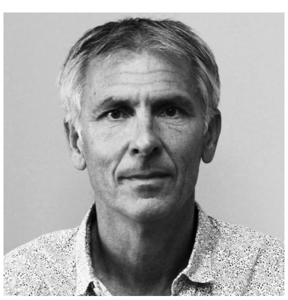

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Abstract

Like many other ageing Victorian earthworks, the railway embankment at Chitts Hill, west of Colchester in Essex, has instability problems that are affecting the track and ultimately pose a risk to the public. The site is approximately 520m long between two overbridges on the 100mph busy London to Norwich railway line. The embankment is located within the River Colne valley and underlying deposits include local soft alluvium overlying the London Clay Formation. Detailed interrogation of the site has taken place as part of the Control Period (CP) 5 and early CP6 asset management. The designer has undertaken desk study, geomorphological mapping and a risk management and optioneering exercise to determine the mitigation extents and solutions. The site has a history of repairs including toe weighting, cess retention and a sheet piled wall but problems persist. In addition to repeated twist faults, alignment issues and frequent maintenance of the track and trackbed, slope movement indicators are evident including a displaced cess walkway, toe bulges, poor drainage, vermin burrowing, tree related desiccation and run-on/run-off transition issues with structures. Understanding the geological complexities, slope movements through inclinometer data, establishing a sound ground model and understanding the robustness of existing interventions is key to selecting the appropriate mitigation. The oversteep clay slopes with ash crests typically show movements of 20-30mm per year at 2-3m depth with occasional more sudden movements triggering Temporary Speed Restrictions (TSRs). The Anglia Targeted Asset

Management (TAM) process includes value engineering principles to fix the areas posing the highest risk while continuing to monitor adjacent areas. The approach allows funding to stretch to other high priority sites across the region meeting the Network Rail earthworks asset policy objectives. Optioneering of the proposed engineering solutions, including cess retention and soil nails will be reviewed together with overcoming the constraints including access and managing ecological and environmental legislation.

KEYWORDS
Slopes – stabilisation; Geotechnical engineering; Railway tracks; Earthworks; Asset management

1. Introduction

Past remediation works have often focussed on a particular section of earthwork exhibiting immediate concerns, rather than assessing failure in terms of the overall condition of the asset. However, by not assessing the entire asset, including both sides of the track and the surrounding natural setting, key failure indicators and an appreciation of scale may be missed. Furthermore, the presence of other failures and/or previously implemented remediation measures along an asset suggest that stability issues may not be limited to the area of current failure and that an ongoing or more widely spread problem could be present. Alternately, the assessment of the entire asset can indicate that an issue is localised only, giving confidence in the chosen remediation measure. It is therefore essential that the asset as a whole is assessed to determine the extent of issues at a site and the nature of instability to ensure the correct mitigation type and extent is implemented to allow the continued serviceability of the earthwork and railway.

It is recognised however, that due to the large scale of many of the Earthworks Renewal projects and the constrained budgets available for remedial works, large-scale heavy engineering solutions across entire sites are not feasible. Therefore, a risk managed and targeted approach to renewal works is needed to ensure the correct remediation is implemented at the worst affected sections, whilst ensuring that a suitable monitoring system is in place to ensure other areas of concern do not deteriorate. This allows for efficiencies in remediation designs and helps to eliminate multiple period spot fixing that will cost proportionately more in the long term (see (Payne, 2018) for another case history).

2. Targeted Asset Management (TAM)

The Targeted Asset Management (TAM) has been developed as a proactive way to meet the Earthworks Asset Policy objectives and optimise budgets. By implementing a risk management approach to undertaking works at the site, the approach moves away from implementing a full fix across an entire site to more sustainable solutions, allowing a larger area of earthwork to be remediated for the available budget by targeting works at the worst affected areas. The TAM risk management approach is being utilised to identify the scope of remedial works at key sites within the Anglia Earthworks CP6 workbank. The TAM process targets stabilisation at the highest risk sections of the earthworks to minimise operational disruption and is undertaken in four stages as outlined below.

TAM Stage 1 (GRIP 3-4) – Site mapping, data review, risk zonation, preliminary geotechnical analysis, optioneering and the Approval in Principle AIP (F001) output.

TAM Stage 2 – Cost estimation, scope ratification and confirmation of work scope.

TAM Stage 3 (GRIP 5-6) – Detailed design of agreed remedial works, construction support, further targeted GI and a future works strategy plan to ensure ongoing asset stewardship.

TAM Stage 4 – Continued monitoring and collation of associated data (Network Rail).

Constant dialog between the Client, Contractor and Designer is key to the TAM process and ensures that the final remediation option selected both tackles the instability issues at a site whilst remaining both affordable and constructible. TAM Stage 1 focusses solely on data collection and considers the asset as a whole, enabling the site to be divided into zones based on current condition. Outline analysis can then be completed for these sections allowing an optioneering process to be undertaken, both in terms of the type of remedial measures and extent of remedial works.

The options determined during TAM Stage 1 are then scrutinised in terms of cost and constructability during TAM Stage 2 resulting in a finalised agreed scope of works.

Detailed design and construction are undertaken during TAM Stage 3 as well as design installation of future monitoring and emergency works designs, enabling an immediate response in the event of an earthworks failure in one of the site zones not remediated as part of the main works.

2.1. TAM STAGE 1

TAM Stage 1 aims to identify and prioritise the key geotechnical risks of the earthwork with potential to cause problems to the safe operation of the railway. A technical risk register is used to record the process and the output enables optimised renewal works at the higher risk areas. Measures such as continued slope movement monitoring are implemented at unremediated areas to control residual risks.

The TAM technical risk register has been agreed by the Anglia Route Collaboration (ARC) that uses 3 main failure categories:

1. Track and Maintenance issues
2. Ground Conditions and Slope Movement Monitoring
3. Visual Movement Indicators

Each of the failure criteria is further subdivided as defined in Table 1 and a risk of low, medium or high defined based on the available data for a particular zone of the site. Where no data is available this is entered into the register, allowing areas of concern where limited data currently exists also to be identified. The zone is then given an overall TAM Risk Category based on the output of the initial failure criteria assessment. These categories are used to determine the extent of remedial works recommended at the site and are largely based on engineering judgement of the initial failure criteria assessment output. The overall risk categories are defined in Table 2.

TABLE 1:

Definition of TAM failure criteria

<table>
<tr><th colspan="3">Category & Definition</th><th colspan="2">Risk Rating</th><th>Risk Definition</th></tr>
<tr><td rowspan="8">1 (Track and maintenance data)</td><td rowspan="4">A</td><td rowspan="4">Service Impacts (TSRs, Track Closures, delay minutes, service disruptions)</td><td>H</td><td>HIGH</td><td>2 TSR/track closure/delay minutes recorded within past 12 months.</td></tr>
<tr><td>M</td><td>MEDIUM</td><td>TSR/track closure/delay minutes recorded within past 5 years</td></tr>
<tr><td>L</td><td>LOW</td><td>No historic TSR/track closure/delay minutes recorded in section.</td></tr>
<tr><td>?</td><td>NO DATA</td><td>No data available for section.</td></tr>
<tr><td rowspan="4">B</td><td rowspan="4">Track Maintenance and Defects (NR track maintenance records and NR track geometry records/LADS data)</td><td>H</td><td>HIGH</td><td>Bespoke track maintenance implemented within past 12 months (hand jack & pack, temporary cess support etc.); more than 3 no. standard maintenance interventions (e.g. tamping/stoneblowing). Track dip/twist recorded within past 12 months.</td></tr>
<tr><td>M</td><td>MEDIUM</td><td>Increased level of track maintenance in section within past 5 years. Bespoke track maintenance implemented within past 5 years; Maintenance intervention run off from adjacent section. Track dip/twist recorded within past 5 years; track dip/twist run off from adjacent section.</td></tr>
<tr><td>L</td><td>LOW</td><td>No historic record of bespoke track maintenance in section. Standard maintenance works only. No historic track dip/twist recorded in section.</td></tr>
<tr><td>?</td><td>NO DATA</td><td>No data available for section.</td></tr>
</table>

Category & Definition			Risk Rating		Risk Definition
2 (existing GI data)	A	In place inclinometers/ instrumentation observations	H	HIGH	Significant rate and/or total movement; deep seated movement in inclinometer; shallow movement in upper slope.
			M	MEDIUM	Minor rate and/or total movement; shallow movement in midslope or toe.
			L	LOW	No/negligible movement.
			?	NO DATA	No instrumentation available for section.
	B	Ground conditions	H	HIGH	Ground conditions deemed indicative of unstable earthwork conditions.
			M	MEDIUM	Ground conditions deemed suggestive of possible unstable earthwork conditions.
			L	LOW	Ground conditions do not indicate any signs of earthwork instability.
			?	NO DATA	No existing GI data available.
3 (visual observations and/or recent evidence)	A	Track area & cess	H	HIGH	Visual and/or recent evidence of significant movement to track, ballast shoulder, cess and/or lineside infrastructure.
			M	MEDIUM	Visual and/or recent evidence of minor movement to track, ballast shoulder, cess and/or lineside infrastructure.
			L	LOW	No visual and/or recent evidence of movement at track level.
			?	NO DATA	Not examined (provide reason).
	B	Slope face	H	HIGH	Visual and/or recent evidence/indicator of significant slope movement on lower, mid or upper slope of embankment/cutting.
			M	MEDIUM	Visual and/or recent evidence/indicator of minor slope movement on lower, mid or upper slope of embankment/cutting.
			L	LOW	No visual and/or recent evidence of movement on slope face.
			?	NO DATA	Not examined (provide reason).

Category & Definition			Risk Rating		Risk Definition
3 (visual observations and/or recent evidence)	C	Drainage	H	HIGH	Visual and/or recent evidence of major drainage issues.
			M	MEDIUM	Visual and/or recent evidence of minor drainage issues.
			L	LOW	No visual and/or recent evidence of drainage issues.
			?	NO DATA	Not examined (provide reason).
	D	3rd Party mass movement feature; ponding; undercutting of embankment toe by 3rd party; loading of slope crest by 3rd part etc	H	HIGH	Visual and/or recent evidence of major issues outside of railway boundary fence.
			M	MEDIUM	Visual and/or recent evidence of minor issues outside of railway boundary fence.
			L	LOW	No visual and/or recent evidence of issues outside of railway boundary fence.
			?	NO DATA	Not examined (provide reason).
	E	Vermin	H	HIGH	Visual and/or recent evidence of major vermin issues.
			M	MEDIUM	Visual and/or recent evidence of minor vermin issues.
			L	LOW	No visual and/or recent evidence of vermin issues.
			?	NO DATA	Not examined (provide reason).

TABLE 2:

Definition of TAM risk category

Risk Category	Definition
High (H)	Known evidence of regular high priority track issues and or above average slope movements with visual signs of instability with high potential to cause public disruption and a threat to the safe operation of the railway.
Medium (M)	Potential to cause continued track issues and affect maintenance regime. Slope movements are considered at or below average and steady. No imminent signs of failure from visual observations. Continue to monitor and have a future works strategy plan in place should it enter HIGH risk.
Low (L)	Minor track and maintenance issues with no cause for concern, below average slope movements (where data) and no visual indicators of failure.

3. TAM Site Case Study – Chitts Hill Embankment

3.1. INTRODUCTION AND SITE OCATION

The Chitts Hill railway embankment is located approximately 2km west of Colchester, Essex as shown on Figure 1. The site runs southwest to northeast on the Liverpool Street to Trowse Lower Junction (Engineers Line Reference LTN1) between 49 miles 69.5 chains (49m 69.5ch) and 50m 16.5ch and is bounded to the London end by a seven arch masonry viaduct over the River Colne and at the country end by an underbridge. The embankment is 10m to 15m high and lies on the edge of a flat flood plain at approximately 10mAOD. The site is surrounded by agricultural fields to the north west and a golf course to the south east.

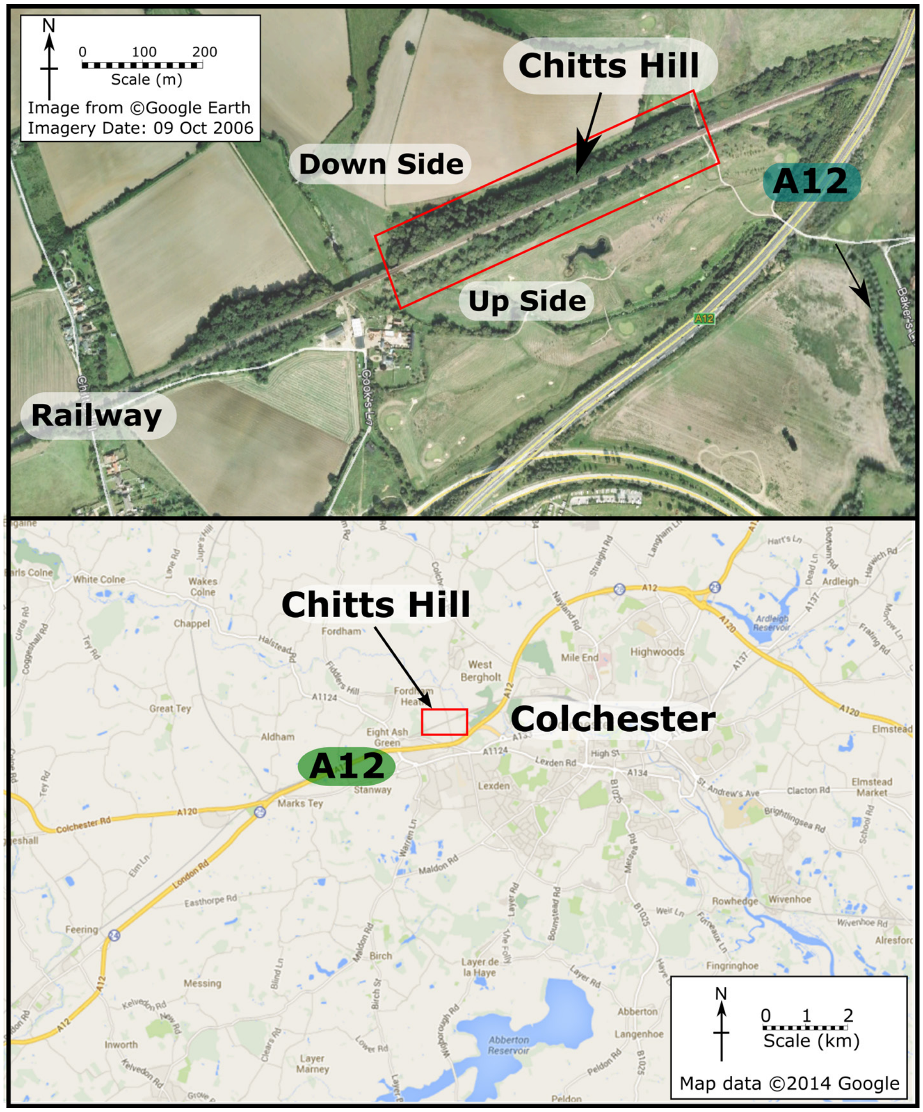
The Up side of the embankment has a history of instability with defects recorded on the earliest available modern inspections in 2005 and historical map evidence indicates past interventions and ongoing instability issues. More recently a section of the embankment between 49m 73ch and 49m 75ch was repaired during 2011. This initially comprised a 4m deep king post and Armco retaining wall at the crest of the embankment to retain the Cess. Following continued deeper seated movement this was supplemented by a more substantial sheet pile retaining wall near the toe of the embankment with regrading of the slope above the toe wall.

3.2. PUBLISHED GEOLOGY AND HYDROGEOLOGY

The British Geological Survey (BGS) Map Sheet 223 of the 1:50,000 series (British Geological Survey, 1982) indicates the geological succession at the site as superficial River Terrace Deposits overlying the bedrock geology of the London Clay Formation. Alluvium is mapped at the south western edge of the site associated with the River Colne and may extend beneath the London end of the embankment. Head deposits and Glacial Sand and Gravel deposits are also mapped on the higher ground to the north of the site and may extend beneath the Country end of the embankment.

FIGURE 1

Site location plan



3.3. HISTORICAL DEVELOPMENT

The site is located on the Great Eastern Mainline between London and Norwich which was opened in stages between 1839 and 1851, with the section through the site completed by 1844. The earliest available historic map is from 1877, on which the railway is already present. The embankment is shown to have an irregular profile toe from the end of the viaduct at 49m 69.5ch through to approximately 50m 00ch from which point the slopes and toe become more regular through to the end of the site at 50m 16.5ch (see Figure 2 for reference to site mileages). The neighbouring land is shown as fields on both sides of the railway and along the Down side the embankment toe is indicated to be marshy or wet by the presence of hydrophyllic vegetation symbology, suggesting that drainage issues were already prevalent at this early stage. A tree filled hollow at the toe of the Down side slope is also shown at approximately 49m 78ch between to toe bulges, which may represent the water concentration feature identified on site.

The 1897 and 1923 maps show no significant changes; however, the 1953-1964 map sheet shows considerable changes to the embankment slopes. Two linear benches are now shown on the Up and Down embankment slopes between 50m01ch to 07ch and 50m10ch to 15.5ch respectively, indicating implemented remedial works due to slope instability issues along the site. The toe of the embankment is also shown to extend to the boundary fence at the London end of the site on both the Up and Down sides, which may indicate further failure and lateral spreading of the slopes. The most recent detailed historic map of the site is from the 1960-1978 period and shows no considerable changes to the site.

3.4. GEOMORPHOLOGICAL SURVEY

A detailed geomorphological survey of both the Up and Down side of the embankment was undertaken during winter 2018/19, with key slope features and defects recorded on a geomorphological map of the site, shown in Figure 2. Numerous indicators of stability issues were noted along the length of the embankment on both the Up and Down side including toe bulging and failure lobe features indicating deep seated failure of the embankment slope (Figure 4E). Indicators of groundwater issues were identified in the form of hydrophilic vegetation and ponding observed in areas where drainage pathways have been blocked by the presence of toe bulges (Figure 4B, D & E). Several historic repairs were recorded in the form of toe berms/weights on both the Up and Down side (Figure 4C & F) and a more recent sheet pile wall and slope regrade (Figure 4A) indicating an ongoing history of slope instability issues at the site.

Available LiDAR data was used both before and during the detailed geomorphological survey to help identify the location and scale of features at the site. The toe bulging and lateral spreading indicated by the historic map data at the London end of the site is visible on the LiDAR imagery as shown in Figure 3. Historic and recent remediation measures are also discernible and aided the identification of these features on site during the detailed geomorphological surveys.

The embankment slopes at the site are heavily vegetated, with large mature deciduous trees present on both the Up and Down sides. These large trees are most densely concentrated at the London end of the site where the historic failures have led to less steeply graded midslopes.

Drainage issues are also key to embankment stability and often are the driving factor behind failure. Historic slipping on both the Up and Down sides of the embankment, visible on the LiDAR, aerial imagery and historic maps, was confirmed during the detailed geomorphological mapping of the site. This has led to a number of large toe bulges which have cut off drainage pathways and on the Down side and resulted in the formation of a significant pond approximately 10m wide and 2-3m deep when full. This pond aligns directly with the current worst failure location on the Up side.

The Country end of the site is founded on sidelong ground where the embankment runs parallel with the River Colne and at this point natural drainage from the high ground to the north into the valley is blocked by the railway embankment. There are no culverts recorded between the viaduct at the London end of the site and the underbridge at the country end of the site, meaning that all water draining from the higher ground to the north of the site must either drain along the length of the embankment to the London end, where drainage pathways are blocked by historic failures, or make its way beneath the embankment as groundwater flow.

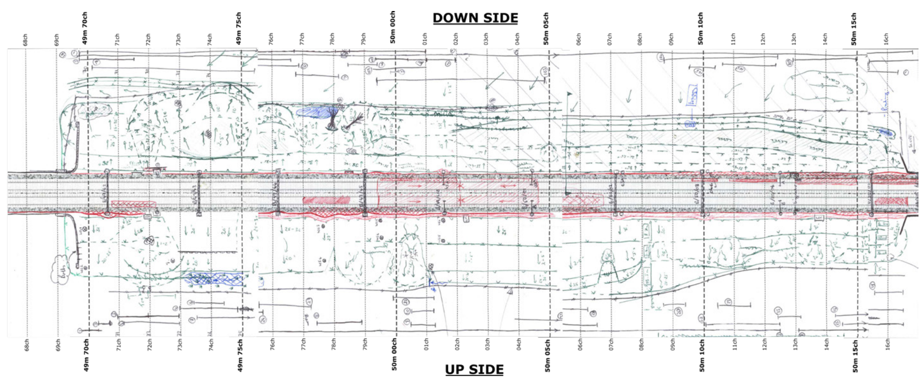


FIGURE 2

Detailed geomorphological mapping undertaken on 4th December 2018 and 22nd January 2019. The detailed map was hand drawn on site based on field observations. The red annotations indicate track issues, blue indicate drainage issues and green indicate slope issues. Each of the numbered items records a detailed observation at the site which are not reproduced here.

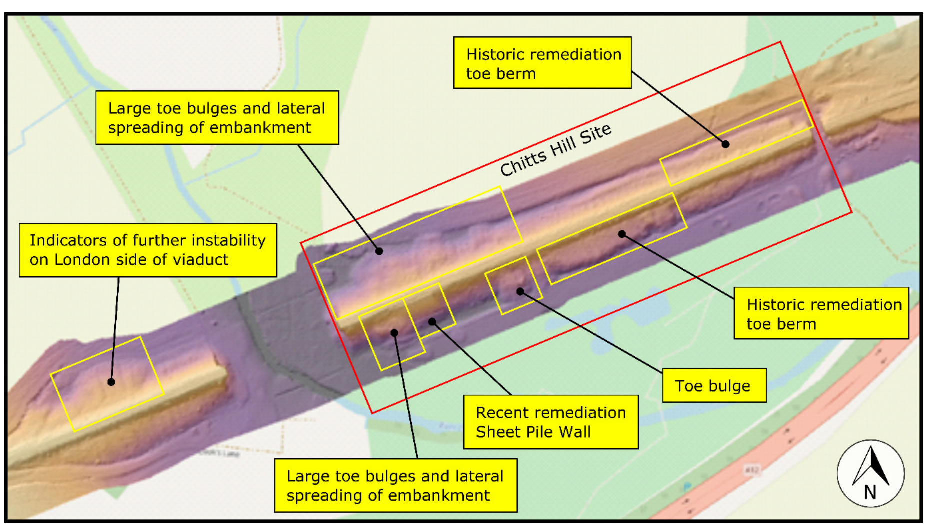


FIGURE 3

LiDAR imagery of the site

FIGURE 4

Site walkover observations of the earthwork slope

A) 49m 73 to 75ch Up Good condition slope above existing sheet pile wall earthwork renewal

B) 49m 74ch Up Hydrophilic vegetation and ponding at toe beneath sheet pile wall earthwork renewal

C) 50m 01 to 05ch Up Toe berm – ash and clinker construction – possible historic stabilisation

D) 49m 77 to 78ch Down Ponding at slope toe – opposite worst defect on Up Side

E) 49m 76ch Down Historic toe bulge blocking toe ditch causing ponding at slope toe

F) 50m 05 to 12ch Down Toe berm on slope – possible historic stabilisation?

3.5. TRACKSIDE ISSUES

During the walkover survey the track and cess area was observed to be in very poor condition at multiple locations along the embankment.

The worst observed area was at 49m 78ch on the Up side where a large dip in the track and cess walkway was recorded along with ballast shoulder thickening up to 1.5m and a lack of ballast shoulder above sleeper level (Figure 5B).

At both ends of the site, where the embankment interfaces with adjacent structures, the track and cess conditions were observed to be in generally poor condition with ballast spalling down the embankment slopes leading to a low shoulder and exposed sleeper ends (Figure 5A & D). A small king post wall has been constructed at the country end of the site to restrain the ballast shoulder, but this was observed to be in poor condition and out of alignment (Figure 5D). Track issues such as these are commonly observed across the network at the embankment/structure interfaces.

Access to the full length of the Down side embankment at track level was not possible due to no walking route being present, nevertheless, it was possible to view the track area periodically along the length of the asset. Some issues were observed on the Down side such as small track dips, loss of the ballast shoulder and exposed sleeper ends, however, the condition of the Down side appeared significantly better than the Up side.

FIGURE 5

Site walkover observations at track level

A) 49m 71ch Up Exposed sleeper ends, ballast spalling down slope and failing cess

B) 49m 78ch Up Large dip in cess walkway and associated track dip

C) 50m 05ch Up Cess area in better condition compared to the rest of the site

D) 50m 15ch Up Sleeper ends exposed, poor cess condition. A small king post retaining wall has been constructed to retain ballast at this location.

3.6. TRACK MONITORING

NR periodically monitor track condition through train mounted equipment which records track geometry and associated ground penetrating radar (GPR) from which the condition of the trackbed can be inferred. The frequency of monitoring across the network is dependent on the frequency of use of the line, with the results of the monitoring presented as Linear Asset Decision Support (LADS) data (Bentley, 2019). LADS data has been consulted for the site and indicates a history of twist faults around 50m 00ch on the Up side between 2016 and 2018, correlating to the large track and cess dip observed during the walkover survey (see Figure 5B). The poor earthwork condition at this location is causing the twist faults observed at track level. Further twist faults are also recorded approximately 50m 16ch adjacent to the underbridge at the country end of the site, which correlate to the defects observed during the walkover survey (see Figure 5D) indicative of the embankment/structure interface issues previously discussed.

3.7. RECENT EARTHWORK REMEDIATION

An Armco type king post retaining wall, designed by URS, was constructed by CML at 49m 73ch to 75ch on the Up side of the embankment in May 2011 (URS, 2011) (see Figure 6B). This structure was designed to remediate a large dip in the track (Figure 6A) which had led to the implementation of a temporary speed restriction (TSR) and was causing maintenance issues at the site. The two king post walls were constructed with 4m long king piles and with two separate rows of piles to allow the wall to be constructed around an OLE stanchion on site. During construction it was identified that the slope failure was also deeper seated than originally thought and therefore the small cess retaining wall was not sufficient to arrest the deeper slope movement. Subsequently a larger lower slope sheet pile retaining wall was designed by URS and installed by CML in summer 2011 (URS, 2011) (see Figure 6C). The wall comprised a 10m long L605 sheet pile with a 2m retained height. The slope behind the retaining wall was regraded with general granular fill to form a uniform slope. Following installation of the sheet pile retaining wall maintenance issues have abated at this section of the site. The sheet pile retaining wall was also observed during walkover survey in December 2018 and appeared to be effective (Figure 6D), however a small amount of vermin burrowing was observed in the granular fill above the sheet pile wall.

FIGURE 6

Recent earthwork remediation

A) 50m 73 to 75ch Up Distorted cess walkway and temporary speed restriction (TSR) speed board, in early 2011 prior to remediation

B) 50m 73 to 75ch Up Two rows of cess retention constructed during May 2011, the slope continued to deteriorate shortly after construction due to deep seated failure (URS, 2011)

C) 50m 73 to 75ch Up Sheet pile wall constructed in summer 2011 to replace the failing cess retention and address deep seated failure mechanism (URS, 2011)

D) 50m 73 to 75ch Up Recent photo along sheet pile wall showing good horrizontal allignment.

3.8. ECOLOGY SURVEYS

Ecology constraints are ever present at all sites across the network. Chitts Hill was initially subject to a Phase 1 habitat survey by Ecus in September 2018. This identified the potential for several protected species and recommended further surveys for great crested newts (GCN), bats, dormice and badgers, as well as a precautionary method of working in respect to common reptile species and nesting birds.

A robust approach to ecology is required to ensure that unexpected issues do not arise that delay a construction project. The majority of potential ecological issues have been managed through detailed surveys, with results for dormice and bats the only two outstanding surveys.

3.9. GROUND MODEL – GROUND INVESTIGATION

Several phases of recent GI have been undertaken at the site; an initial phase of GI was specified by Atkins (Atkins, 2014) and undertaken on the Up side embankment slopes in 2015 by Topdrill (Topdrill, 2015) to inform design work undertaken in CP5. The GI work comprised 8 windowless sample exploratory holes to a maximum depth of 12mbgl undertaken at varying heights up the embankment slope. The exploratory holes were undertaken using Topdrill Limited (Topdrill) Lightweight Limited Access Modular Rig (LLAMR) (see Figure 7A).

This rig enabled good samples of the embankment fill to be obtained; however, samples of the underlying natural ground were limited to 1-3m due to the presence of sand and gravel deposits which caused early refusal in some of the positions. Upon completion inclinometers were installed in all 8 exploratory holes.

Further supplementary GI works were specified by Atkins and undertaken at the site by Topdrill during 2019 comprising 9 windowless sample exploratory holes and 3 dynamic/rotary core boreholes. The exploratory holes were undertaken to a maximum depth of 12mbgl and inclinometers were installed in all exploratory holes upon completion. The supplementary GI allowed the underlying natural ground profile to be confirmed with the use of a rotary drilling rig (See Figure 7B) at the toe to penetrate the underlying sand and gravel layers.

FIGURE 7

Ground investigation utilising a LLAMAR drilling rig and a rotary drilling rig

A) LLAMAR Rig set up at embankment crest at Country end of site for window sampling.

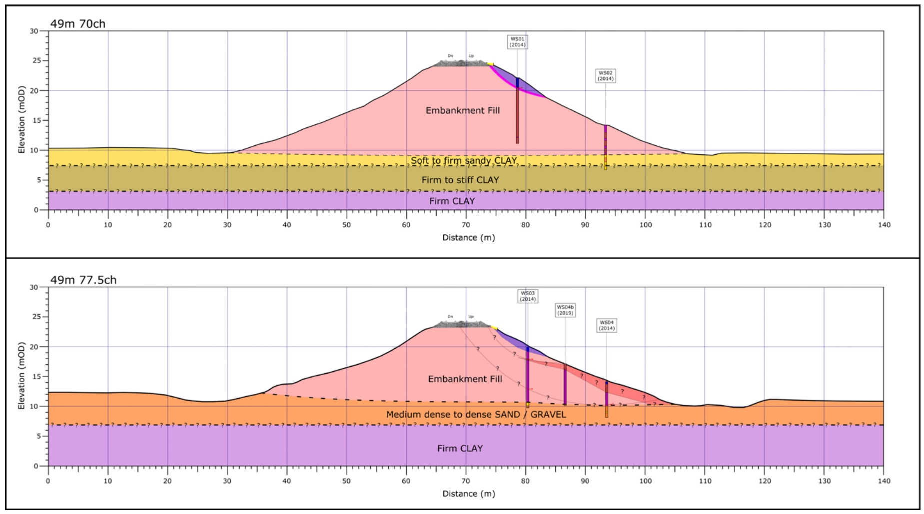
B) Rotary rig set up at embankment toe at London end of site for continuous sampling

3.10. GROUND CONDITIONS

The GI works undertaken to date have enabled the development of a detailed ground model for the site, which will be subject to further refinement during the detailed design process following the most recently installed slope monitoring data becoming available. Two sections produced to allow stability analysis to be undertaken at 49m 70ch and 49m 77.5ch are shown in Figure 8.

FIGURE 8

Ground models at 49m 70ch and 49m 77.5ch



Typically, the embankment slopes at the site feature an over steepened upper slope comprised of loose ash and clinker deposits, overlying the embankment core (see Figure 9). Due to the nature of construction used on the early railway construction across the UK, and subsequent historic stability problems at the site, the composition of embankment fill does show variations across the site, but typically was found to comprise slightly gravelly sandy clays. At some sections along the site horizons of softer clay were logged within the embankment fill which from the initial analysis appear to align with areas of movement identified in the inclinometers installed upon completion.

FIGURE 9

Embankment fill material encountered during 2015 GI works

A) Ash and clinker overlying cohesive embankment fill in upper slope

B) Absence of ash and clinker in lower slope.

The underlying natural ground conditions, as anticipated based on interpretation of slope failure mechanisms and slope morphology, were found to vary along the site. At the London end of the site up to 3m of soft alluvial soils associated with the River Colne were encountered, whereas the middle and country end of the site was underlain by more competent sand gravel and stiff sandy clay representative of river terrace and head deposits.

These changes are directly linked to the embankment traversing a number of different geological settings, from the base of the river valley at the London end of the site onto the lower flanks of the valley side slopes at the country end.

The top of the London Clay formation was also confirmed along the length of the site, ranging from approximately 3mOD at the London end of the site to approximately 8mOD at the country end of the site, tying in with the nearby historic borehole data along the A12 corridor.

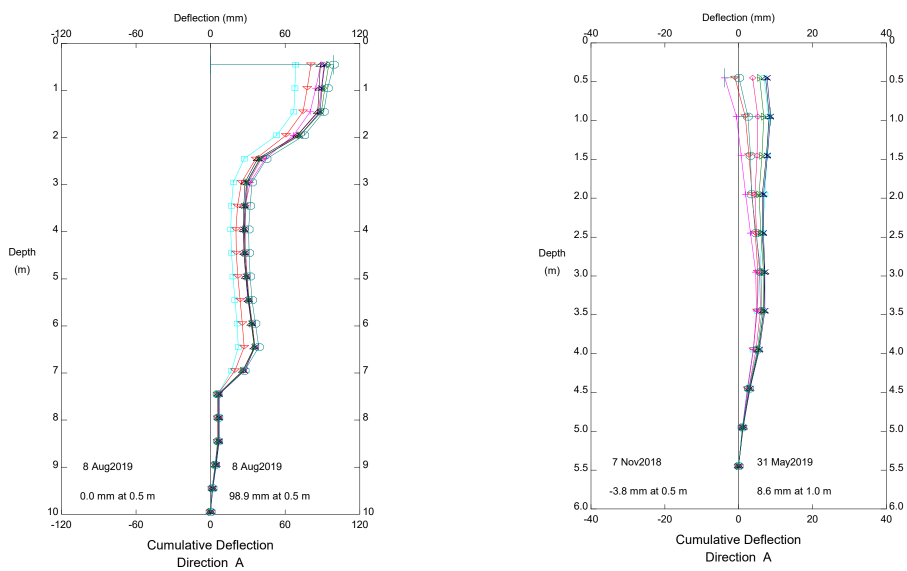
3.11. SLOPE MONITORING

Eight inclinometers were installed at the site during the 2015 GI and these installations have been monitored since installation. The installations are targeted at the worst affected areas of the site (see Figure 2) and movement trends observed in the inclinometers varied across the site.

At the worst affected area of the site at 49m 77.5ch total movement of up to 91mm was observed over the 4-year monitoring period (Figure 10). This movement was observed to be deep seated with a failure plane identified at approximately 7mbgl at the slope crest. This deep-seated movement was also observed in a number of the other inclinometer installations at the site.

FIGURE 10

Inclinometer monitoring results indicating deep seated movements at 49m 77.5ch



A) 49m 77.5ch WS03 (slope crest) 99mm max movement between 2014 and 2019. Two potential zones of movement are visible – one at 2.5mbgl and one at 7mbgl

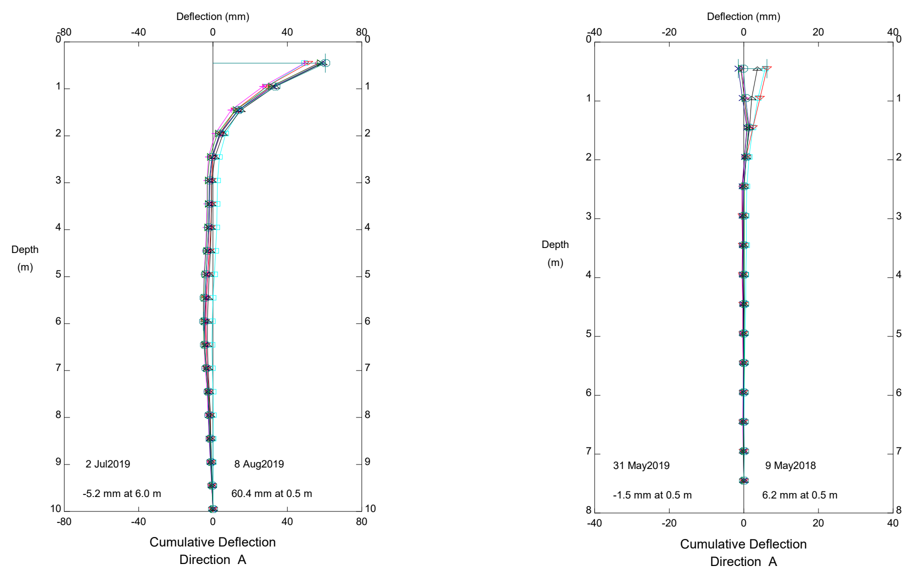
B) 49m 77.5ch WS04 (mid-slope) 8mm max movement between 2014 and 2019. Inclinometer is founded at shallow depth (5.5mbgl) therefore deeper-seated movement could be occurring below the base of the inclinometer

A different failure mechanism was observed adjacent to the viaduct at 49m 70ch. At this location a high magnitude of movement was observed (57mm max) but this progressive movement was observed at shallow depth at up to 2mbgl at the crest of the embankment (Figure 11A). It is thought that this shallow failure is caused by a loose ash layer which is confined to the upper shoulder of the embankment slope with some desiccation related movement evident in Figure 11B.

Twelve additional inclinometers were installed at the site at the start of 2019 to allow more detailed examination of the slope movement at the site, the monitoring results from these installations are not yet available.

FIGURE 11

Inclinometer monitoring results indicating shallow depth failure adjacent to viaduct



A) 49m 70ch WS01 (slope crest) 60mm max movement between 2014 and 2019

B) 49m 70ch WS02 (mid-slope) 6mm max movement between 2014 and 2019

4. Chitts Hill TAM Stage 1 Output

The Chitts Hill site has been subjected to the TAM Stage 1 process and following the detailed site mapping the site was subdivided into zones. The zones were then assessed in line with the TAM failure criteria and assigned an overall TAM Risk Category as can be seen in Figure 12.

The Up side has been subdivided into eight zones, the division of these zones being determined both by physical changes in the shape of the earthwork (indicative of different failure mechanisms) and the condition at track level, which has the most immediate impact on the operational safety of the railway. Each of these zones has been scrutinised in detail and all available data analysed to determine the resultant TAM Risk Category. The quantity of available GI and slope monitoring data on the Up side has enabled the zoning to be targeted, enabling resultant remediation works to target the worst affected areas.

In contrast, the Down side has less individual zones as the information available for this side of the earthwork is limited, as this did not form part of the original site scope. Therefore, the subdivision is based entirely on site observations and available track data. However, as a result of undertaking this assessment, various issues have been identified on the Down slopes which were not previously known, including various historic failures, historic intervention works and drainage issues that may be directly affecting the Up side condition at the site. Due to the investigation undertaken there are now plans to undertake drainage improvement works on the Down side to address the identified issues.

The TAM zoning process has allowed the proposed remedial works to be tailored to the specific requirements of each zone, which ultimately will allow a greater percentage of the site to be fixed within the available budget. The data also allows a detailed analysis of the mode of failure in these areas, giving NR the option to consider smaller localised fix in some areas (such as run on/run off of structures) to maintain the serviceability of the railway without implementing a full-scale embankment remediation throughout.

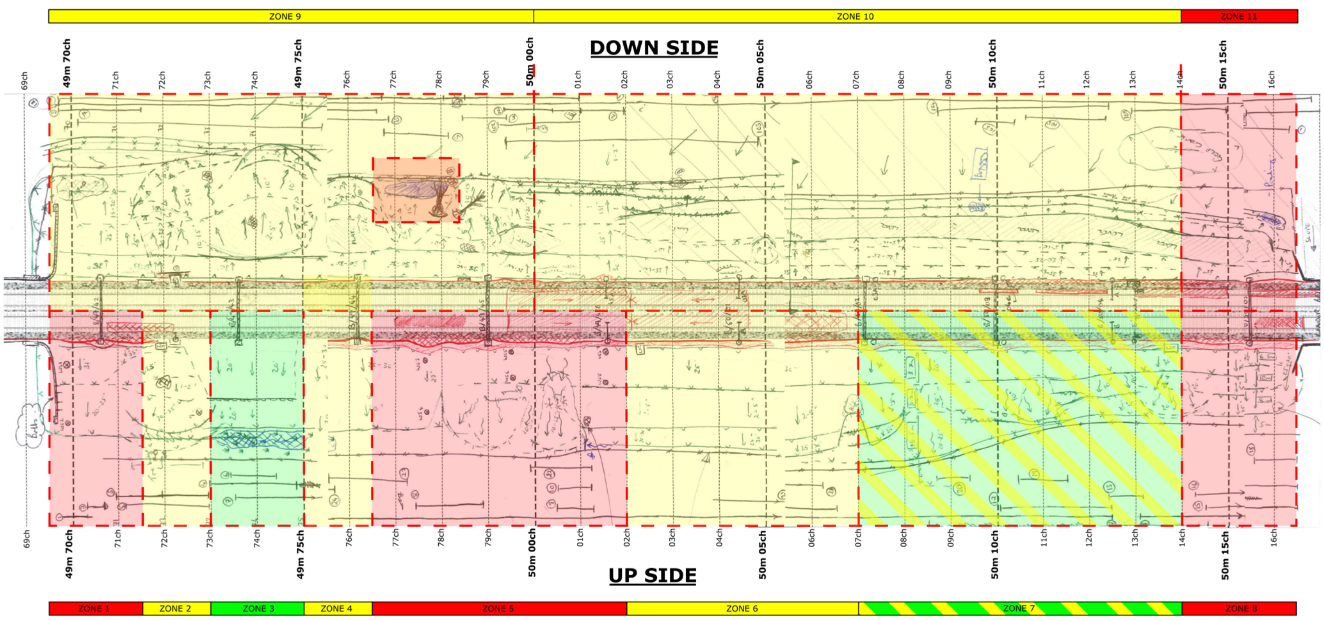
The zoning and remedial works options for the site will be further refined during the final phases of TAM Stage 1 once the ongoing GI works are completed and during the TAM Stage 2 scope ratification process, ensuring that the chosen remedial works are robust and fulfil Network Rails requirements.

FIGURE 12

Chitts Hill site TAM
risk zonation

Risk Categorisation Scoring

Initial Risk Map (Risk Zone)	Side	Mileage	1 (Track and maintenance data)		2 (Existing GI data)		3 (Visual site observations)				
			A Service Impacts	B Track Maintenance and Defects	A Instruments	B Ground	A Track	B Slope	C Drainage	D 3rd Party	E Vermin
Zone 1	Up	49m 69.5ch to 49m 71.5ch	?	M	H	H	H	L	L	L	H
Zone 2	Up	49m 71.5ch to 49m 73ch	?	M	?	M	H	H	H	L	H
Zone 3	Up	49m 73ch to 49m 75ch	?	L	?	?	L	L	H	L	M
Zone 4	Up	49m 75ch to 49m 76.5ch	?	M	?	?	M	M	H	L	M
Zone 5	Up	49m 76.5ch to 50m 02ch	?	H	H	H	H	H	M	L	M
Zone 6	Up	50m 02ch to 50m 07ch	?	M	M	M	M	M	M	L	M
Zone 7	Up	50m 07ch to 50m 14ch	?	M	?	?	L	M	M	L	M
Zone 8	Up	50m 14ch to 50m 16.5ch	?	H	?	M	H	M	M	L	H
Zone 9	Down	49m 69.5ch to 50m 00ch	?	M	?	?	L	H	H	L	M
Zone 10	Down	50m 00ch to 50m 14ch	?	M	?	?	M	L	M	L	M
Zone 11	Down	50m 14ch to 50m 16.5ch	?	H	?	?	H	M	H	L	M



5. Design Optioneering

Following completion of the TAM Stage 1 Risk zonation, initial analysis and design optioneering was undertaken for the earthworks renewal at the site. Initial design options were considered for all high (red) and medium (orange) areas of the site (Figure 12). Three options were considered in the optioneering exercise; toe berm with slope regrade, soil nails with cess retention and a sheet pile retaining wall with slope regrade. The design solutions need to manage both the deeper seated failure in the embankment and the near surface failure in the upper layer of ash fill.

Outline geotechnical analysis for each option was undertaken in accordance with BS EN 1997-1:2004: Geotechnical design (including National Annex) (BSI, 2010) (BSI, 2007), NR's design guidance document NR/L3/CIV/071 (Network Rail, 2011) and BS 8006-1:2010 Code of practice for strengthened/reinforced soils and other fills (BSI, 2010). The design life of the stabilisation measures was 60 years for the soil nail option and 120 years for the slope regrade and sheet pile wall option. Rail traffic loading was taken from BS EN 1991-2 (BSI, 2010) and maintenance loading was considered over the cess area. Geotechnical design parameters were derived from the GI works based on in-situ and laboratory test results and were checked against back analyses of the embankment slopes. Groundwater level was taken at the toe of the embankment due to ponding observed at various locations along the asset and a pore pressure coefficient ru=0.2 was considered in the embankment fill material.

The outline design output indicated that all three options were feasible for renewal of the earthwork. The three options were taken forward to TAM Stage 2 in which construction pricing was estimated, the soil nail option was selected as the preferred option following this assessment.

6. Asset Managers Perspective

During Victorian railway construction most of the fill was end tipped using horse-drawn earth wagons with little or no compaction. Settlements were large but accepted as an inevitable consequence of the construction method, initially mitigated by topping up with readily available materials such as locomotive ash and more recently mitigated by speed restrictions and ballast packing. Monitoring slope movements at various depths allows the identification of clear failure modes with deep seated failures in the over-steep unengineered London Clay embankment and shallower near surface movements associated with degradation of the upper embankment layer of ash fill evident at Chitts Hill in common with many other embankments in the Anglia region.

The Targeted Asset Management (TAM) process has proved useful in managing the challenges faced by infrastructure managers holding limited budgets and high performance expectations of rail customers. Detailed interrogation of slope movement trends, track problems and the development of a ground model enabled optimised mitigation for higher risk zones of the embankment slope, demonstrating sound asset management. The Chitts Hill site was subdivided into areas of low, medium and high risk against a set of failure criteria with the view that initially only the higher risk and some of the medium-risk areas would be mitigated earlier. The earthworks renewal portfolio is subject to budget constraints and the use of a more innovative risk-based and focused mitigations releases funding to maximise the number of site that can be treated across the wider regional earthworks portfolio.

Risks associated with other adjacent untreated areas are controlled through ongoing observational and instrumentation monitoring. With increased rail traffic and continued unpredictable weather in the future, it is important for Network Rail to have such a proactive strategy of Planned interventions to avoid needing shorter term Reactive or Emergency management of slope instability which would result in much less cost-effective asset management.

7. Conclusion

The Chitts Hill embankment site demonstrates the challenges of working on large linear earthworks that have been in constant use for over 150 years. Old earthworks such as this experience a variety of issues with fundamentality different underlying causes. A holistic approach is required for the investigation of the assets in order to fully understand the driving causes behind the issues to ensure that an appropriate remediation strategy is implemented. Access difficulties and limited availability of historical records add to the difficulty of investigating the assets.

A pragmatic approach is required to ensure that necessary works are undertaken to maintain the safe operation of the railway within limited available budgets. This has been achieved at the Chitts Hill site through the use of the Targeted Asset Management (TAM) approach which has allowed renewal works to be targeted at the worst affected areas of the asset whilst risk control measures are implemented at other high priority sections.

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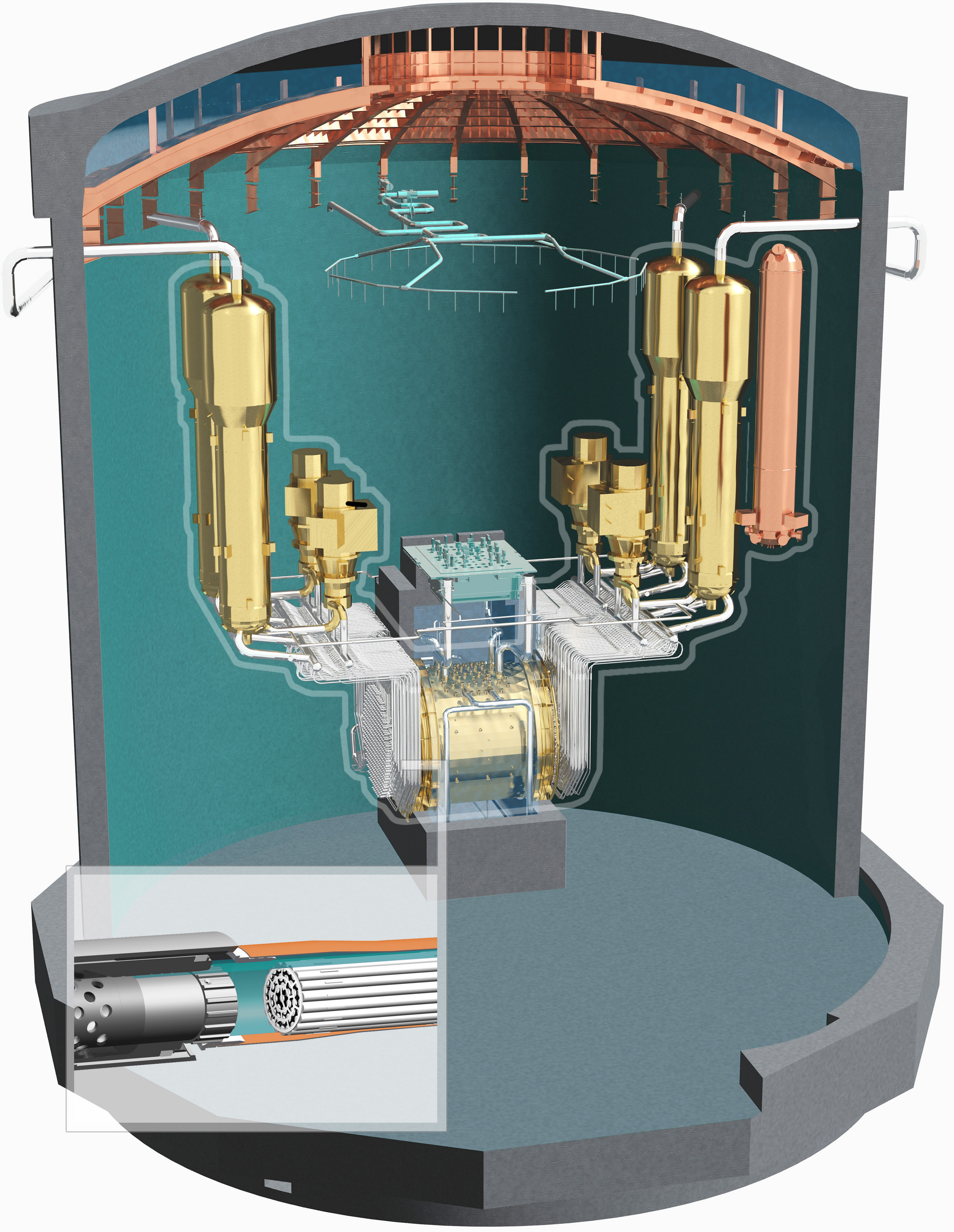

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Structures Asset Management

06: PRAISE-CANDU – A Probabilistic Fracture Mechanics Software for Evaluating Piping Systems

Abstract

PRAISE-CANDU, a Monte Carlo based Probabilistic Fracture Mechanics (PFM) software has been developed for use in fitness-for-service assessment, risk-informed in-service inspection, and large-break loss-of-coolant accident reclassification. The software has the capability to model crack initiation, crack growth and crack stability for degradation mechanisms including fatigue, stress corrosion cracking (SCC), and flow accelerated corrosion (FAC) as well as material aging (i.e., time dependent material properties). The combination of all four mechanisms, which is important for CANDU nuclear reactor piping systems, is a unique feature of PRAISE-CANDU. Application of the current version of PRIASE-CANDU can be easily extended to any industry with piping systems. Moreover, the modular design of the software allows it to be modified or appended to analyse other structures.

This paper provides a general overview of the software and its capabilities along with some discussion on each of the modules. The validation results for several benchmarking test cases with other PFM software and plant operating experience for fatigue, PWSCC and FAC degradation mechanism are also presented.

KEYWORDS

Probabilistic Fracture Mechanics Software; Fitness-for-Service; Monte Carlo; Fatigue and Stress Corrosion Cracking

Nomenclature and Acronyms

ASME	American Society of Mechanical Engineers
B&PV	Boiler and Pressure Vessel
BWR	Boiling Water Reactor
CANDU	CANada Deuterium Uranium, AECL trademark
COD	Crack Opening Displacement
CSA	Canadian Standards Association
CSNI	Committee on the Safety of Nuclear Installations
DDM	Data-Driven Methodology
DFM	Deterministic Fracture Mechanics
DMW	Dissimilar Metal Weld
FAC	Flow Accelerated Corrosion
FEA	Finite Element Analysis
IAEA	International Atomic Energy Agency
IGSCC	Intergranular Stress Corrosion Cracking
K	Stress Intensity Factor
LBB	Leak-Before-Break
LBLOCA	large-break loss-of-coolant-accident
NURBIM	NUclear Risk Based Inspection Methodology for passive components
OPEX	Operating Experience
PFM	Probabilistic Fracture Mechanics
PHT	Primary Heat Transport
PRAISE	Piping Reliability Analysis Including Seismic Events
PTW	Part-Through Wall
PWR	Pressurized Water Reactor
PWSCC	Primary Water Stress Corrosion Cracking
SCC	Stress Corrosion Cracking
SMR	Small Module Reactor
TW	Through-Wall
WGIAGE	Working Group of Integrity and Ageing of Structures
xLPR	extremely Low Probability of Rupture

1. Introduction

The use of Probabilistic Fracture Mechanics (PFM) has drawn increasing interest in the evaluation of fracture response and reliability of degraded structures. Its capability to account for uncertainties in individual parameter has provided insight for risk-informed decision making for design and fitness-for-service assessments.

In nuclear piping system, deterministic Leak-Before-Break (LBB) analysis is a widely used fracture mechanics methodology. It is used to demonstrate that a postulated through-wall crack can be detected by the available leakage monitoring systems without challenging the pipe's capability to withstand any design basis accident loading. As a supplemental LBB requirement, operator's response time (i.e., the time from the detectable leakage to unstable crack size) is also considered.

The major input parameters in satisfying LBB for any piping systems are the applied stress, material properties, leak rate estimation, and leak detection capability. Deterministic LBB analysis provides the ratio of crack sizes at detectable leak rate and rupture and time to failure. All of these parameters are accompanied by uncertainty, either from lack of enough data or inherent randomness. To account for the uncertainty in the input parameters and their propagation to outputs, safety factors are applied for an analysis with a postulated through-wall crack, i.e., it is assumed that a crack has initiated and grown through-wall. For example, U.S. NRC Standard Review Plan 3.6.3 employs a safety factor of 2, 10, and 1.4 on the rupture crack size, detectable leak rate and loads, respectively, for deterministic LBB analysis.

The compounding of conservatism in individual input parameters sometimes results in overly conservative results. PFM method provides a more realistic result by considering a few of the input parameters to be distributed based on the measurement data, thus eliminating the need for conservative safety factors. Uncertainties in the fracture mechanics model are also bounded by considering the input variables as random. Additionally, surface crack initiation, surface crack growth, and in-service inspection can be considered to investigate their effects on the failure probability. PFM methodology is not limited to LBB applications, but can also be used for inspection strategy optimization, fitness-for-service assessment, high energy line break assessment, mid-life design change, and large-break loss-of-coolant-accident (LBLOCA) reclassification of CANDU primary heat transport (PHT) and auxiliary piping systems.

2. PRAISE-CANDU Overview

PRAISE-CANDU is a Monte Carlo-based PFM software and it stands for Piping Reliability Analysis Including Seismic Events for CANada Deuterium Uranium reactor and was primarily developed for nuclear piping systems. This Grade A[9] software was developed in compliance with CSA Standard N286.7-16 Quality Assurance of Analytical, Scientific and Design Computer Programs for Nuclear Power Plants[10]. The software has the capability to model crack initiation and crack growth due to fatigue and stress corrosion cracking (SCC) and to model both circumferential and axial crack orientation in a cylindrical structure, like pipes. Extensive verification and validation have been performed on PRAISE-CANDU for the purpose of software quality assurance. The following are a few of the software design features:

- The capability to perform both probabilistic and deterministic fracture mechanics analyses.
- Modular construction for ease of future changes.
- Consideration of both design and non-design loads.
- Time-dependent material properties.
- Inspection model that includes sizing uncertainty and repair.
- Consideration of wall thinning from inside surface.
- Its capability to solve a large number of realizations efficiently.
- Supports batch run.

The current version of the software, PRAISE-CANDU 2.1, was deployed in Version 4.6 of Microsoft® .NET (C#) Framework. The software has the capability to run in both Windows 7 and Windows 10 and 32 bit and 64-bit platforms.

PRAISE-CANDU provides a probabilistic treatment of initiation and growth of cracks (SCC and fatigue). Figure 1 illustrates all the analysis options available to the user. Combination of these features leads to the following 14 degradation mechanisms.

1. Fatigue Initiation and Growth
2. Fatigue Growth Only
3. SCC Initiation and Growth
4. SCC Growth Only
5. Fatigue Initiation and Growth with FAC
6. Fatigue Growth with FAC
7. SCC Initiation and Growth with FAC

8. SCC Growth with FAC
9. Fatigue Initiation and Growth with SCC Growth
10. Fatigue Growth with SCC Growth
11. SCC Initiation and Growth with Fatigue Growth
12. Fatigue Initiation and Growth with SCC Growth and FAC
13. Fatigue Growth with SCC Growth and FAC
14. SCC Initiation and Growth with Fatigue Growth and FAC

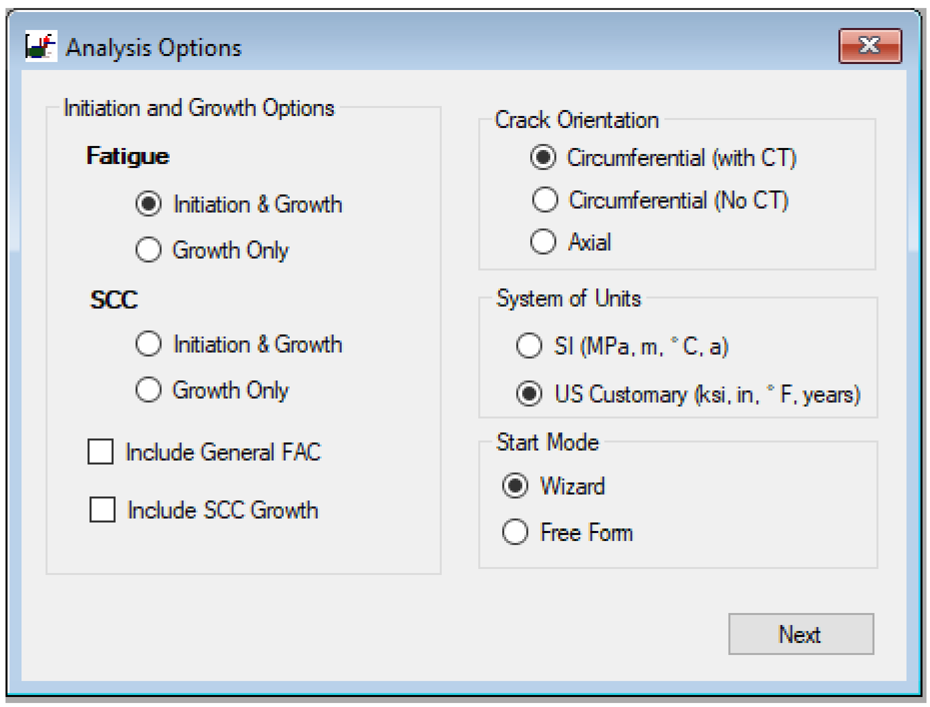
Three options are available for crack orientation:

a. Circumferential crack with crack transitioning (with CT)
b. Circumferential crack without crack transitioning (no CT)
c. Axial crack with crack transitioning

In total, there are 42 (14×3) analysis options.

FIGURE 1

Analysis options for PRAISE-CANDU 2.1



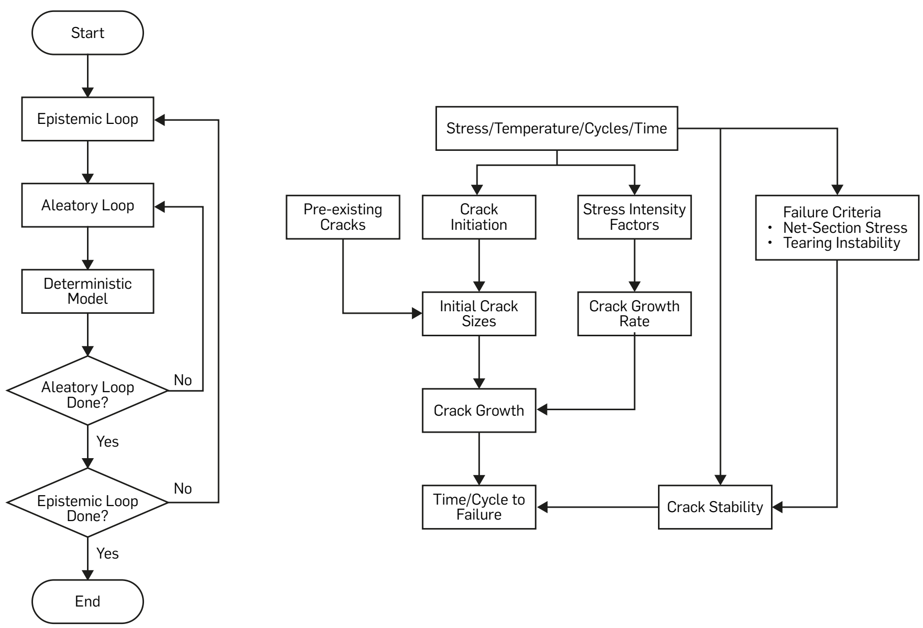
3. General Architecture

In PRAISE-CANDU, Monte Carlo simulation is employed to obtain probability results from the underlying deterministic lifetime (time-to-rupture, etc.) models. Monte Carlo simulation is conceptually simple and has been widely applied to many topics. Basically, Monte Carlo simulation involves sampling the input random variables from their respective distributions to define a set of inputs to the deterministic lifetime models. A proper random number generator is key to obtaining meaningful results.

PRAISE-CANDU also has the capability to separate the randomness due to inherent randomness (aleatory) and lack of knowledge (epistemic) by implementing a two-loop system as shown in Figure 2(a). The inner loop is a conventional Monte Carlo simulation using the aleatory random variables that defines one (time-dependent) result, while the epistemic random variables are sampled in the outer loop. For each epistemic sampling, a set of aleatory Monte Carlo simulations is performed. Each Monte Carlo simulation follows the deterministic modules as shown in Figure 2(b). The user has options to implement a two-loop or single-loop simulation.

FIGURE 2

Analysis data flow



4. Modules in PRAISE-CANDU

Only a few of the important modules/deterministic models in PRAISE-CANDU software are discussed in this section. These deterministic models implemented were adopted from the latest developments available in recognized codes, standards, and peer reviewed papers, and hence making the code slightly more realistic and accurate. The references are provided in sub-sections below.

4.1. CRACK INITIATION AND POSITIONING

PRAISE-CANDU has the capability to model both pre-existing crack and initiating cracks and crack initiation due to fatigue cycle or SCC can be modelled. Fatigue crack initiation model in PRAISE-CANDU is based on the fatigue life curves developed by Argonne National Laboratory[11]. Power Law[1] and Garud[15] initiation model has been implemented for SCC degradation. The key inputs to these initiation models can have deterministic input or have any one of the distribution types as defined in Section 4.10.

For circumferential crack, multiple cracks can initiate and grow in the segments that compose a given weldment. Initiated multiple cracks are placed randomly within their segment, while the initiated crack sizes are user-defined input parameter. Multiple cracks, if present in a piping system, can coalesce as they grow, which are based on Section XI, Article IWA-3000 of the ASME B&PV Code, 2017 edition for part-through wall crack. If there are two through-wall cracks, they will coalesce when the crack tips touch. The length of the new crack is equal to the sum of the lengths of each crack. For axially oriented cracks, only one crack is considered.

4.2. CRACK GROWTH MODULE

In PRAISE-CANDU, once a crack initiates, the crack grows from a Part-Through Wall (PTW) crack to Through-Wall (TW) crack. Its growth is controlled by the crack-tip stress intensity factor (K) and it depends on the loading condition and pipe geometry. The rate of growth is controlled either by fatigue or SCC degradation mechanism or both combined.

- Fatigue Crack Growth: Fatigue Crack growth rate can be defined as R-Dependent Paris law based on Reference[2] or tabular input with respect to ΔK and R-ratio (Kmin/Kmax). A simplified version of the fatigue crack growth relation described in ASME Code Case N-643-2[4] is also implemented.

- SCC crack Growth: PWSCC crack growth developed for Alloy 600 and related weld materials and expanded under Reference [19] as part of xLPR 1.0 development is implemented. IGSCC model is based on ASME B&PV Code[3] is also implemented.

4.3. STRESS INTENSITY FACTOR

For axial crack, K solutions for PTW cracks in cylinders based on API 579-1/ ASME FFS-1[7] is implemented, while the K solution based on Article C-7420, Appendix C of ASME B&PV Code XI[3] is used for TW axial cracks.

Circumferential crack PTW K solution is based on validated curve fits provided in ASME B&PV Code Appendix A (Article-3532)[5]. For TW, K solution is based on Tahakshi[28].

4.4. LOADS

Loads/stresses are a key input parameter. They control the initiation, growth, and final instability of cracks. Service stresses, residual stresses, and seismic stresses are the major classes. Service stresses are input to PRAISE-CANDU by a table of forces, moments, pressure, temperature gradients, and number of cycles. These piping loads are used to define inputs to the computation of the crack-tip SIF for crack growth, crack driving force, crack displacement, and for the net section stress stability assessment. For the PTW crack, the peak stress is a calculated crack by use of the ASME B&PV Code Equation (11) of NB-3650 of Section III[6]. Both the uniform through-wall stress due to pressure (p), axial force (F), and moment (M) and through-wall gradient stress due to temperature gradient are considered.

Seismic loads are input separately from service loads in the form of tables, and they contribute to both crack growth and stability calculations, and they are not considered for axially oriented crack. Residual stresses can be input in terms of a table or in form of equations with linear and circumferential variations and are only considered for PTW K calculation.

4.5. INSPECTION MODULE

In PRAISE-CANDU, the inspection procedure consists of detecting, sizing, and possible remedial action. PRAISE-CANDU employs either a tabular input of detection probability as a function of crack depth, or two closed-form equations with respect to crack depth and probability of not detecting a crack. PRAISE-CANDU also has the capability to “size” the crack, depending on the user-defined inspection uncertainty. When the calculated crack “size” is less than the user defined repair depth, the simulation continues with the crack remaining in service. If the crack "size" is greater, then some remedial action is assumed. The type of mitigations include a weld repair, replacement or a mid-life remedial action depending on the user input.

A weld repair is considered to be removal of the defect and filling the removed area with weld metal. This itself can introduce defects of a random size, which are then subject to inspection. The post-repair defect distribution is user defined. Replacement involves making the weld essentially defect-free.

4.6. CRACK INSTABILITY

Crack stability module determines the critical crack length based on the material property and loading conditions. In addition to the TW crack stability module, in PRAISE-CANDU, a crack stability assessment is performed for PTW cracks for scenarios when a surface crack can rupture before the crack becomes TW and the leak could become detectable.

PRAISE-CANDU has the capability to calculate instability by net section collapse or J-tearing instability or both. For circumferential crack, the net section collapse is based on Reference[18], which considered the effect of multiple cracks (PTW and TW) on the stability of the pipe.

For axial crack, limit load solution for both PTW and TW and J-integral for PTW crack from Vol. 3 of Ductile Fracture Handbook [32] is implemented, while for TW crack, J-integral from Reference[17] is implemented.

4.7. CRACK OPENING DISPLACEMENT

A TW crack subject to external loads would cause the crack to open and leak. The pipe leakage is mainly dependent on the crack opening and this opening is termed as Crack Opening Displacement (COD) and it depends on the crack length, loads and the material property. In PRAISE-CANDU, for circumferential crack, COD estimation scheme is based on Reference[32]. This solution is empirically adjusted to account for the large deformation theory, crack-tip plasticity, and includes bending correction due to axial force. Additionally, the solution is applicable for various Rm/t and also provides COD at inner and outer surface. For axial crack, the COD solution is based on Reference[8].

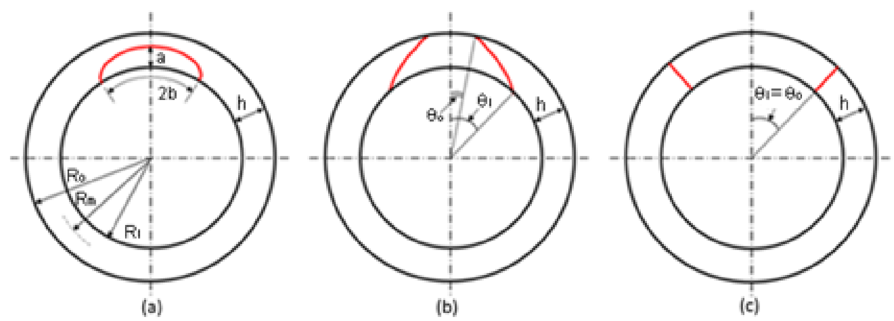
4.8. CRACK TRANSITIONING

Through the years of the PRAISE-CANDU development, significant advancement has been made on how the crack transitions from PTW to TW crack. It was a general practice that once the depth of the PTW (Figure 3(a)) circumferential crack exceeds 95% of the wall thickness, an idealized through-wall circumferential crack is considered (Figure 3(c)), i.e., the crack angle through the wall thickness is treated as constant. Analysis with this type of crack transition corresponds to the selection of Circumferential (No CT) option in Figure 1. However, based on the References[21],[22], [26], the approach of idealized TW crack transition does not accurately represent the actual behaviour.

In PRAISE-CANDU, the latest crack transitioning model is implemented based on Reference[23], which includes the crack transition model for both K solution and COD for the natural transitioning circumferential cracks by the selection of Circumferential (with CT) option in Figure 1. For axial cracks, crack transition model as described in References [24], [25] is implemented in PRAISE-CANDU.

FIGURE 3

Circumferential cracks in cylinder



4.9. FLOW ACCELERATED CORROSION

Thinning of the pipe wall from the inside surface can occur due to flow accelerated corrosion (FAC). Of the commonly used piping steels, carbon steel is the most susceptible. In PRAISE-CANDU, wall loss due to FAC is considered to occur all around the circumference with an axial extent that allows the problem to be considered as simply a thinner pipe. In PRAISE-CANDU, the time-dependent FAC rate can be defined to depict a more realistic corrosion process.

4.10. NUMERICAL SAMPLING

PRAISE-CANDU computes the probability of leak opening areas of various sizes and rupture as functions of time for a location in a piping system. Many of the inputs to the deterministic model are subject to inherent statistical scatter and/or uncertainty. This scatter is characterized by considering some of the inputs to be random variables, whose distributions are based on analysis of data or engineering judgment. Random variables in PRAISE-CANDU can have one of six distribution types shown below.

- normal
- lognormal
- Weibull (exponential and Rayleigh as special cases)
- symmetrical triangular
- uniform
- tabular

5. Validation Activities

Several benchmark activities were performed with an earlier version of the code in 2013[13] and[29] with then available PFM software and theoretical solutions and good agreement in results were observed. In this paper, benchmarking results with the latest version of the code are presented.

5.1. NURBIM PROJECT FATIGUE CASES

The fatigue benchmark problem was established in the NURBIM (NUclear Risk Based Inspection Methodology for passive components) Project[20] using four PFM computer programs. The benchmark consists of base cases and sensitivity studies for small, medium, and large pipe. The dimensions of the pipe are listed in Table 1. The growth parameters and random variables used in the analysis are listed in Table 2 and Table 3.

TABLE 1:

Pipe dimensions - NURBIM

	Outer Diameter (mm)	Wall Thickness (mm)
Small pipe	88.9	11.1
Medium pipe	324	33.3
Large pipe	861	62.2

TABLE 2:

Fatigue crack growth parameters - NURBIM

	C, mm/cycle/(MPa $m^{1/2})^{m*}$	n*
Low	1×10^{-9}	3.93
Base	5.06×10^{-9}	3.93
High	1×10^{-8}	3.93

*$da/dN = C \cdot (\Delta K)^n$

TABLE 3:

Random variables for sensitivity analysis - NURBIM

Random Variable	Low		Base		High	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
Yield Strength (MPa)	75	7.5	150	15	225	22.5
Ultimate Strength (MPa)	225	15	450	35	675	45
Yield Strength (MPa)	150	8.4	300	16.8	450	25.2
Fracture Toughness (MPa $m^{1/2}$)	140.6	10.6	265.7	20	444.7	33.5
Number of Cycles	250	0	500	0	1000	0
Crack Aspect Ratio	1	1E-3	3	1E-3	10	1E-3

Sensitivity studies were performed on crack aspect ratio, crack depth and density, yield strength, flow strength, loads, number of cycles, crack growth rate, inspection quality, and inspection interval. A total of 63 cases were performed using PRAISE-CANDU. Some of the results compared against the four PFM software used in NURBIM are shown in Figure 4 and Figure 5. Percentage differences of the calculated probabilities are shown in Table 4. Overall, PRAISE-CANDU produces slightly conservative results when compared to other software's.

TABLE 4:

NURBIM fatigue case – Percentage differences in calculated probabilities

Software	Percentage Difference %*									
	Base Case			Sensitivity Study - Crack Depth			Sensitivity Study - Inspection POD			
	Small Pipe	Medium Pipe	Large Pipe	Low	Base	High	No	Poor	Good	Advanced
WinPraise 4.31	-42	-52	-83	-62	-50	-39	-51	9	6	-14
PROST	-54	-39	-15	-49	-37	-26	-39	-13	9	6
ProSACC 0.92	-65	-67	-90	-76	-67	-52	~	~	~	~
PRODIGAL	107	-27	202	-3	-26	-44	-27	111	-14	~

** Percentage Difference = (Comparing software – PRAISE-CANDU 2.1)/ PRAISE-CANDU 2.1*100*

FIGURE 4

Comparison of base case for PRAISE-CANDU 2.1 - NURBIM fatigue case

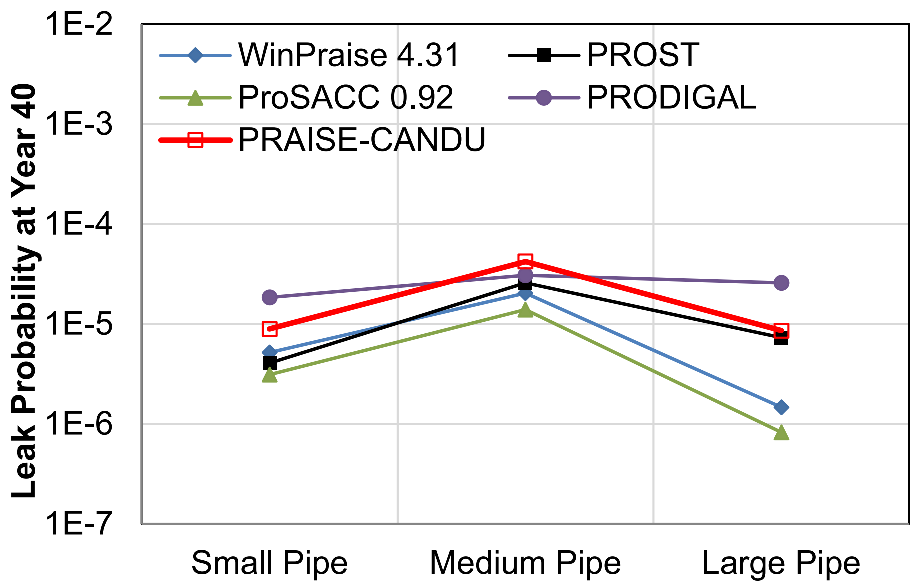
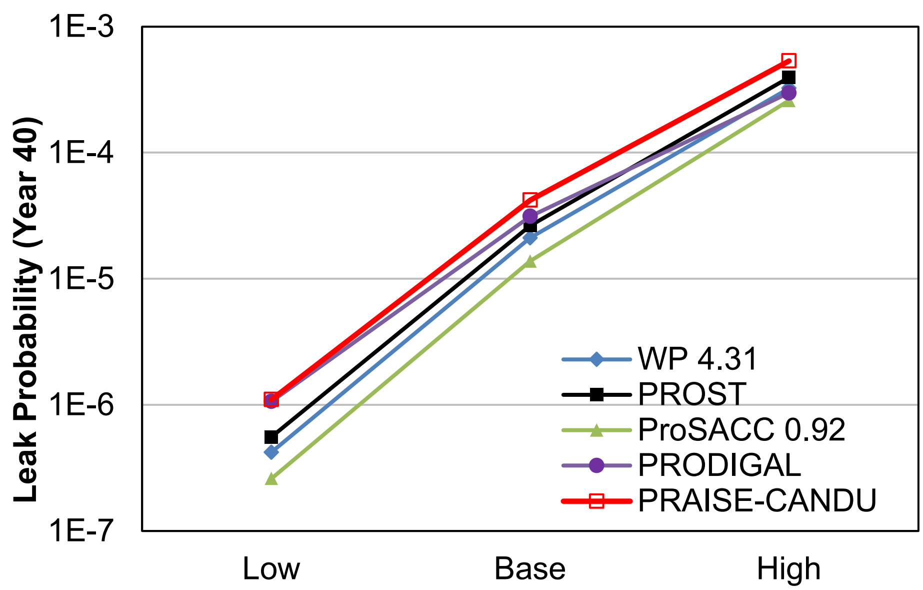
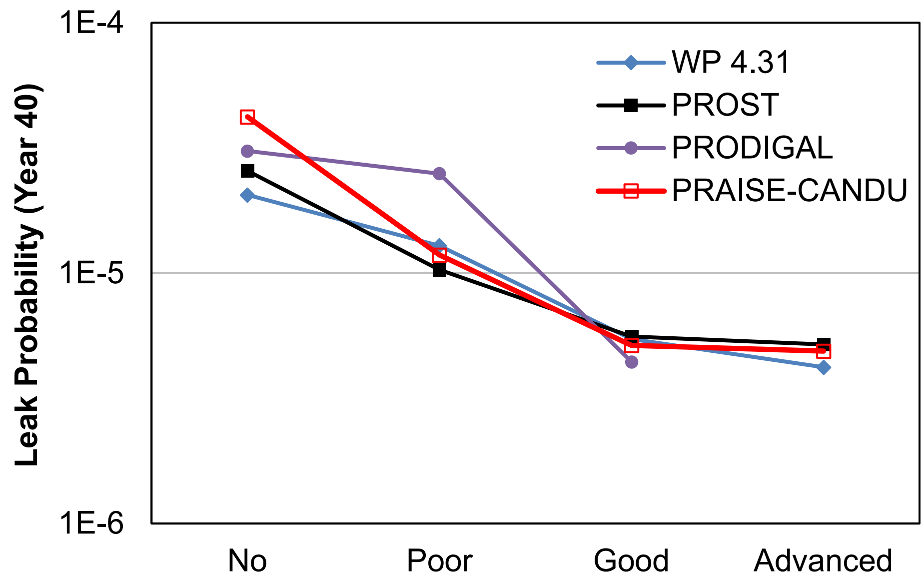


FIGURE 5

Comparison of some of the sensitivity studies for medium pipe size for PRAISE-CANDU 2.1 – NURBIM fatigue case



(a) Crack Depth



(b) Inspection POD

5.2. IAEA BENCHMARK WITH OPEX - PWSCC CASE

Worldwide Operating Experience (OPEX) with respect to primary water stress corrosion cracking (PWSCC) applicable to Alloy 60/82/182 in large-diameter reactor coolant systems is considered for the benchmarking activities. PFM results were benchmarked against failure probability calculated using data-driven methodology (DDM). Details of the DDM analysis are provided in Reference[16].

PFM analyses were performed to calculate the failure probabilities and frequencies of various break size over 40 years. The simulation considered 16 initiation sites around the pipe circumference of a pipe with an outside diameter of 952.1mm and wall thickness of 82.55mm. The in-service inspection is performed at 6th, 16th, 26th and 26th year. The random variables are listed in Table 5. One billion Monte Carlo realizations are used in order to obtain the extremely low failure probability with sufficient accuracy. Two-step analysis was performed: the simulation was first calibrated against the annual leak probability obtained from OPEX using DDM methodology, and then the failure frequencies at various break sizes were predicted. Figure 6 shows the comparison of the annual frequencies calculated from PRAISE-CANDU results and DDM-based results and the results are in excellent agreement.

TABLE 5:

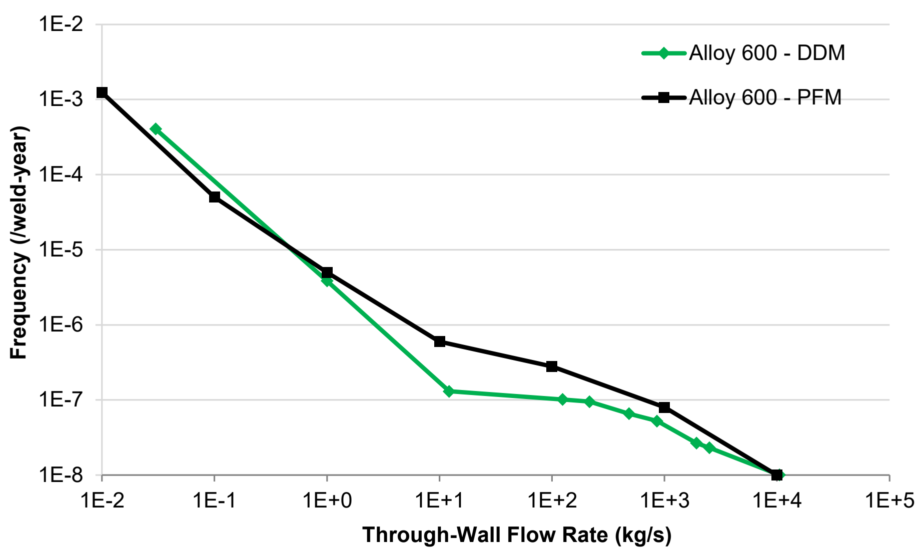
Random variables OPEX –PWSCC case

Random Variable	Distribution Type	Parameter 1	Parameter 2
Crack initiation coefficient of Garud Relation (year)	Lognormal	1.39×10^{-13}	0.7301
PWSCC growth rate coeficient (mm/year)	Lognormal	6.311×10^{-2}	0.6071
Initial crack depth (mm)	Normal	3	0.75
Initial crack length (mm)	Normal	9	0.15
Flow strength (MPa)	Normal	400	33
Yield stress (MPa)	Normal	172.5	27.3
Ramberg-Osgood coefficient (MPa)	Normal	563.8	43.6
Axisymmetric WRS profile, ID WRS stress (MPa)	Normal	350	60
Axisymmetric WRS profile, zero crossing	Normal	0.25	0.05

Note: For lognormal distribution, the parameter 1 is median value and parameter 2 is shape parameter. For normal distribution, the parameter 1 is mean value and parameter 2 is standard deviation.

FIGURE 6

Comparison of annual failure frequency for PRAISE-CANDU 2.1 – OPEX – PWSCC case



5.3. CSNI BENCHMARK – PWSCC CASE

The metals sub-group of the Working Group of Integrity and Ageing of Component and Structures (WGIAGE) of the Committee on the Safety of Nuclear Installations (CSNI) of the Nuclear Energy Agency (OECD/NEA) launched an activity on the Benchmark on Probabilistic Fracture Mechanics for Piping Applications (PFM Benchmark) in 2021. The main objective of the program is to understand the differences in the PFM codes and the methodology implemented by each organization across the world. As an initial stage of the program, deterministic fracture mechanics (DFM) models implemented in the PFM software were compared across various codes (15 in total). As this is an ongoing project, only the results of deterministic analysis are provided in this paper. The pipe geometry along with initial crack size (circumferential) and loading conditions are provided in Table 6. For this benchmark project, PWSCC degradation mechanism is used, and the crack growth model is shown in Table 7. Figure 7 shows the box-and-whisker plot of time to first leak and time to rupture estimated by various computer codes and PRAISE-CANDU 2.1 is around the median value of 18 submissions.

TABLE 6:

Geometry and loads - CSNI benchmark

Variable	Value
Initial Crack Depth (m)	1.50E-3
Initial Crack Half-Length (m)	3.00E-3
Pipe Outside Diameter (m)	0.38
Pipe Wall Thickness (m)	0.04
Operating Temperature (°C)	320
Operating Pressure (MPa)	15.5
Total Axial Membrane Stress (MPa)	0.117
Total Bending Stress (MPa)	30.05
Yield Strength, Weld and Base Materials (MPa)	316.5
Ultimate Strength, Weld and Base Materials (MPa)	542.4
Elastic Modulus, Weld and Base Materials (MPa)	196,800
Weld Residual Stress (MPa)	Linearly varying from +400 at ID surface to -400 at OD surface

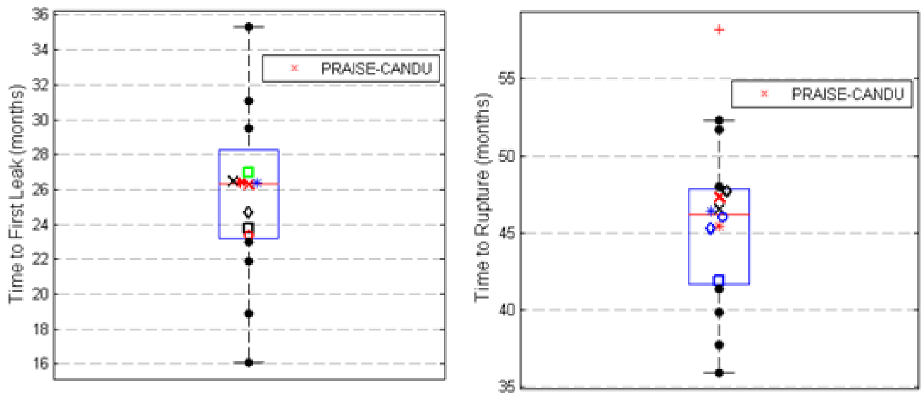
TABLE 7:

Crack growth model

Variable	Value
Crack Growth Model	$\dot{a} = \left[e^{-\frac{Q}{R}\left(\frac{1}{T} - \frac{T}{T_{ref}}\right)} \right] \alpha f K^{\beta}$
Q (kJ/mol)	130
R (kJ/mol-K)	$8.314\mathrm{x}10^{-3}$
T_{ref} (K)	598.15
α (m/s)/(MPa-$m^{0.5}$)	$2.16\ \mathrm{x}10^{-12}$
f	1.0
β	1.6

FIGURE 7

Comparison of time to leak and rupture for PRAISE-CANDU 2.1 – CSNI benchmark PWSCC base



6. Applications

The PRAISE-CANDU has been developed as the primary focus for applications like fitness-for-service, LBLOCA and LBB for CANDU reactors. However, the software can be used for other applications. For example, probability of cracking in Dissimilar Metal Weld (DMW) in weldments between SA-106 and Alloy 600 has been studied in References [30] and[12]. Effect of inspection interval and method for optimizing the inspection interval has been detailed in Reference[31] for CANDU reactor feeder pipes when subjected to FAC.

Given the general concept of fracture mechanism, the software can be used for applications in Boiling Water Reactor (BWR), Pressurized Water Reactor (PWR), Small Module Reactor (SMR), and oil/gas downstream piping systems.

7. Conclusion

PRAISE-CANDU is a state-of-the-art PFM code that includes all of the latest developments in the field of probabilistic fracture mechanics for pipes. The code can be used for various applications including probabilistic safety/risk assessment, optimization of inspection program, and fitness-for-service assessments. Currently, the code is applicable to any piping structure. However, it would be expanded to analyse other structures, as well.

The software has been developed under a software quality assurance program in full compliance with CSA N286.7-16.

Acknowledgements

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07: Improved Structural Health Monitoring of Great Belt Bridge Hangers and Deck Using Digital Image Correlation

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Abstract

Informed decisions on timely intervention for effective bridge maintenance activities rely on good quality, accurate, and reliable asset condition data. The principal difficulty with adding several traditional monitoring systems is that they produce vast quantities of inconsistent data and are labour intensive. Gathering information requires structural health monitoring (SHM) and inspection and, for it to be useful, it must be accurate, inexpensive, and easy to interpret, and must avoid interfering with traffic flows (whether rail or highways). Digital image correlation (DIC) is a noncontact photogrammetry technique that can be used for monitoring by imaging a bridge component periodically and computing movement and deformation from images without traffic disruption. This paper describes the use of DIC for the monitoring of the Great Belt Bridge wind-induced hanger vibrations and traffic-induced bridge deck displacements. The paper also presents how vision-based monitoring helped to better understand the structural behavior of key suspension bridge components without any traffic disruption. To the authors knowledge, these are one of the first such long-term SHM campaigns carried out on a major suspension bridge.

KEYWORDS

Structural Health Monitoring; Digital Image Correlation; Bridges; Cable vibrations; Fatigue

1. Maintenance of Global Transportation Infrastructure

The ongoing cost of maintaining the world's physical infrastructure is extremely high. Tens of billions of dollars is spent annually on repairing bridge structures that have deteriorated under loads and environmental conditions to the stage where expensive reactive maintenance is required. Civil infrastructure system owners are often faced with a challenging set of management decisions. Despite the growing gap in building new infrastructure, the worldwide stock of existing infrastructure is worth about $50 trillion USD, which is of the same order of magnitude as the global stock market capitalization ($55 trillion USD) and comparable, to a certain extent, to the global GDP ($72 trillion USD). This existing stock offers a great opportunity to narrow the infrastructure gap if governments are capable and willing to optimize the operations and maintenance (O&M) of their infrastructure assets (Wong & Almedia 2014). The ongoing cost of maintaining the world's physical infrastructure is extremely high. Considering bridges alone, it's estimated that some $70 billion USD is spent annually on repairing bridge structures that have deteriorated under loads and environmental conditions to the stage where expensive reactive maintenance is required.

The principal difficulty with adding several traditional monitoring systems is that they produce vast quantities of inconsistent data and are labor intensive. Asset managers or bridge operators typically do not know what to do with this data unless there are very clear trigger levels associated with this data and clear interventions defined if they are exceeded. Gathering information requires structural health monitoring and inspection on a grand scale and, for it to be useful, it must be accurate, inexpensive, easy to interpret, and avoid interfering with traffic flows (whether rail or highways).

Therefore, there is a need to introduce recent innovations in digital technologies, such as vision-based remote sensing to improve current SHM strategies, optimize performance and enhance the efficiency of maintenance programs. This paper presents how vision-based monitoring helped to better understand the structural behavior of key Great Belt Bridge structural components without any traffic disruption.

2. Structural Health Monitoring of Bridges - Challenges

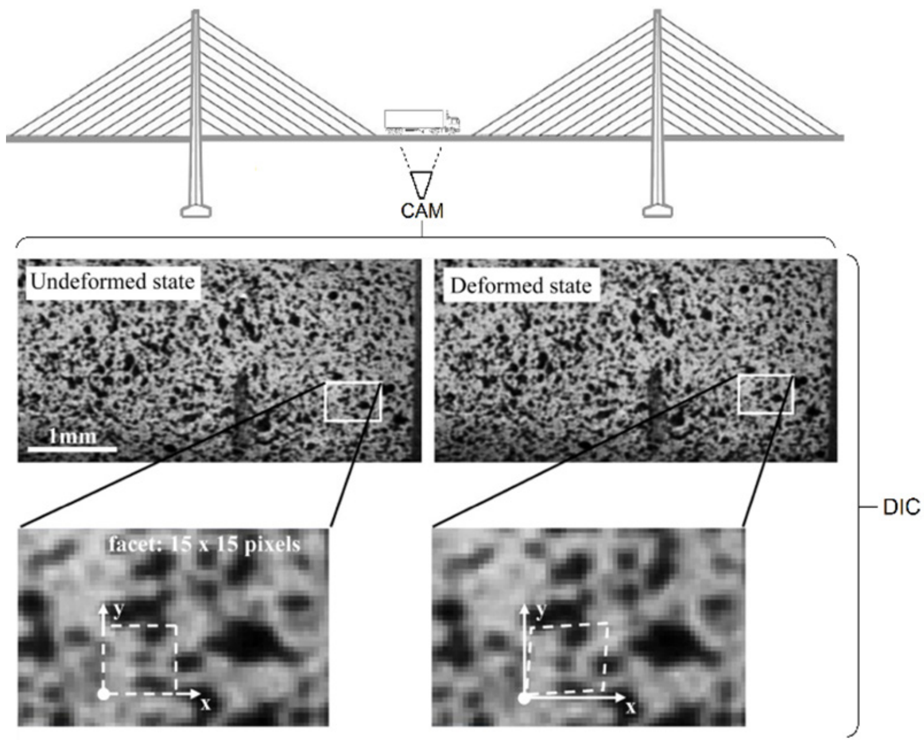
Instruments such as strain gauges, accelerometers, fibre optic sensors, and displacement transducers are becoming increasingly common in structural health monitoring. These types of sensors can, however, possess drawbacks such as the need for external power and cabling/antenna for data transmission, high data acquisition channel counts and the limitation of only measuring at discrete points or along a line, so it is necessary to have an idea of where to expect damage when placing the instrumentation. These sensors can be used effectively to continuously monitor abnormalities that indicate damage, but the type and severity of the damage can still be difficult to identify from discrete point measurements. Furthermore, asset managers are continuously searching for new technologies that will allow them to gather information about their structures without traffic disruption.

3. Digital Image Correlation (DIC)

DIC represents a photogrammetry technique used for accurate measurements of surface deformation. The digitized images (e.g., of a bridge deck) are compared to match facets from one image to another by using an image correlation algorithm (Fig. 1). Image analysis involves capturing a reference image of a bridge component surface in its undeformed state. As the load is applied (e.g., traffic or train load), additional images are collected. The algorithm involves a stage-wise analysis in which each stage consists of one image resulting in a description of displacements occurring on the surface of the bridge component. The evaluation of a correlation measurement results in coordinates, deformations, and strains of the surface.

FIGURE 1

Digital image correlation technique



Real-time operation, off-line analysis, remote access to equipment, and live reporting of results is possible and has been used. The DIC system can be operated with as little as one individual and does not have to be in contact with the bridge, therefore avoiding any potential conflicts with traffic or difficult geography.

3.1. CAMERA-BASED MONITORING METHODS IN BRIDGE ENGINEERING

Research activity on the application of photogrammetry in bridge-related projects has been widely dispersed within the last 30 years (Jiang et al. 2008). Early applications of this technique include measurement of the deflection of a three-span steel bridge under dead load (Jauregui et al. 2003) and identification of the bridge deformation (Bales 1985). DIC was also employed to measure the geometry of a suspension bridge (Li & Yuan, 1988) and deformation of steel connection of a pedestrian bridge (Johnson 2001). It was shown that DIC is a complementary tool of the conventional measuring systems such as LVDTs and strain gages. The last decade have seen an increased application of the DIC in the measurement of the vertical deflections of steel and concrete bridges due to traffic (De Roover et al. 2002, Lee & Shinozuka 2006a, b, Santini-Bell et al. 2011, Chiang et al. 2011, Yoneyama & Ueda 2009, Yoneyama et al. 2007) and train transit (Busca et al. 2012).

In general, it was concluded that DIC system accuracy is comparable to existing displacement measurement techniques and DIC is an easier way to measure displacement of multiple points at once. DIC was also proposed as a method to assess dynamic characteristics of suspension bridge cables (Kim & Kim 2013). In this study, a non-contact sensing method to estimate the tension of hanger cables by using digital image processing based on a portable digital camcorder was proposed. Moreover, DIC technique has been used to record the strain on a concrete girder during a full-scale bridge failure test (Sas et al. 2012) and for the measurement of the displacement field on a cracked concrete girder during a bridge loading test (Küntz et al. 2006). In both cases, the photogrammetry method was able to detect a change in loading condition.

Recent applications of DIC include fatigue testing of bridge stay cables. Here, the vision-based system allowed for the measurement of the interwire movement (fretting fatigue) being the governing mechanism responsible for the fatigue life reduction in modern stay cable assemblies (Winkler et al. 2014, Winkler et al. 2015). DIC was also employed in structural monitoring to measure deflections and strain at fatigue-sensitive bridge detail with a view to avoid the need for strengthening (Winkler & Hendy 2017).

4. Great Belt Bridge

The Great Belt East Bridge has a central span of 1624m with 535m side spans and pylons reaching up to 254m. The bridge carries a four-lane motorway plus emergency lanes. A 31m wide welded steel box girder is continuous between the anchor blocks over the whole suspension bridge length (Fig. 2).

FIGURE 2

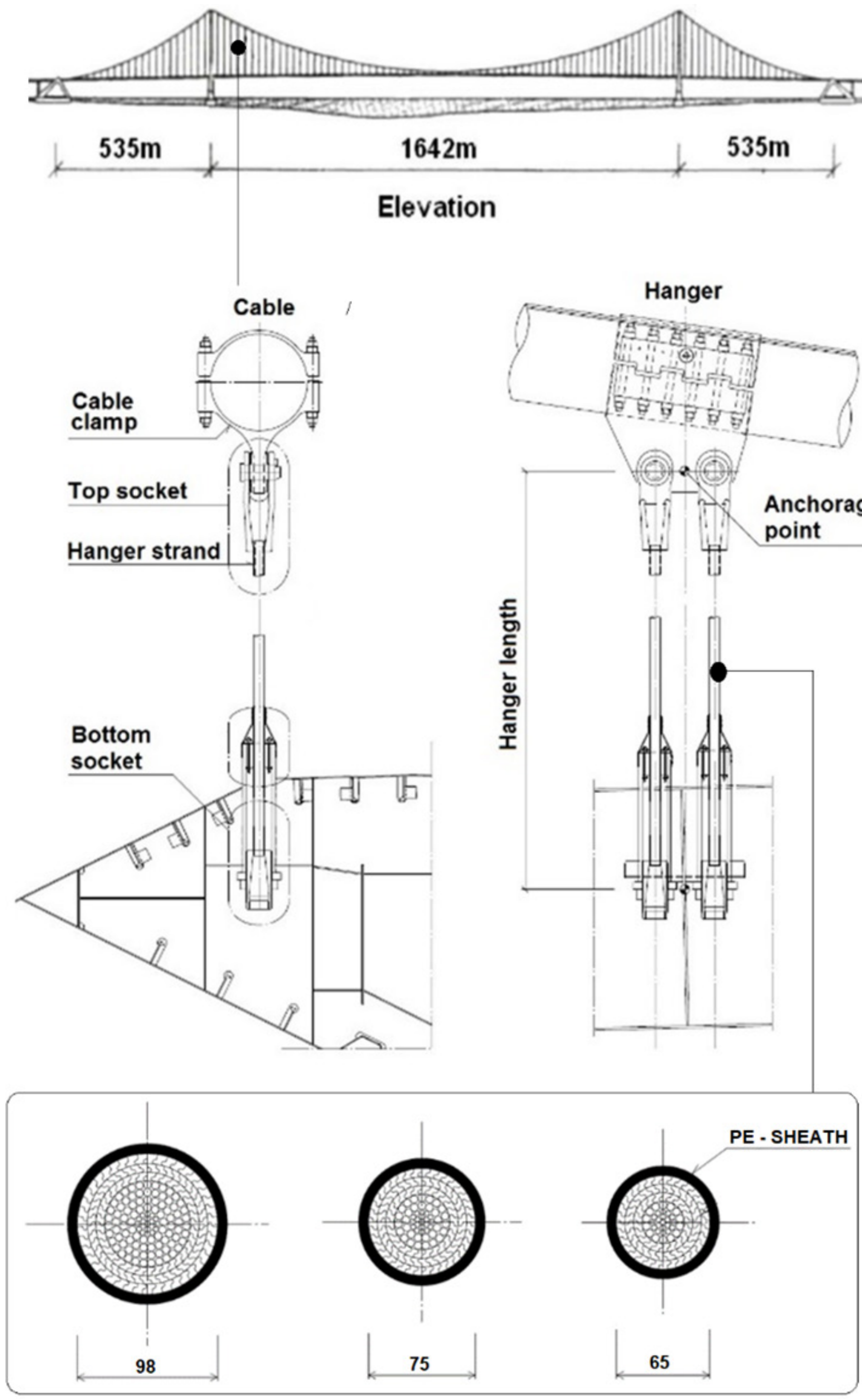
Great Belt East Bridge

5. Hangers & Deck – Structural Details

The Great Belt Bridge girder is supported every 24m by pairs of hangers (helical ropes) protected by a polyurethane (PE) sheath. The hangers have been designed so that one pair of hangers can be removed whilst being replaced without disrupting the traffic on the bridge (Fig. 3).

FIGURE 3

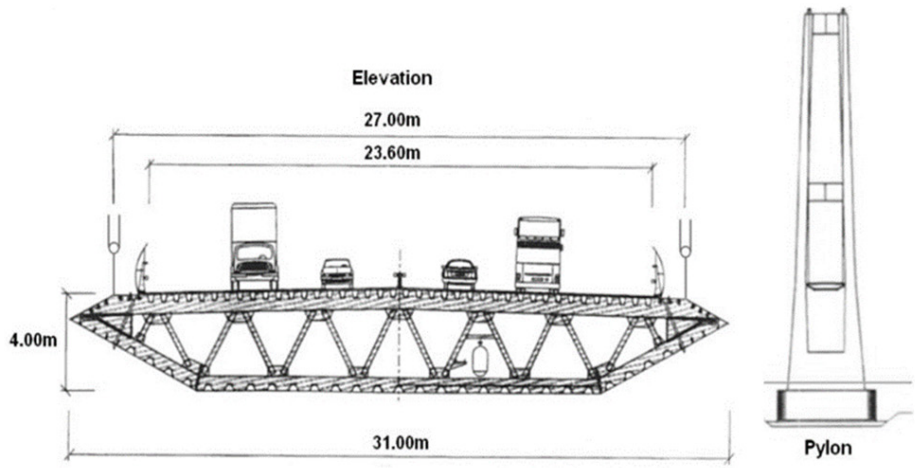
Hanger structural details



The bridge has an orthotropic steel deck (OSD) meaning it has different stiffnesses in longitudinal and transverse direction (Fig. 4). The deck consists of a structural plate stiffened with ribs, allowing it to directly bear vehicular loads and to contribute to the bridge structure's overall load-bearing behaviour. The stiffening elements enhance the bending resistance of the plate and increase the total cross-sectional area of steel in the plate.

FIGURE 4

Orthotropic steel deck structural details



6. DIC System

The DIC monitoring system was assembled on the mid-beam located 130m above the sea level (Fig. 5). High precision measurement (mm accuracy) was achieved with the camera being setup up to approx. 100m away from the hangers. The cameras were mounted within weatherproof steel housings on a steel plate designed to provide a required field of view (FoV). In addition to the camera housing, infra-red (IR) lights were also positioned in order to provide IR light during hours of darkness. The structural monitoring campaign using DIC was carried out on the longest hangers of the Great Belt Bridge (147.2, 147.1, 146.2, 146.1). Due to page limitation, only vibration monitoring data corresponding to the longest hanger (147.2) will be presented.

FIGURE 5

DIC camera system

7. Monitoring Results

The normalized greyscale correlation tracking algorithms were used for the image analysis. Calibration was achieved by measuring a distance within the image. No editing of the raw data has been undertaken. The permanent system allows for offline analysis. The DIC camera system has been successfully collecting video data for more than one year. FoV obtained from the system is shown on Figure 6.

FIGURE 6

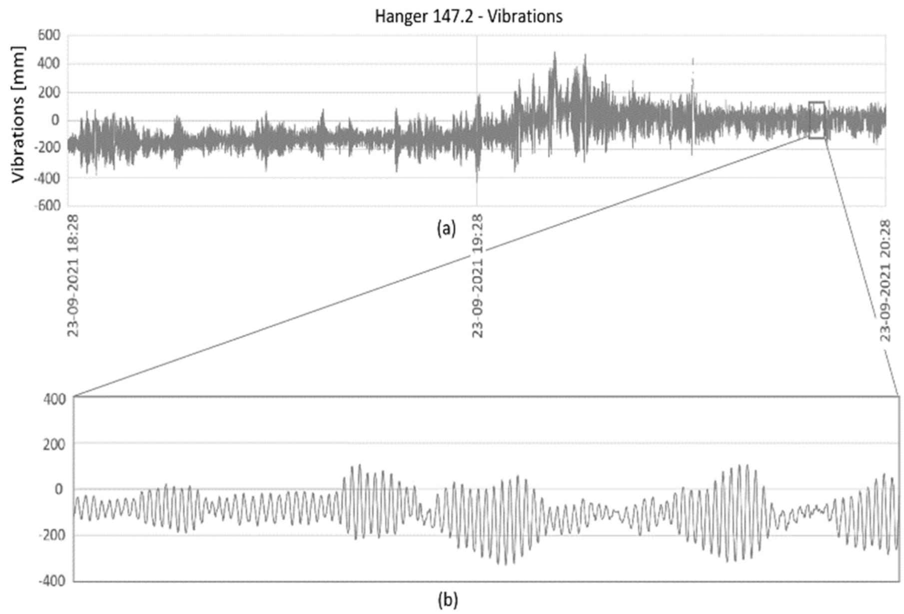
Longest hangers on the Great Belt Bridge

7.1. WIND-INDUCED HANGER VIBRATIONS

Figure 7a shows the wind-induced vibrations of hanger 147.2 (movement in horizontal direction). The corresponding wind speed range was approximately 7-12m/s. DIC data can be further used to estimate the natural frequency of the hangers. In this case, the frequency was estimated as approximately 0,45 Hz (Fig. 7b).

FIGURE 7

Wind-induced vibrations (a), estimation of natural frequency (b)



7.2. VALIDATION OF DIC DATA WITH ACCELEROMETERS

The Great Belt Bridge overall monitoring system is comprised of several different sensors, including accelerometers. Various sensors collect data that can be accessed via asset management system KonMos. The results from the vision-based DIC system were used to verify and validate the data obtained from the accelerometers. An example of such comparison can be seen on Figure 8.

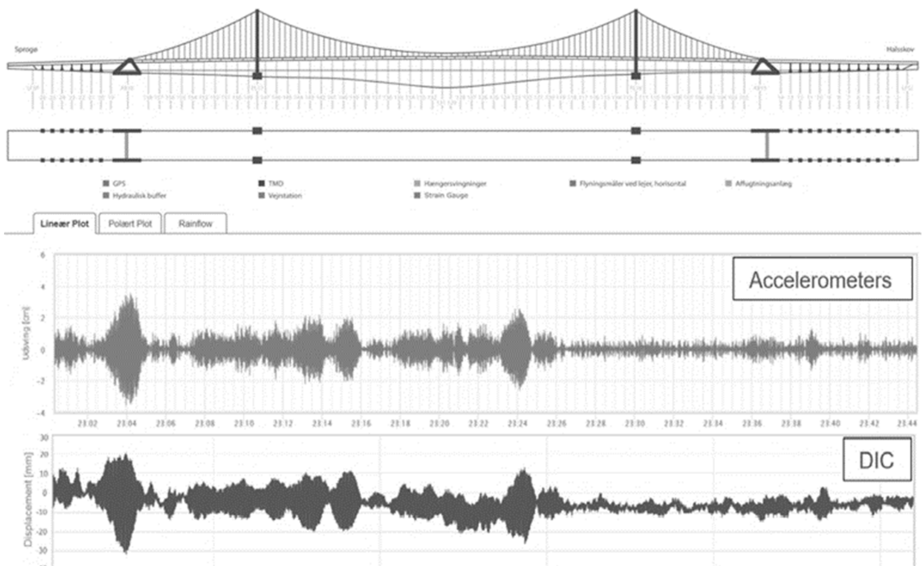


FIGURE 8

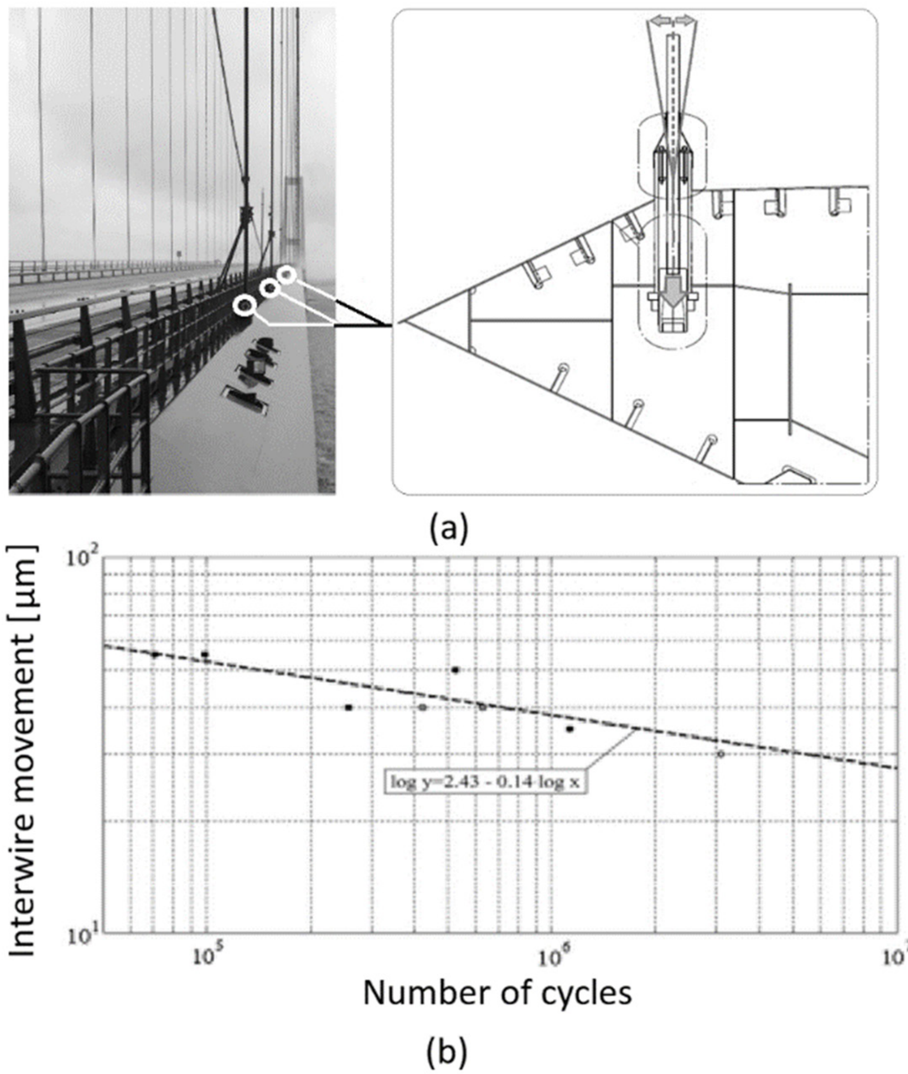
Comparison of DIC data and accelerometer data

7.3. FRETTING & BENDING FATIGUE OF HANGERS

The monitoring data can be used to correlate peak amplitude with bending stress and inter-wire movement (fretting fatigue) at hanger sockets (Fig. 9a). Subsequently, the DIC and accelerometer monitoring data can then be used for comprehensive hanger fatigue life analysis. Having information about the global movement of hangers and local fatigue behaviour, one can use a relatively new S-N fatigue curve (Fig. 9b) that accounts for fretting and bending stress at anchorages to estimate the remaining service life of hangers (Winkler et al. 2015, fib Bulletin 89). Even though the abovementioned SN curve was developed for steel monostrands and not for helical ropes, it still provides valuable insight into estimation of the fatigue life.

FIGURE 9

Hanger anchorage detail (a), S-N curve for fretting fatigue of monostrands (b)

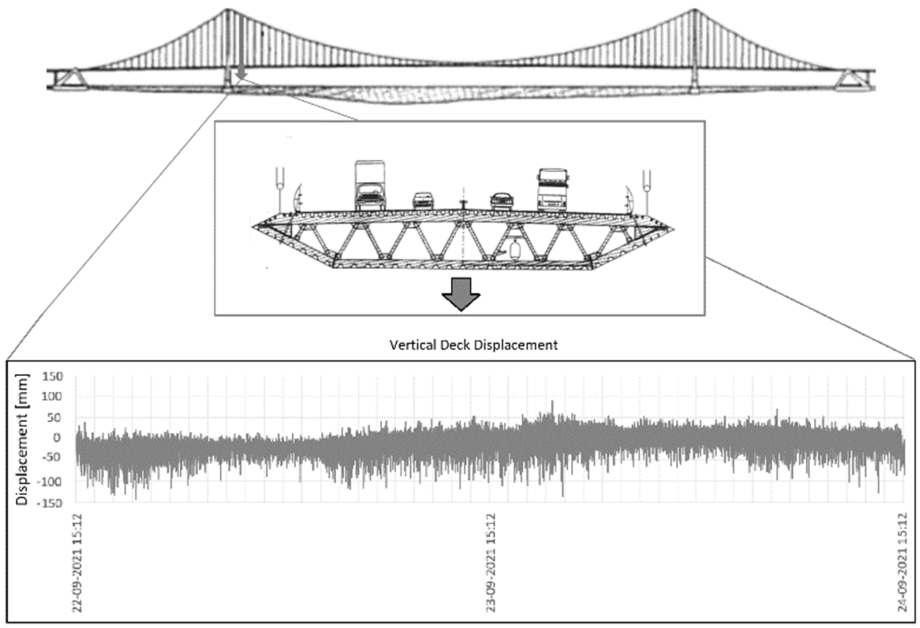


7.4. TRAFFIC-INDUCED DECK DISPLACEMENTS

The DIC system also provided statistically relevant data on traffic load. From the collected data, it can be seen that the bridge deck predominantly deflects vertically within a range of 100mm (Fig. 10). This information can be used to estimate the average stress range of a hanger (axial load variations), which can then be used for fatigue analysis and estimation of remaining service life.

FIGURE 10

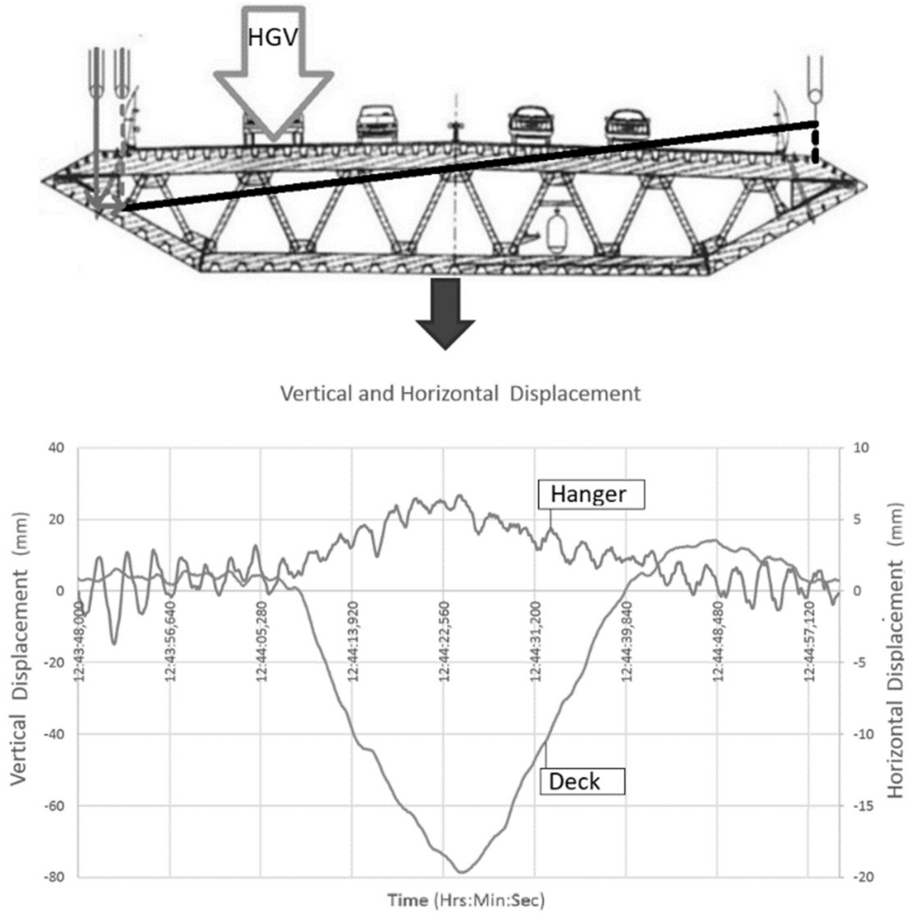
Vertical deck displacement



One of the key observations from the DIC monitoring is the relationship between the overall vertical and horizontal displacement of the hangers. It is evident that whilst the hanger displaces vertically, there is also an overall horizontal displacement. This displacement can be attributed to the base of the hanger moving towards the centre of the deck as a result of deck rotation when subject to loading from wind and a Heavy Goods Vehicle (HGV) on one side of the structure. This concept is illustrated in Figure 11.

FIGURE 11

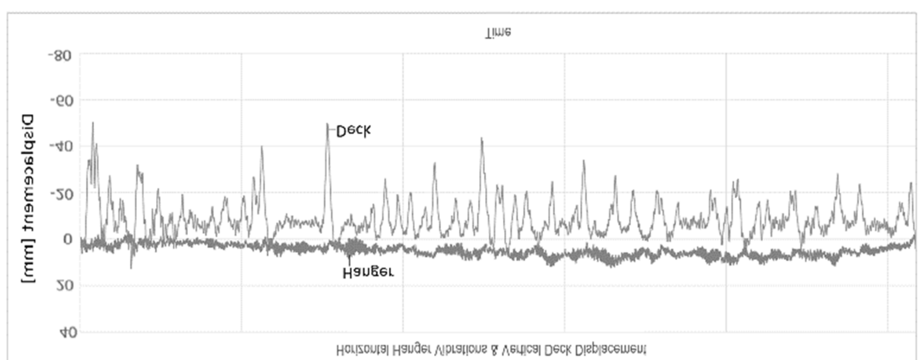
Bridge deck and hanger displacements



It can be seen from the graph below (Fig. 12) that the local wind induced movements should be considered together with the global hanger movements caused by the deck rotation. Both local and global cyclic hanger movements will affect the fatigue life.

FIGURE 12

Simultaneous hanger and deck movements



8. Conclusions

Atkins was appointed by Sund & Bælt to carry out structural monitoring and to quantify wind-induced hanger vibrations and traffic-induced deck displacement of the Great Belt East Bridge. The main goal of the project was to develop a reliable and robust system that enables acquisition of statistically relevant monitoring data to be compared with the accelerometer system.

The monitoring system proposed by Atkins was based on Digital Image Correlation (DIC). DIC is a vision-based, contactless, high-precision measurement technique that employs cameras and image correlation algorithms to calculate movement, deformation, and stress on a bridge component from the pixel movements without any traffic disruption.

The DIC camera-based system was used as a cost-effective data gathering tool to provide information on the structural behaviour of bridge components. The system was remotely controlled via LAN connection from the Atkins office.

In total, 12 months of DIC videos from 2021 were collected and secured. Moreover, monitoring data from months with significant wind conditions were acquired.

The DIC system allowed us to collect high precision data (-/+ 1mm precision from approximately 100m) and provided previously unavailable monitoring data.

The results of the long-term monitoring showed a good agreement between the data collected using DIC and the data received from the accelerometer system.

From the collected data, it can be seen that the bridge deck predominantly deflects vertically within a range of 100mm.

DIC provided new quality of data, much higher precision, and gave the ability for visual data verification and offline analysis of the videos/images. Even very small hanger movements were picked up. The offline analysis capability can be used in several cases. For example, there can be a windy period where the data from accelerometers will be questionable. In such cases, it is possible to re-analyse the DIC data from a certain period and measure corresponding hanger vibrations.

Furthermore, monitoring data on traffic loads were also captured and can be used to verify how many (potentially overloaded) HGVs are crossing the bridge per month.

One of the findings of this monitoring campaign is that DIC data showed that vertical displacements (due to heavy traffic) are inducing deck rotation, causing the hanger to displace globally. This, in turn, is most likely inducing local rotation at the hanger anchorage and local bending, and fretting fatigue. The accelerometers are only measuring hanger movements due to wind but are not providing information on global hanger displacements. To properly estimate fatigue life, the local wind-induced movements should be considered together with the global hanger anchorage movements due to heavy traffic load.

The outcome of monitoring campaigns provided previously inaccessible information, helped to better understand the behaviour of the hangers, and informed next steps with respect to the maintenance strategy.

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08: Making Our Communities More Resilient: A Case for Monitoring and Managing Aging Dams

Abstract

Many states are homes to numerous dams and water retaining structures, some of which are aging with minimal or no maintenance. Most of these structures no longer serve the purpose for which they were constructed. These structures pose a flood risk to downstream homes, businesses, schools, and highways. Some states, in conjunction with the Natural Resources Conservation Service (NRCS), have started the development of the DamWatch tool. DamWatch is a web-based application intended to provide real-time monitoring of rainfall, snowmelt, and streamflow that could threaten the safety of the dam and provide alerts prior to the failure of the dams.

Although most states recognize the need for the DamWatch tool, some are unable to develop it due to constraints like cost, time, and lack of data such as detailed topography. Dam risk data needed to build this database can be expensive to develop and up until recently have been cost prohibitive. A typical approach for this analysis is to use the Hydrologic Modeling System (HEC-HMS) for the rainfall-runoff and reservoir routing modeling and the River Analysis System (HEC-RAS) for the dam breach modeling. Some states, in conjunction with consultants such as Atkins, have developed innovative technical approaches to complete these tasks at a fraction of the cost and time it normally takes to complete similar assignments, without sacrificing the accuracy of the results. This paper discusses an innovative, simplified dam breach method that Atkins used to simulate and map dam breach inundation, and to develop dam risk products needed for emergency management throughout the U.S. state of North Carolina. The method relies on high quality terrain data in conjunction with the 2D hydraulic routing utilities in HEC-RAS and breach flood wave hydrograph development based on breach parameters estimated using regression equations.

KEYWORDS

Dams; Flood risk; Dam risk; Resiliency; Mitigation

1. Introduction

Dams are essential components of flood control and water supply infrastructure. Dams are used to impound water to create reservoirs that supply water for drinking and other household activities. These reservoirs also serve recreational purposes such as fishing, boating, and become habitats for various wildlife. The reservoirs created by dams also provide water and head for hydropower generation in addition to helping attenuate peak flood waves, thereby reducing flooding in downstream areas. Figures 1 and 2 are images of Lake Lynn dam and reservoir in North Carolina. The reservoir is used for both flood control and recreational purposes.

FIGURE 1

Lake Lynn Dam, Reservoir and downstream area, NC

FIGURE 2

Lake Lynn Reservoir in NC is used for both recreation and flood control

Dams, however, pose significant risk to downstream property and human lives when they fail. For this reason, dams require constant monitoring and maintenance. Unfortunately, many of the dams in the United States and across the globe have been left unmaintained due to various reasons. These structures become more likely to fail as they age. State emergency management agencies recognize the risk posed by these aging dams and often require dam owners to perform Emergency Action Plans (EAP) for use in the event of a failure. There are however many dams without owners, and some dam owners do not have the funds to engage the services of engineers to develop the EAPs because the traditional approach to the hydrological and hydraulic modeling needed in the development of the EAPs can be cost prohibitive. The traditional modeling approach usually involves the use of hydrological models such as the U.S. Army Corps of Engineers' (USACE) HEC-HMS model (USACE, 2000) for the rainfall-runoff and reservoir routing modeling and hydraulic models such as USACE's Hydrological Engineering Centre – River Analysis System (HEC-RAS) (USACE, 2021) for the dam breach modeling. The state of North Carolina is one of several that have partnered with Atkins to develop simplified, cost-effective approaches for modeling dam breaches and creating inundation maps and other dam risk products needed for emergency management. North Carolina is home to about 6,000 dams, most of which do not have EAPs. Developing inundation maps and other dam risk products required for emergency management using the traditional approach would be cost prohibitive and take years to develop.

It was therefore essential for Atkins to develop an alternative approach to developing the dam risk products at a fraction of the cost and time it would normally take. The approach that Atkins developed is described in this paper. The approach involves development of flood breach hydrographs based on regression estimates that were developed based on data from historical dam failures and development of HEC-RAS models on a basin scale. This approach allows for the use of only one modeling software for mapping multiple dams within a given watershed. This approach reduces the cost for developing dam risk products by more than an order of magnitude and at the same time maintains the high level of accuracy needed for emergency management.

2. Methodology

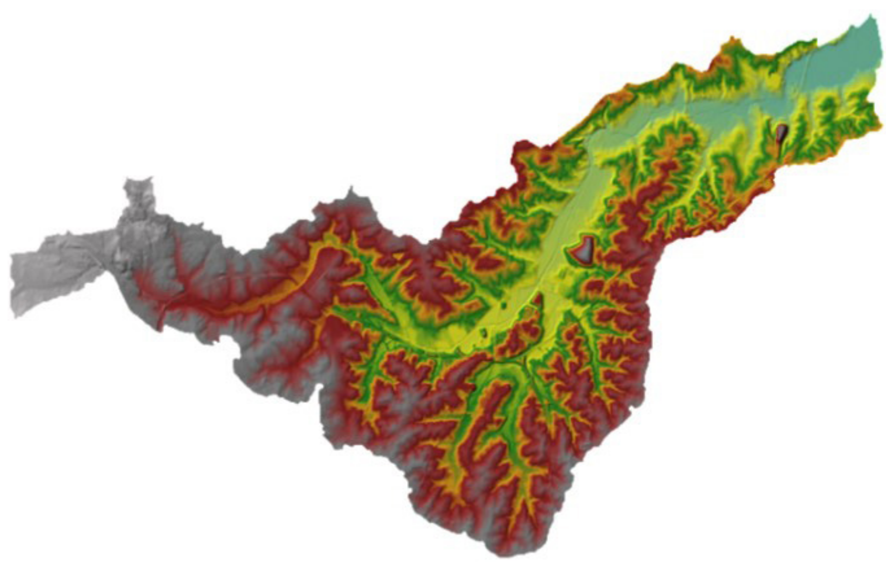
2.1. DATA COLLECTION

2.1.1 TERRAIN DATA

North Carolina is fortunate to have high resolution terrain data. From 2014 through 2018, the North Carolina Risk Management Office in partnership with the North Carolina Department of Transportation and other partners collected LiDAR data for the entire state. The project was completed in five phases. The resolution of the LiDAR data ranged from 2 points per meter during the first three phases of the data collection to 8 points per meter during the last two phases (NCEM, 2019). This data was made available to Atkins by the state as a 10-foot Digital Elevation Model (DEM). The horizontal and vertical coordinates of the terrain data were the North American Datum of 1983 (NAD83), and the North American Vertical Datum of 1988 (NAVD88), respectively. This data was used in the development of the reservoir storage parameters and for the HEC-RAS models. An example of this terrain data is shown in Figure 3.

FIGURE 3

QL2 Lidar data

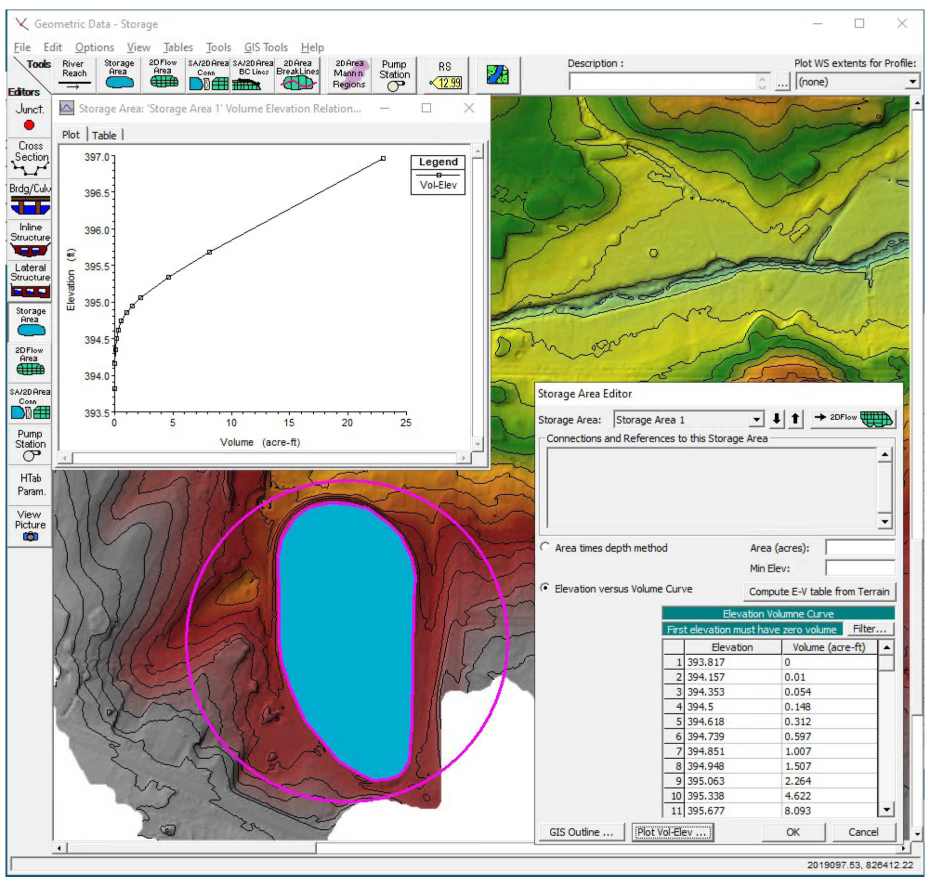


2.1.2 DAM AND RESERVOIR CHARACTERISTICS

The North Carolina Department of Environmental Quality (NCDEQ) maintains a dam inventory database within the state. The dam inventory data includes pertinent dam location and geometry data such as dam coordinates, normal and maximum impoundment storage, dam height, and surface area. The inventory data, however, is not complete and lacks data for many of the dams in the state. The North Carolina dams inventory database was first reviewed for the accuracy of the data on the dam height, maximum impoundment storage, and reservoir surface area. When available, data from the dam inventory database were used in developing the peak breach outflow. When not available from the dam inventory database, the dam height, maximum impoundment storage and reservoir surface area were estimated from a 10-foot DEM using HEC-RAS. A HEC-RAS storage area geometry file was created, and the boundaries of the reservoir were defined as a storage area. Elevation-storage information was extracted for the storage area. The storage developed from this method only accounts for storage above the normal pool water level. Storage below the normal pool water level was accounted for using the equation of a cone with surface area and dam height as inputs.

FIGURE 4

Elevation-storage data from HEC-RAS



2.2. FLOW DATA

Baseflow is an input for developing the dam breach hydrographs. North Carolina already has most of the area within the 100-year floodplain defined and mapped. The purpose of the analysis was to estimate the additional area of inundation from a breach. The 100-year flow was therefore used as a baseflow in the development of the peak breach hydrographs. In locations where there is an existing mapped 100-year floodplain, the flowrate used in developing the 100-year floodplain was used. In locations without an effective 100-year floodplain, the 100-year flowrate was estimated using StreamStats (USGS, 2016).

2.3. DAM BREACH HYDROGRAPH DEVELOPMENT

2.3.1 PEAK BREACH OUTFLOW AND TIME TO FAIL ESTIMATION

Failure of the dam could occur at any point along the perimeter of the dam embankment. The first step is to select a probable failure location which will result in the worst-case scenario in terms of peak breach outflow and area of inundation. Calculation of the peak breach flow rate and time to fail was performed based on the dam parameters at the selected breach location using two different empirical equations, and the worst-case scenario was selected for use in the breach analysis. The equations used are published in Froehlich, 1995 and Froehlich, 2008.

The Froehlich (1995) regression equation was derived based on data from 63 earthen dams. These dams ranged in height from 12 to 305 ft and have storage capacities ranging from 11 to 535,000 ac-ft. The Froehlich (2008) regression equation was derived based on data from 74 earthen dams. These dams ranged in height from 10 to 305 ft and have storage capacities ranging from 11.3 to 535,000 ac ft.

Inputs to these regression equations include maximum impoundment volume and dam height for Froehlich (1995), and maximum impoundment volume, dam height, and reservoir surface area for Froehlich (2008). The outputs from these regression equations are peak breach outflow and breach formation time.

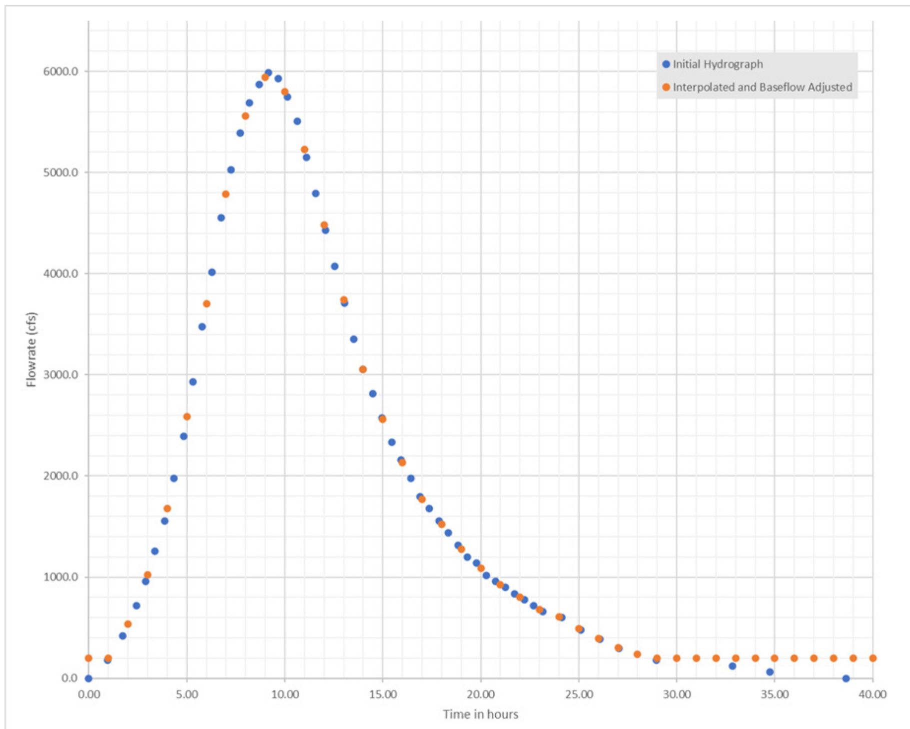
The mode of failure for the embankment was conservatively assumed to be overtopping.

2.3.2 BREACH HYDROGRAPH DEVELOPMENT

The peak discharge and time of failure computed using the regression equations were fit to the USGS Dimensionless Hydrograph (USGS, 1993) to develop the dam breach hydrograph. A 1-minute interval or less was used as the time ordinates of the hydrograph to ensure that the peak breach outflow is captured. The most conservative breach hydrograph in terms of peak breach outflow and flow volume was applied as the breach hydrograph in the hydraulic model. The baseflow (100-year peak discharge) was included as a constant baseflow at the start and end of the hydrograph. An example hydrograph is shown in Figure 5.

FIGURE 5

Dam breach hydrograph



2.4. 2D HYDRAULIC MODEL

A 2D HEC-RAS model was developed with extents from the downstream face of the dam to a downstream location where the impact of the breach wave is considered negligible. The downstream extents of the hydraulic models were determined using one of the following criteria:

1. The maximum elevation resulting from the dam breach is within 1-ft of the Federal Emergency Management Agency (FEMA) 1% elevation at this location.
2. The vertical elevation tolerance between the flood elevations with and without a dam failure at this location is less than 1 foot.
3. Where dam breach flood flows are contained within a large downstream reservoir.
4. Where dam breach flood flows enter a bay or ocean.
5. At a location where the breach flood wave takes more than 24 hours to reach. 24 hours was considered an adequate flood warning time by local emergency management.

Manning's n values were estimated using the 2016 National Land Cover Dataset (NLCD) (USGS, 2019). No hydraulic structures were modeled, though break lines and Internal Connections were used to refine the 2D mesh at crossings and significant topographic features. The assigned Manning's n values for each NLCD class are listed in Table 1. The selected Manning's n values for the various NLCD classes are appropriate based on general engineering literature (Chow, 1959) and engineering judgement.

A GIS layer of the roads and railroads within the state were enforced as break lines within the mesh. Additionally, hydraulic structures within the computational domain were simplistically accounted for by burning openings corresponding to the opening area of the hydraulic structures into the terrain to create a hydro-corrected DEM. The process of hydro-correcting the terrain was automated using python scripts.

TABLE 1:

Manning's N values for different land cover types

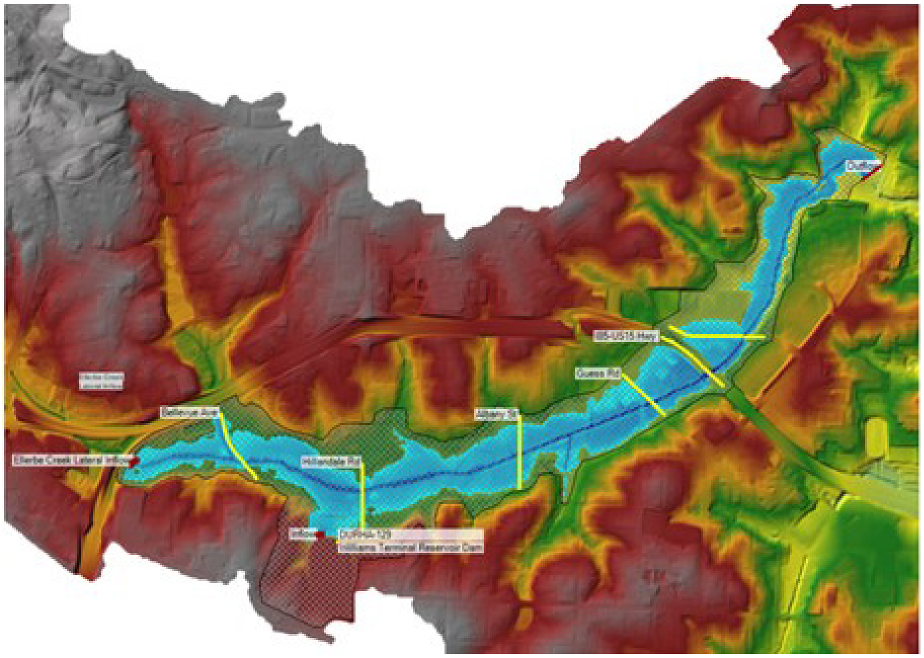
NLCD	Normal N Value	Range of n Values	Land Cover
11	0.03	0.025-0.05	Open Water
21	0.04	0.03-0.05	Developed, Open Space
22	0.1	0.08-0.12	Developed, Low Intensity
23	0.12	0.06-0.14	Developed Medium Intensity
24	0.15	0.12-0.20	Developed, High Intensity
31	0.03	0.023-0.03	Barren Land (Rock/Sand/Clay)
41	0.13	0.10-0.16	Deciduous Forest
42	0.13	0.10-0.16	Evergreen Forest
43	0.13	0.10-0.16	Mixed Forest
52	0.1	0.07-0.16	Shrub/Scrub
71	0.045	0.025-0.05	Grassland/Herbaceous
81	0.06	0.025-0.06	Pasture/Hay
82	0.06	0.025-0.06	Cultivated Crops
90	0.12	0.045-0.15	Woody Wetlands
95	0.08	0.05-0.085	Emergent Herbaceous Wetlands

Initial runs were made to further refine extents, resolution, and settings for breach simulations. The average nominal mesh size was set to 50-ft cells and was adjusted upwards as needed based on the simulation completion time. A variable time-step based on Courant condition was specified and the Courant condition was checked using velocity and cell outputs to ensure a Courant value of one is not exceeded. The 2D mesh outlet condition was set to Normal Depth, and the Full Momentum equation set was used in the hydraulic simulation.

The baseflow was run for a period to establish steady state conditions that are representative of the 100-year floodplain prior to the onset of the breach hydrograph. This was achieved by specifying a warm-up time in the simulation window for each plan. The use of a warm-up time saved hours of simulation time as compared to using a hot-start file with the results of an independent baseflow simulation. An example computational domain is shown in Figure 6.

FIGURE 6

HEC-RAS computation domain



3. Results and Discussion

Dam risk products that communicate flood risk in the downstream areas of a dam in the event of a dam failure were an essential component of the project deliverables. These products are meant to be used in the State Emergency Response Application (SERA), which is a web-based application developed by North Carolina Emergency Management, to assist emergency responders and public safety officials in responding to emergencies and hazards. The dam risk products will also be used in DamWatch. Critical information about High Hazard Dams such as dam location, type of dam and spillway etc. are populated in SERA and the dam risk products complement this data and provide useful information about downstream flooding impacts if a dam fails.

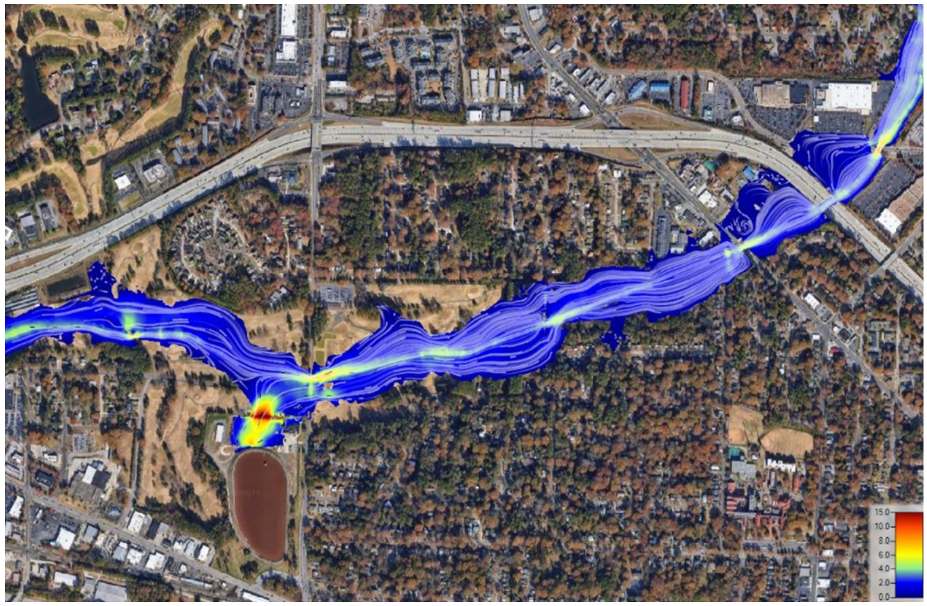
The SERA products that were created include shapefiles and rasters that were generated and exported from HEC-RAS (using RAS Mapper) after the completion of the dam breach simulation. These layers allow the end user to visualize the extents of flooding and the magnitudes of maximum depths, water surface elevations and velocities in the inundated areas. Figure 7 shows an example of using a maximum depth raster that visualizes the inundation extent during the event of a dam breach. Figure 8 is an example map showing the maximum flow velocities within the inundation area.

FIGURE 7

Location and overtopping breach inundation

FIGURE 8

Maximum velocities within inundation area



Arrival Time rasters that document the arrival time of the peak flood wave at each inundated cell in the HEC-RAS mesh are also generated in RAS Mapper.

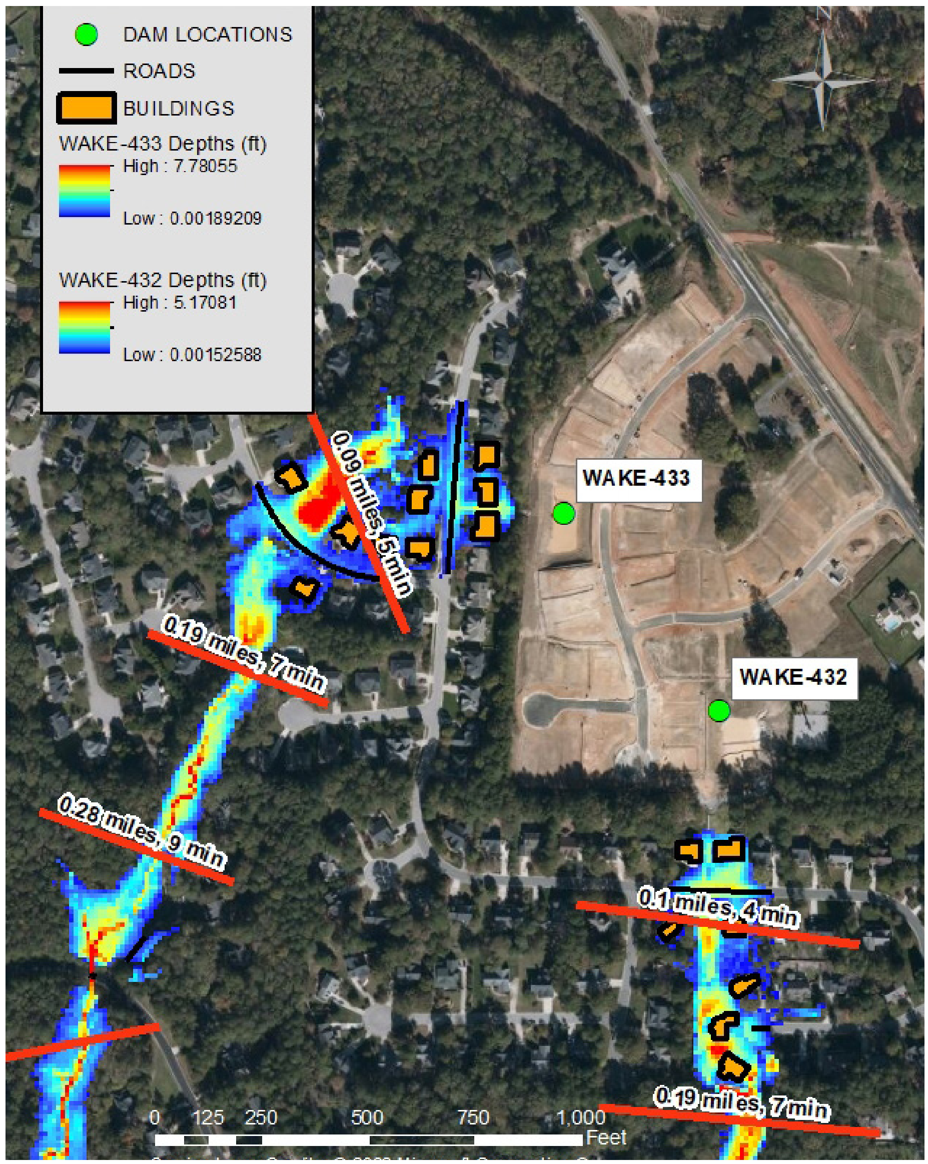
It is to be noted that the arrival time of the peak flood wave is of interest rather than the arrival time of the initial flood wave since the modelling approach assumes that a base flow is already present in the downstream channel at the time of the dam failure.

In addition to the inundation extents, SERA application also visualizes nearby buildings and roads that are predicted to be impacted by a failure event. Atkins developed shapefiles of impacted roads and buildings with information about flooding depths, water surface elevations and earliest arrival time of the peak flood wave at the building and road locations and estimated building damage costs. To create these products, existing state-wide datasets for roads and building footprints were clipped to the extent of the inundation boundary. Water surface elevations and depths at building locations were then extracted from the RAS Mapper layers discussed above and populated in the building shapefile. Damage costs at the impacted buildings due to the predicted flooding were estimated using an existing building flood risk estimation tool developed by the state of North Carolina that considers structure and occupancy characteristics and depth of flooding to estimate the total structure and content damage costs. In the case of impacted roads, dam risk information that was populated include arrival time of the peak flood wave as mentioned above and length of the road that was impacted.

A series of cross-sections along the downstream spanning the inundation boundary were generated to document the arrival times of the peak breach flood wave as it travelled downstream. Depending on the inundation area, the time difference between consecutive cross-sections was about 5 minutes for urbanized areas, and 30 minutes for rural areas. These arrival time cross-sections (and arrival time information documented at impacted buildings and roads as discussed above) provide useful information that can be used to determine when to evacuate a specific location, where to build flood barrier structures and for developing an overall emergency response plan. Figure 9 shows an example of arrival time cross-sections that document how far downstream of the dam they are located and the arrival time of the peak flood wave at that location.

FIGURE 9

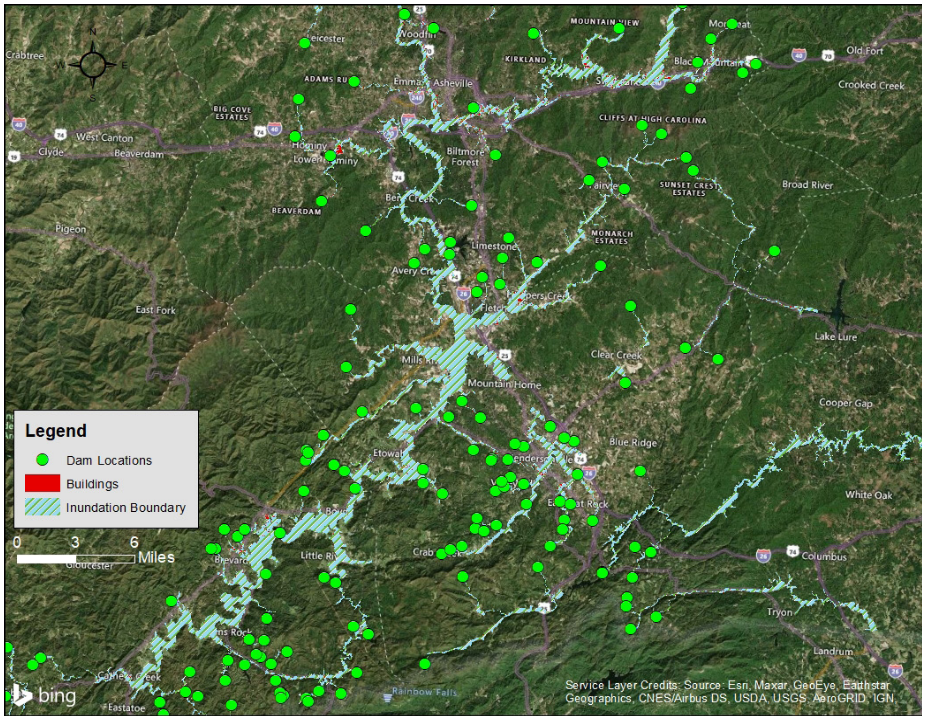
Impacted buildings, roads and arrival time cross-sections



The above SERA products were created for dams across state. Figure 10 provides an overview of the inundation mapping layers created for a sample set of dams. Geospatial programming techniques were leveraged to develop the SERA products in a cost-effective, timely and error-free manner.

FIGURE 10

Overview of downstream inundation maps created for a sample set of dams within the project scope



4. Conclusions

This paper details a cost-effective approach to modeling, mapping, and development of dam risk products for hundreds of dams within the State of North Carolina. The method relies on high quality terrain data in conjunction with the 2D hydraulic routing utilities in HEC-RAS, and breach flood wave hydrograph development based on breach parameters estimated using regression equations. The innovative approaches described in this paper will enable towns, regions, states and even entire countries to cost effectively plan and prepare for the eventual failure of the many dams that have been left unattended to over the years. The dam risk products detailed in this paper provide essential information for identifying communities and infrastructure at risk in the event a dam fails, for warning downstream communities, and help in emergency management decisions such as evacuations, closing of roads, schools, churches, etc. This paper demonstrates how the result of this work is being used, and can be used, to monitor and manage dam risk. Monitoring and mitigating against dam breach risk will make communities more resilient.

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