e-mosty

ISSUE 02/2020 June

THE ROSE FITZGERALD KENNEDY BRIDGE

OVERVIEW DESIGN CONSTRUCTION TRAVELLERS PRESTRESSING

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Photos Front and Back Cover: The Rose Fitzgerald Kennedy Bridge, Ireland Photo Credit: BAM PPP Ireland

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Dear Readers

*This special issue is dedicated to the PPP Project of the N25 New Ross Bypass, **with a focus on The Rose Fitzgerald Kennedy Bridge** in Ireland. The bridge is the longest in Ireland and also the longest extrados concrete bridge in the world.*

The first article provides an introduction to the project, its history, alternative road networks, evaluation methodology, preferred routes, bridge options, architectural considerations and PPP tender process. The article was prepared by The Authority which is Transport Infrastructure Ireland and by Mott MacDonald Ireland.

In the next article, Ronald Yee focuses on the Architectural Design of the Bridge. Yee Associates have been involved since 2001 during the first phase route selection when they evaluated the aesthetic implications and architectural opportunities of each route option with subsequent evaluation of the Bridge options in respect of their architectural design merit. The article is accompanied with Ron Yee´s own illustrative sketches.

The detailed design of the Bridge, design principles and the solution adopted are described in an article prepared by Arup and CFCSL. The article also includes drawings of the Bridge.

It is followed by an article about the Design and Construction by BAM Dragados. The article describes the Construction Sequence, Temporary Works design, Deck closure, Cable stay system, Bridge lighting and the operation and maintenance phase.

Wing and Formwork Travellers are described in the subsequent article which has been prepared by Rúbrica Engineering.

The detailed description of cable stay system and prestressing technologies is provided in the last technical article of this special issue prepared by Tensa.

*I would like to thank all authors and the companies involved for their cooperation, **Larry Mackey, Tim Abbott and Mary Bowe for their assistance, Richard Cooke for reviewing this issue, and Guillermo Muñoz-Cobo Cique (Arup) for his final check.***

*I would also like to **thank our partners for their continuous support**.*

My company has been affected by the current situation and as a result we have decided to extend the scope of services we provide. Our advertisement is on page 69. We hope that both our magazines (e-mosty and e-maritime) provide references of what we can do. We are happy to offer all our experience and knowledge and look forward to our possible cooperation.

*In September I will go on offering partnership with e-mosty and e-maritime magazines. Our **partnership offer** is on page 06. If you are interested in cooperating with us as our partner, please contact us. General information on partnership with our magazines can be found on e-mosty or e-maritime.*

We are now preparing a special issue of e-maritime magazine which will be dedicated to the Monaco Land Project. It will be released on 30th June on www.e-maritime.cz with open access. You can also subscribe.

Next e-mosty magazine will be released on 20th September, it will focus on Vessels and Equipment for Bridge Construction.

Magdaléna Sobotková

Chief Editor

e-mosty

The magazine e-mosty ("e-bridges") is an international, interactive,
peer-reviewed magazine about bridges.

It is published on www.e-mosty.cz and can be read free of charge
(open access) with possibility to subscribe.

It is published quarterly: 20 March, 20 June, 20 September and 20 December.
The magazines stay **available on-line** on our website as pdf.

The magazine brings original articles about bridges and bridge engineers
from around the world. Its electronic form enables publishing
of high-quality photos, videos, drawings, links etc.

We aim to include **all important and technical information**
and show the grace and beauty of the structures.

We are happy to provide media support for important bridge
conferences, educational activities, charitable projects, books etc.

Our Editorial Board comprises bridge engineers and experts
from the UK, US and Australia.

The readers are mainly bridge engineers, designers,
constructors and managers of construction companies,
university lecturers and students, or people who just love bridges.

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e-maritime

The magazine **e-maritime is an international, interactive, peer-reviewed magazine about ports, docks, vessels, and maritime equipment**.

It is published on www.e-maritime.cz **three times a year:**
30 March, 30 June and 30 November.

September Issue is shared with the magazine e-mosty ("e-bridges"):
"Bridges, Vessels and Maritime Equipment"
which is published on 20 September on www.e-mosty.cz.

It can be read **free of charge** (open access) with possibility to subscribe.
The magazines stay **available on-line** on our website as pdf.

The magazine brings **original articles about design, construction, operation and maintenance of ports, docks, vessels, and maritime equipment from around the world**.

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We aim to include all important and technical information
and show the grace and beauty of the vessels and structures as well.

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e-mosty

Bridge Design, Construction, Maintenance

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The magazine e-mosty was established in April 2015 and its first issue was released on 20 June 2015 as a bilingual English – Czech magazine aimed mainly for Czech and Slovak bridge engineers.

Very quickly it reached an international readership.

In 2016 we extended the already existing Czech and Slovak Editorial Board by two bridge experts from the UK, and since then four more colleagues – from the USA, Australia and The Netherlands – have joined us.

Since December 2016 the magazine has been published solely in English.

Each issue now has thousands of readers worldwide.

Many of our readers share the magazine in their companies and among their colleagues so the final number of readers is much higher.

Most importantly the readership covers our target segment – managers in construction companies, bridge designers and engineers, universities and other bridge related experts.

The magazine e-maritime was established in 2018 and its first issue was released on 30 March 2019.

The magazine is published in English. It is going to cover a vast range of topics related to vessels, maritime equipment, ports, docks, piers and jetties - their design, construction, operation and maintenance, and various maritime and construction related projects.

The Editorial Board already has two members – from the UK and the Netherlands.

Both magazines are with Open Access with possibility to subscribe (free of charge).

In January 2019 we established their own pages on LinkedIn with constantly increasing number of their followers. Number of subscribers of both magazines is also increasing.

We also know that the readers usually go back to older issues of both magazines.

ROSE FITZGERALD KENNEDY BRIDGE
NEW ROSS, IRELAND

Mary Bowe, Authority's Representative, Transport Infrastructure Ireland

John Murphy, Project Director, Mott MacDonald Ireland

Joe Shinkwin, Commission Manager, Mott MacDonald Ireland

INTRODUCTION

This project has been promoted and directed by Transport Infrastructure Ireland - TII (formerly National Roads Authority) and the Authority's Representative is Mary Bowe, Chartered Engineer.

The need for a second river crossing providing a bypass of the town of New Ross has been recognised for many years in studies and development plans.

The original N25 and N30 routes at New Ross passed right through the town, crossing the River Barrow over O'Hanrahan Bridge and travelling along the quays. These are key commercial and tourist routes and O'Hanrahan Bridge was the only crossing point at New Ross – its unavailability for any reason would necessitate significant detours on minor roads.

Delays on both the N25 and N30 routes were common – including queues of several kilometres and delays of up to half an hour at peak times.

CONSTRAINTS STUDY

In March 1999, Mott MacDonald were appointed by Wexford County Council to determine the need for and location of a Second River Crossing & Bypass of New Ross.

A Constraints Study was published in February 2001 which identified the following main considerations:

- The River Barrow was used in connection with both commercial shipping and recreational/ pleasure craft.

  The Port of New Ross had stated that a second river crossing should not interfere with the navigation of the River Barrow.
- Candidate Special Areas of Conservation, Proposed National Heritage Areas and rare plant species were identified in the study area.
- A large number of archaeological sites had been identified in the study area with a potential for further sites of archaeological or historic value on detailed inspection.
- The landscape rises on either side of the river with hills and tributary valleys forming an undulating countryside.

  Visual constraints identified include the scenic river valley, ridgelines, steep hill sides and areas of woodland. The Wexford Development Plan identified views from N30 and N25 to be preserved or improved.

ROUTE SELECTION

After the constraints had been identified, the first phase of the Route Selection was undertaken.

A variety of alternative corridors were selected for consideration having regard to traffic performance, road network connectivity, topography, alignment design, constraints identified in the Constraints Study and feedback from Public Consultation.

A total of 46 alternative road networks comprising various combinations of twelve different route corridors, (Routes A to L) were tested and evaluated. These are illustrated in Figure 1.

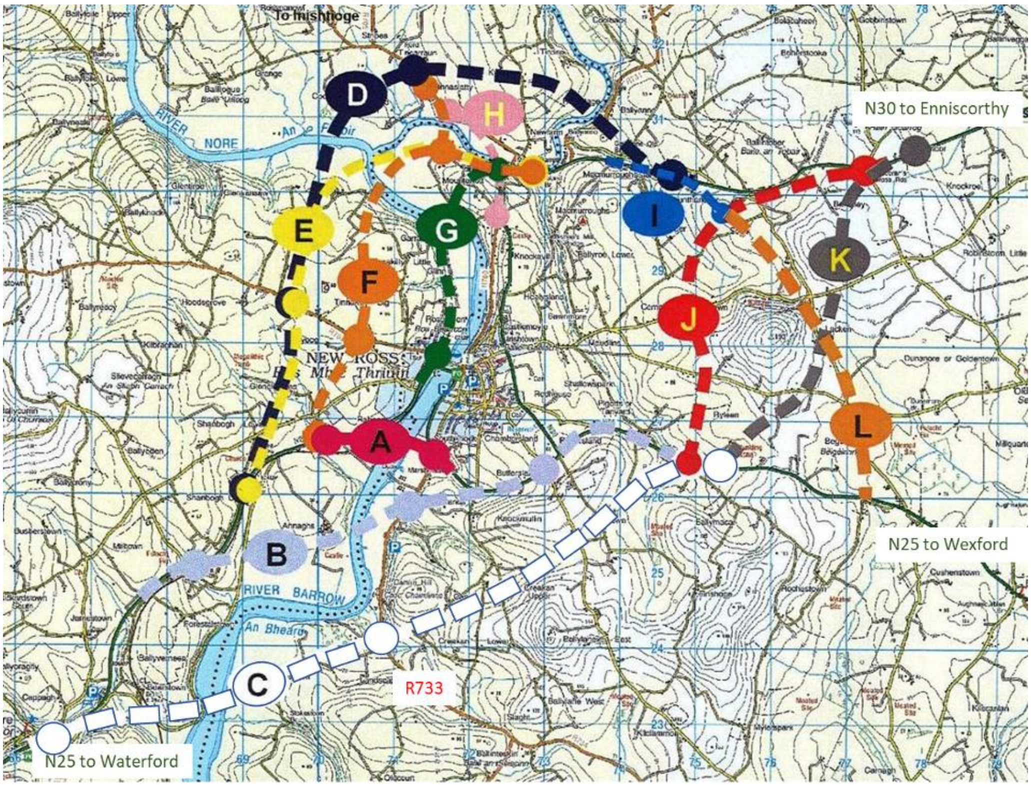


Figure 1: Combinations of 12 corridors used to establish 46 Alternative Routes

Evaluation Methodology

The 46 scheme options were assessed in terms of their environmental impacts and also having regard to their performance in terms of traffic, effectiveness as a bypass, and economic performance.

A variety of crossing types were also considered as appropriate at each river crossing location including high level bridges, fixed medium or low level bridges, opening span medium or low level bridges, and tunnels.

Environmental evaluation included ecology, water quality and fisheries, archaeology, landscape, geology and hydrogeology, recreation and amenity.

Traffic modelling and forecasting was also undertaken for all of the routes and initial cost estimates prepared for evaluation.

Arising from that evaluation, three crossing locations/routes (A, C and D) were shortlisted for more detailed consideration, culminating in a direct comparison between routes A and C and the selection of Route C as the preferred route.

In addition to performing better on Environmental comparisons, a key feature of the selected route was a journey saving of 3km for every N25 to N25 trip and 5.7km for every N25 to N30 trip.

All routes required a significant crossing of the River Barrow – the journey savings generated the benefits to justify the cost. The Route Selection Report outlining the preferred route was published in October 2002 – see Figure 2.

PREFERRED ROUTE

Road Type

An analysis of Traffic projections was undertaken to determine the road cross-section resulting in the following recommended cross-sections:

Glenmore to R733 (excluding new bridge)	Type 1 Dual Carriageway
River Barrow Bridge	Reduced Type 1 Dual Carriageway
N25 Bypass (R733 to Ballymacar Br.)	Type 2 Dual Carriageway
N30 Bypass (Ballymacar Br. to Corcoran's Cross)	Type 2 Dual Carriageway

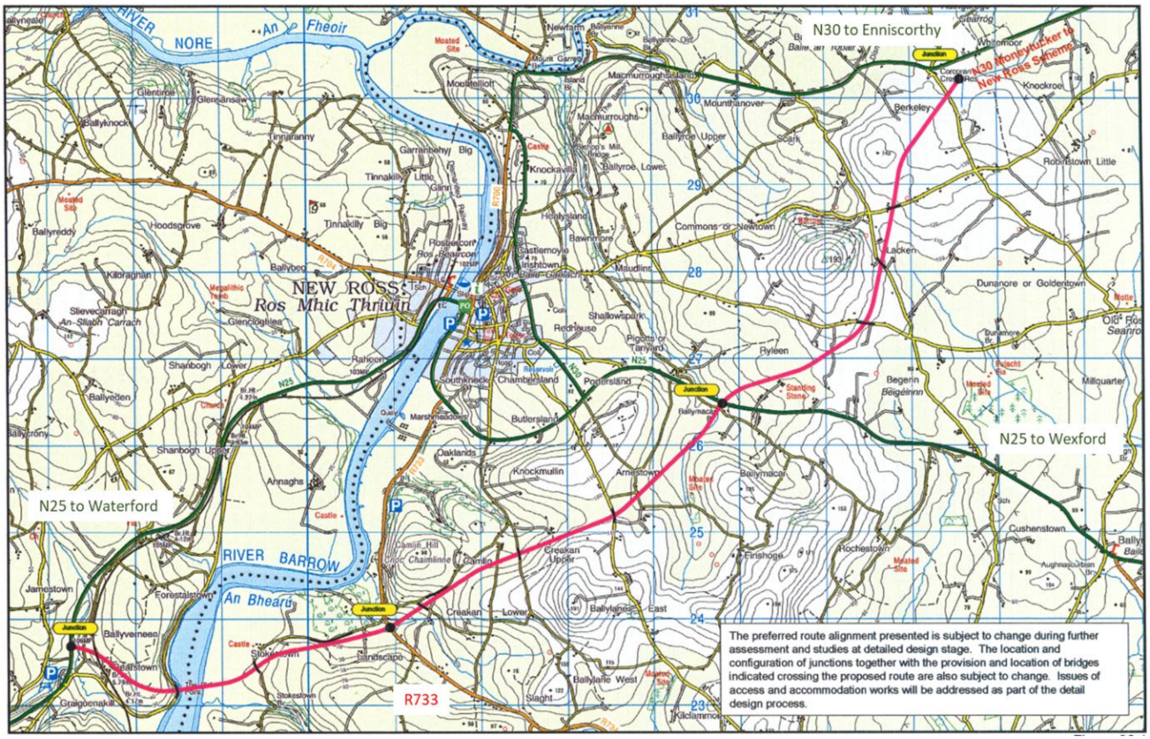


Figure 2: Preferred Route Corridor

RIVER BARROW CROSSING

Port of New Ross

New Ross port, sited some 32km from the sea, is managed by the New Ross Port Company. A capital dredging programme in 1999 deepened the approach channel allowing the port to accommodate vessels of 6000 dead weight tonnage (DWT).

Port facilities are located on both sides of the River Barrow at New Ross and the principal commodities handled include oil, fertiliser, animal feedstuffs, coal and mineral ores.

During consultation at Route Selection stage the Port indicated that a vertical clearance of 36m above mean high water spring (MHWS) tide of the navigation channel would be desirable for a non-opening high level crossing.

The location of the crossing point at Pink Rock (See Figure 3) lent itself to the provision of a high level bridge as the elevation of the land on the immediate west side of the river was greater than 36m above the river level meaning that the bridge did not have to rise from water level to a height of 36m and back down again – significantly reducing the length of bridge required compared to other locations.

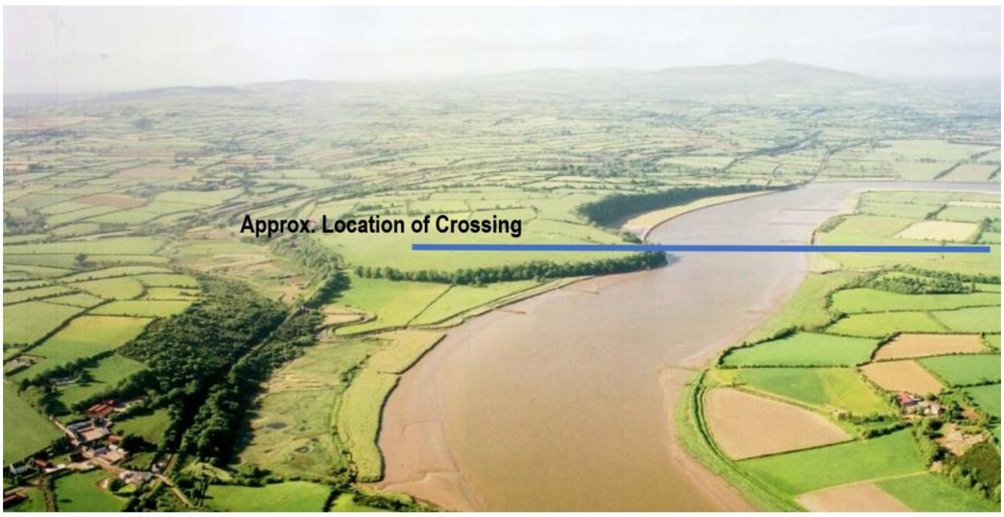


Figure 3: View of River Barrow and Pink Rock looking north

Barrow Crossing Alternative Types

Bridge Options Report

Following the publication of the Route Selection report in 2002, a detailed examination of the bridge options for the river crossing was undertaken.

Initially, 8 options were reviewed which included:

i) Haunched Box Girder Bridge
ii) Cable Stay Bridge – Vertical Tower
iii) Cable Stay Bridge – Inclined Tower
iv) Arch – Single Span
v) Arch – Single Span with "V" Piers
vi) Arch – 3 Span
vii) Extrados Bridge – 2 Pylon
viii) Extrados Bridge – 3 Pylon

Following the initial review, four of the options were selected to undergo a more detailed analysis with a view to recommending a preferred option.

These were:

- Haunched Box Girder Bridge
- Arch – Single Span with "V" Piers
- Arch – 3 Span
- Extrados Bridge – 3 Pylon

A bridge options report was prepared which developed the preliminary design of these four options to a level sufficient to determine comparative costs of each option taking account of constructability, programming and whole life issues.

GEOMETRY

Horizontal and Vertical Highway Alignment

The horizontal and vertical highway alignments adopted for the four options considered were based on those determined for the scheme at the route selection stage.

In the case of the vertical alignment, a 36m navigational clearance option was adopted for the purposes of the bridge options study.

The structure is essentially straight (except towards the abutments) and is orientated in an east-west direction.

The road alignment is approximately orthogonal to the direction of the River Barrow. The road cross-section adopted comprises two 7.0m carriageways flanked by 0.5m nearside and 1.0m offside hard strips, with a min. 0.6m raised kerb along each edge of the structure.

As a minimum, a 0.6m wide high containment barrier is provided between the two carriageways, but to accommodate the structural form of the structures being considered, the extent of the division between the two carriageways varies as required. No footways are provided across the structure.

Navigation Clearances

Data relating to riverbed profiles were plotted to determine the width of the required navigation channel beneath the structure.

Taking this profile and its intersection with the -3m Chart Datum contours (i.e. where the depth of water at MHWS is 7.5m or greater - as specified by the Port of New Ross), resulted in a design channel width of 117m.

The soffit lines of the respective bridge options were developed to provide a 36m clearance above high tide (Mean High Water Spring) over the width of the navigation channel as required by the Port of New Ross.

Based on the above criteria, the preliminary vertical alignment determined for the preferred route was found to be adequate to accommodate the construction depth of the deck for all options except the haunched box girder, where a higher road alignment was found to be necessary.

It was envisaged that during construction there would likely be times when the navigation channel would be obstructed to a greater or lesser degree by either temporary works (e.g. formwork, scaffolding etc.) or construction plant (e.g. barges, craneage etc.).

Whilst consideration of the constructability of each option took such issues into account (with a view to minimising disruption), it was generally envisaged that periodic short-term "possessions" of the waterway would need to be agreed between the contractor and the harbour authority during the construction phase.

Air Clearance

No power lines cross the site of the proposed bridge structure. There are also no airfields in the immediate vicinity of the structure, and the structures are not significantly higher than the surrounding topography.

On this basis, it was assumed that there were no restrictions on air clearance at the bridge site.

ARCHITECTURAL CONSIDERATIONS

Design Philosophy

The architectural design strategy adopted for the four bridge options was based upon a holistic approach to the aesthetic and functional requirements for the crossing.

The proposed bridge designs were considered to provide a balance between the different functional and pragmatic engineering requirements, whilst being sensitive to their setting.

Setting

The proposed bridge crosses the River Barrow downstream of New Ross in the vicinity of Pink Rock, a notable local natural feature.

The Barrow valley at this point has two distinct characteristics; the eastern side of the valley comprises a flood plain with grazing pasture divided into fields by hedgerows whilst the western side is wooded and steeply sloping.

Flowing southward, the river is wide and gently curved, narrowing down to approximately 280m at the crossing point.

To the north of the bridge site the western bank of the river has been realigned leaving an open area of reclaimed land from which the new crossing can be viewed.

The proposed location for the bridge offers an excellent setting in which to place a striking structure.

The bridge “leaps” from the Pink Rock on the western side and crosses the river to slope down across the flood plain towards the gently rising hills at Stokestown on the eastern side.

The bridge deck would be of the order of 40 metres above the river and would afford potentially spectacular views of the surrounding landscape and the River Barrow.

From ground level, the box girder option provides a relatively unobtrusive solution, visible from within the immediate landscape only.

By contrast, the additional height associated with the arched and extradosed options results in an impact over a much wider geographical area.

A number of Artist’s sketches were prepared to assist with evaluation of the alternative structures – principally from a visual and aesthetic viewpoint. The sketches are illustrated in Figures 4 to 7.

Haunched Girder Bridge Option

Figure 4: Artist’s Sketch of Haunched Girder Bridge Option

In contrast to the arched and extradosed options, which make a visual statement on the surrounding landscape, the box girder option focuses on functional simplicity, with a modestly elegant design complimenting the boldness of its scale.

The sides of the single box girder are inclined to reduce their visual mass which, together with long edge cantilevers which put the structure in shade, further reduces the apparent visual depth of the structure when viewed from a distance.

Three Arch Bridge

Figure 5: Artist’s Sketch of Three Arch Bridge Option

Three Arch Bridge option comprises three arches decreasing in size in line with the elevation of the deck.

The arch members are positioned along the centreline of the bridge with pairs of hangers arranged vertically at regular intervals supporting the deck.

The central positioning of the arches adds to the visual drama of crossing the valley whilst still allowing unimpeded views off the bridge by road users (subject to open parapets being provided, with minimal wind shielding requirements).

In elevation the line of the structure can be likened to the path of a stone skimming across the surface of the water.

Single Arch Bridge

Figure 6: Artist's Sketch of Single Arch Bridge Option

This option has a bold single arch composed over the river linked by a high level viaduct forming the eastern approach.

As for the three-arch option, the arch is positioned along the centreline of the bridge, adding to the visual drama of crossing the valley whilst still allowing unimpeded views off the bridge.

The position of the arch also clearly defines the act of crossing the river itself.

Three Tower Extrados Bridge

Figure 7: Artist's Sketch of Extrados Bridge Option

This option is a bold contemporary design statement expressing intent for the future.

The bridge has a tripartite composition with a larger central element forming the visual focus of the scheme.

To emphasise the slenderness of this form of bridge the leading edge of the deck is made as thin as possible by utilising a trapezoidal cross section with haunches growing out of the soffit where additional structural depth is required for the longer spans.

COSTINGS

Cost estimates were prepared for the various bridge options under consideration. Measured approximate quantities were prepared for each option and priced on a common basis using experience of projects of a similar nature.

Account was taken of the likely unique temporary works requirements, programme, construction techniques, special plant and significant material imports, and implications of undertaking the works at the particular location.

RECOMMENDED BRIDGE

All four options were confirmed to be appropriate for the crossing in terms of functionality and architectural impact although it is recognised that the latter is a subjective conclusion.

All four bridge options studied were expected to have only a modest environmental impact with visual impact possibly being the more contentious impact due to the subjective nature of the impact.

Critically, all 4 options minimised impact on the qualifying interests of the SAC (Special Areas of Conservation).

The construction of all four options was found to be feasible within the then known constraints of the site.

Navigational arrangements would require positive agreement with the appropriate authorities as this dictated both the vertical profile of the bridge and the length of the main river span.

The construction periods for the arch options were expected to be approximately 30% longer than either the box girder or extradosed options which would require approximately 30 months to construct.

Having regard to all of the considerations, the Extrados option was selected on the basis that it had a relatively small increase in cost over the Box Girder bridge but it was considered to be significantly better from an aesthetic viewpoint and had a lower alignment with a slightly shorter overall length.

SITE INVESTIGATIONS

A number of site investigations were carried out to inform the design process for the New Ross Bypass project including various land-based ground investigation comprising cable percussion and rotary drilled boreholes, trials pits, exposure logging, in-situ testing and laboratory testing.

An over water ground investigation was also carried out at the site of the proposed River Barrow crossing, which comprised cable percussion and rotary drilled boreholes, in-situ testing including static Cone Penetration Testing and laboratory testing.

The field work was carried out between September 2006 and January 2007.

CPO AND EIS ORAL HEARING

Wexford County Council applied to An Bord Pleanála for approval of the Environmental Impact Assessment (EIS) and confirmation of the Compulsory Purchase Order (CPO) in November 2007. An Oral Hearing into the CPO and the EIS was held by An Bord Pleanála in April 2008.

At that time of the oral hearing it was envisaged that the N25 New Ross Bypass scheme would be procured by means of Public Private Partnership (PPP).

As a result, some latitude would need to be given to the bidding consortia in relation to detail design, however, it was also desirable to enshrine the essentials of the bridge within the Scheme Orders to ensure that the chosen structure would be delivered.

For the purposes of the Scheme Orders, the essential features of the recommended scheme were described as follows at the Oral Hearing and these were enshrined in the Schedule of Commitments:

- The main tower will be of the order of 25 metres in height above deck level.
- There shall be a single line of towers (and cables) in the centre of the structure.
- The two adjacent towers shall be of the order of 15 metres in height above deck level
- The two main spans will be of the order of 230 metres.
- The approach spans on the western side shall be generally of the order of 45m, 60m and 86m (starting from the west).
- The approach spans on the eastern side shall be generally of the order of 45m, 60m, 60m and 86m (starting from the east).
- The cables shall be arranged in parallel to one another.
- The towers shall flare towards the top with curves oriented to compliment the inclination of the support cables.
- A design navigation clearance of 36metres above high tide (Mean High Water Spring Tide) shall be provided over the width of the navigation channel (defined by the Port of New Ross as the width of water with an available draught of 7.5m or more at Mean High Water Spring).
- Architectural lighting shall be incorporated to delineate the bridge at night and shall be designed so as not to conflict with navigational lighting.

In December 2008, An Bord Pleanála issued an order granting approval to the Scheme and confirming the CPO Order.

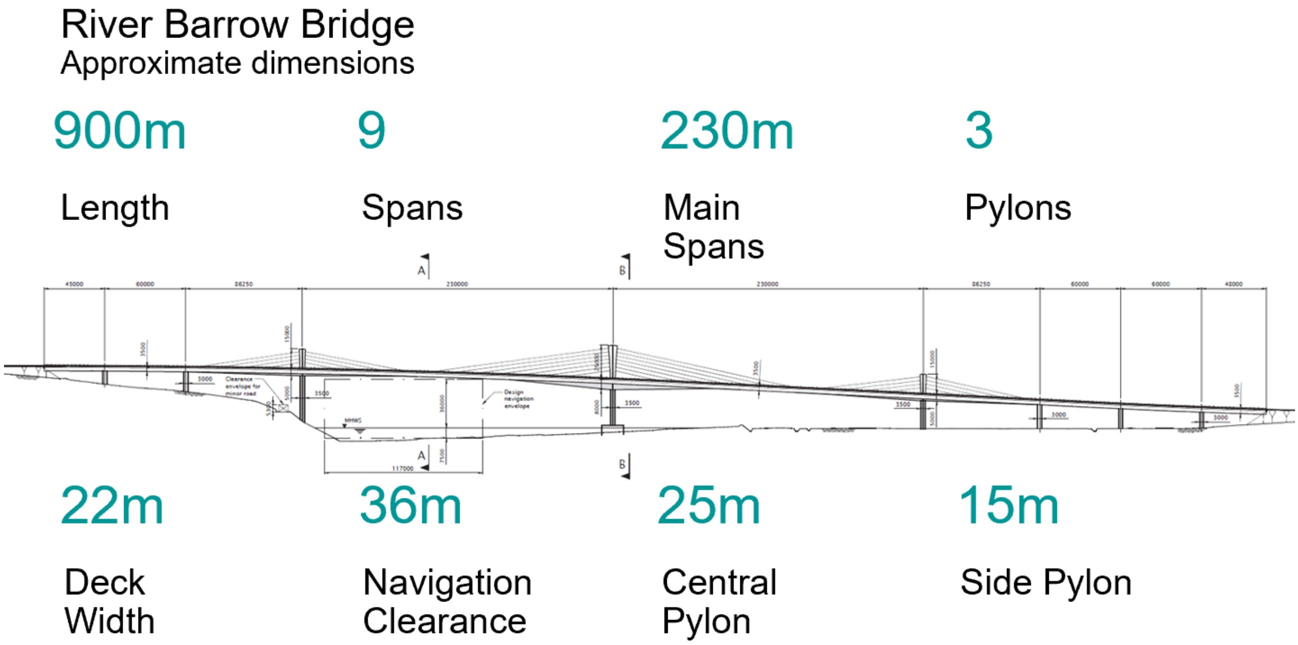


Figure 8: River Barrow Bridge – key features

SECOND PPP ROADS PROGRAMME

In June 2009 the National Roads Authority (NRA, now Transport Infrastructure Ireland - TII) released a Second PPP roads programme.

The N25 New Ross Bypass was included and at the time it was envisaged that the N25 New Ross Bypass would be joined with the N11 Gorey to Enniscorthy.

However, in early 2013, it was decided to progress the schemes as two separate PPP projects and Mott MacDonald Ireland were appointed to progress the N25 New Ross Bypass Scheme through Advance Works, Construction Documents Preparation, Tender & Award, Construction & Implementation, Handover, and Closeout.

PPP TENDER PROCESS

The tender process for the PPP scheme was originally begun in March 2010 but was suspended later in 2010.

In July 2012 the Government announced an Infrastructure Stimulus Package to provide investment for a range of important public infrastructure projects including the N25 PPP Scheme.

The Tender process for the N25 New Ross Bypass PPP Scheme then re-commenced with the publication of a notice in the OJEU on March 22nd 2013.

Four consortia were prequalified for the tender. They were:

- BAM Iridium
- Banba Consortium
- Sli Nua
- Direct Route

The Invitation to Negotiate including tender documents was issued in November 2013 and following a number of tender consultation meetings tenders were returned in September 2014.

Following evaluation of the tenders, both technical and financial, a contract was awarded to BAM Iridium and the contract was signed on 26th January 2016.

CONSTRUCTION - DESIGN REVIEW PROCESS

The PPP Agreement between the *Authority* (National Roads Authority) and the *PPPCo* (BAM Iridium) comprises the NRA PPP Contract and NRA PPP Contract Schedules. The Contract Schedules include Tender Proposals/ Conceptual Design (Sch 28), Quality and Environmental Management (Sch 10), Certification Procedure (Sch 5), Land Issues Roads and Orders (Sch 2), Third Party, Construction and O&M Requirements (Sch 3, 4 & 7).

Obligations of the PPPCo include the design, construction, operation, maintenance and financing of the Works.

Responsibility for the design, construction, supervision and commissioning of the works lies with the New Ross Joint Venture (NRJV) comprising a joint venture between BAM Civil and Dragados SA.

Design and construction of the works was required to be undertaken in accordance with the Construction Requirements (Sch 4), the Conceptual Design (Sch 28) and the Certification Procedure (Sch 5).

The Certification Procedure provides for the submission of Quality Documentation, Design (Highway, Structures, Earthworks, etc), Departures from Standard, Archaeology, Ecology, Alternative Conceptual Designs, Third Party Consultation, Road Safety Audits and Temporary Works Design under an appropriate certificate by the PPPCo to the Authority. The review of the documentation is a function delegated by the Authority's Representative (AR) to Mott MacDonald Ireland (MMI) and the Authority's Site Representative (ASR).

Documentation received from the PPPCo under certificate, in accordance with the provisions of the Certification Process (Sch 5) was examined for compliance with the Agreement including examination for:

- Consistency with the Conceptual Design included in Schedule 28
- Compliance with the Certification Procedure
- Consistency with the EIS and compliance with the Orders
- Compliance with the Construction Requirements
- Compliance with other provision of the Agreement.

The design/documentation reviewed by Mott MacDonald included the following:

- Quality Documentation (Design, Construction, O&M) including Method Statements
- Alternative Conceptual Designs
- Review of Departure and Variation applications
- Design Review including Site Clearance, Fencing, Ecology, Road Layout, Structures, Earthworks, Drainage, Utilities, Lighting, Kerbs, Pavement, Signs & Road Markings, Safety Fencing, Landscaping and Environmental Works, Accommodation Works
- Temporary Works and Third Party certification
- Traffic Management and Road Safety Audits

CONSTRUCTION

Construction commenced in early 2016 and the bypass, including the Rose Fitzgerald Kennedy Bridge, was officially opened on 29th January 2020.

The project opens exciting opportunities for the area supporting future growth and sustainability – a key transport link in the Southeast Region of Ireland.

It Improves access to vital services and employment centres in the region and significantly reduces traffic congestion, journey times and journey lengths.

It is estimated that the scheme will lead to a reduction of approximately 20 fatal collisions over a 30-year period.

It will also have the environmental benefit of a reduction of approximately 0.5 million tonnes of carbon.

The Rose Fitzgerald Kennedy Bridge is Ireland's longest Bridge at 887m long.

The innovative design incorporating an extrados bridge produced an aesthetic and landmark bridge over the River Barrow which ensured minimal impact on the SAC, ensured continued navigation clearance of 36m above high water for shipping, and produced a unique iconic structure which will be identified with and will represent the locality of New Ross, the region and the country.

Figure 9: View of the Rose Fitzgerald Kennedy Bridge

ARCHITECTURAL DESIGN

OF THE ROSE FITZGERALD KENNEDY BRIDGE

Ronald Yee, Yee Associates

INTRODUCTION

The extrados bridge typology, with external pre-stressed tendons, is a relatively recent development in concrete cable supported bridge technology.

The concept, first proposed by French engineer Jacques Mathivat in 1988, employs stay cables for strengthening rather than supporting the bridge deck, it is structurally much closer to a pre-stressed cantilever bridge than a cable stayed bridge.

Alluding to the term "extrados" which describes the upper surface of an arch barrel, Mathivat coined the term "extrados pre-stressing" for a unique system of arranging the pre-stressing tendons outside of the deck and maintaining high eccentricity using a short tower over the pier support.

Aesthetically the extrados bridge is characterised by a low tower height to main span ratio of between 1:8 and 1:15 (with around 1:10 being the most common) compared to a typical cable-stay tower height to main span ratio of 1:5 and results in a much flatter cable angle of typically 15 degrees from horizontal.

The form is ideally suited for spans between 100 - 250m and is architecturally useful where the overall height, overhead navigational clearance or aesthetic considerations have made a cable stayed alternative less viable.

The web database Structurae.net lists 79 extrados bridge structures built around the world however Ireland and the United Kingdom have yet to realise one.

N25 NEW ROSS BYPASS, IRELAND

Since Yee Associates were already acting as aesthetic advisors to the Mott MacDonald team on the N25 River Suir Crossing of the Waterford Bypass it seemed natural that we should continue to collaborate on New Ross Bypass.

Our involvement started in 2001 during the first phase route selection stage when we evaluated the aesthetic implications and architectural opportunities of each route option.

Preferred Route

Following public consultation a route bypassing the town to the south was chosen as it could also be extended to connect with the wider road network linking to the North East.

The route crosses the River Barrow downstream from New Ross in the vicinity of Pink Rock.

There the landscape on either side of the Barrow is quite different in character.

The eastern side of the valley is gently sloping farmland divided into fields by hedgerows, whilst the western side is densely wooded and steeply sloped.

Flowing southward the river is broad and gently curved but narrows down to approximately 280m at the crossing point.

The terrain being much higher to the west, results with a road/deck alignment that slopes down noticeably towards the east.

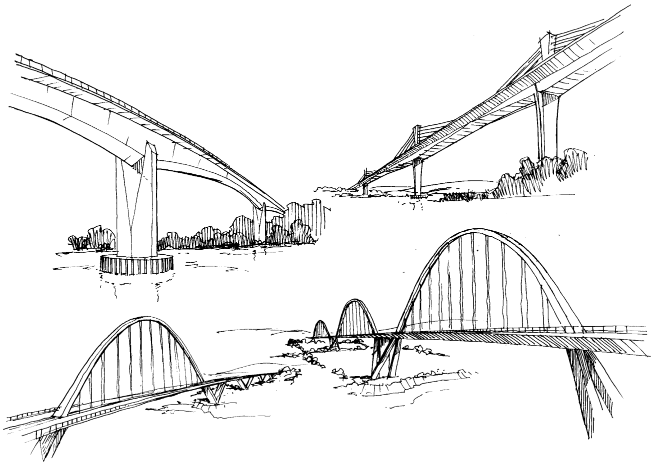


Figure 1: Four bridge options were developed for detail investigation. A. Haunched Box Girder; B. Three Arch Bridge; C. Single Arch Bridge; and D. Three Towered Extrados Bridge. Yee Associates.

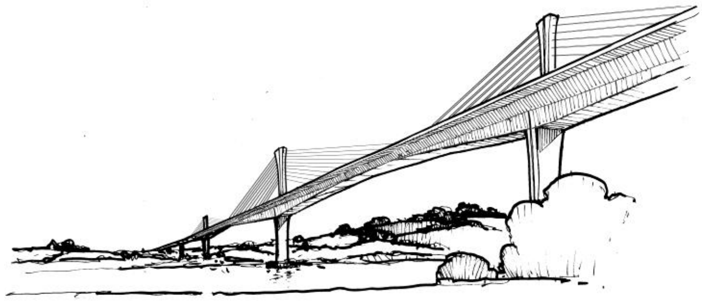


Figure 2: The 2005 Options Study Report recommended that the three towered extrados bridge should be developed in further detail. Yee Associates.

Bridge Options

During the initial stages a considerable number of bridge options for crossing the estuary were brainstormed, sketched and structurally explored.

These ranged from modest multi-span viaducts with long approach embankments, to extremely long span landmark structures at the limit of engineering technology.

After evaluation, four bridge options were identified as having sufficient merit for further investigation and development:

A. Haunched Girder Bridge;
B. Three Arch Bridge;
C. Single Arch Bridge; and
D. Three Tower Extrados Bridge,

with each option being approximately 900m long in total.

Preferred Bridge Option

On completion of the Options Study a report was submitted to the National Roads Authority (now TII - Transport Infrastructure Ireland) in 2005, recommending that the extrados bridge should be further developed in more detail, as it offered the best balance of overall performance, constructability and cost.

In addition the extrados bridge form was considered a bold contemporary design statement reflecting Ireland’s political intent for modernising its future.

Bridge Architecture

The bridge´s architectural design has a classical tripartite arrangement with a larger central element forming the visual focus of the composition.

To enhance the slender appearance of its form the leading edge of the deck is made as thin as possible by utilising a trapezoidal cross section, with tapered haunches emerging from the flat soffit where the deck needs to achieve extra structural depth for the longer spans.

Above the deck the cable support structure is arranged in a single plane along the centreline. This provides visual drama for users of the bridge whilst maintaining unimpeded panoramic views off the bridge.

The towers are shaped to enhance their appearance with subtly curved flares oriented to complement the inclination of the support cables.

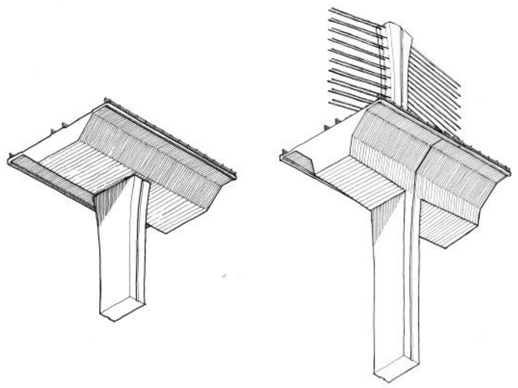


Figure 3: Isometric design sketch of the main extrados bridge elements: the approach span deck and pier; and the intermediate tower, haunched deck and pier. Yee Associates.

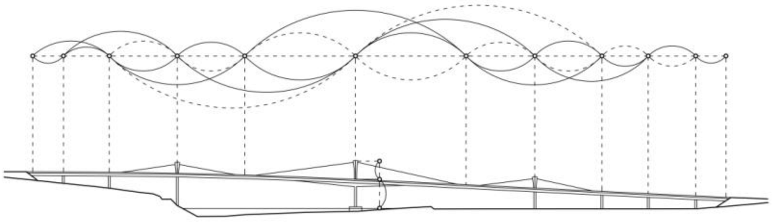


Figure 4: The Golden Proportions of the N25 River Barrow Crossing, now called the Rose Fitzgerald Kennedy Bridge. Drawn by G Baird.

Below the deck the shaping of the pier stems oriented transversely to the deck with curved tapers that visually flow into the sloped underside of the deck cantilevers.

Golden Proportions

The concept design was architecturally developed and structurally dimensioned using golden proportions to create a harmonious overall composition, with particular regard to the key views from up and down the estuary.

The bridge is 887m long and is divided into 9 spans: two main spans of 230m, two side spans of 86m and decreasing approach spans; three towers of 1 x 25m and 2 x 15m positioned on the centre line of the bridge deck; and a deck width of approximately 22m with a navigation clearance of 36m for shipping.

The overall scheme for the N25 New Ross Bypass was approved in 2008.

Lighting

To enhance the bridge´s slim lines, architectural feature lighting will illuminate the bridge during the hours of darkness.

Discrete computer controlled LED floodlighting will highlight the pylons and cable arrays from deck level whilst a continuous string of LED luminaires will delineate the bridge edge line.

For environmental reasons there will be no highway lighting on the bridge, but navigational beacons will guide shipping and provide warning to low flying aircraft.

CONSTRUCTION

Tendered through Public Private Partnership (PPP) initiative, the contract to finalise the design, build, finance, operate and maintain the N25 New Ross Bypass PPP scheme was awarded to the consortium BAM Iridium assisted by highway engineers Arup and specialist bridge engineers Carlos Fernandez Casado, with a timetabled commencement date of 26th January 2016, and a planned completion of mid-late 2019.

COMPLETION

After nearly 20 years of design and development the Rose Fitzgerald Kennedy Bridge was completed in January 2020 and was officially opened on 29th January 2020.

It is the longest bridge in Ireland and boasts the longest spans of their type in the world.

The following figures are Ron Yee´s own illustrative sketches of the bridge during construction, clearly showing many of the key stages and critical features of how the bridge was built.

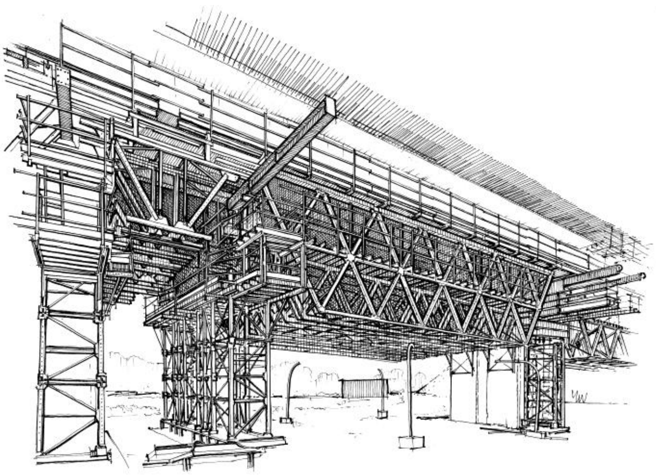


Figure 5: 16th July 2018. Installation of the gantry formwork for the first construction stage of the box girders of the eastern approach spans. The falsework is needed to support the bridge deck whilst the concrete is poured, once it has gained enough strength the falsework can be removed and moved on to the next section.

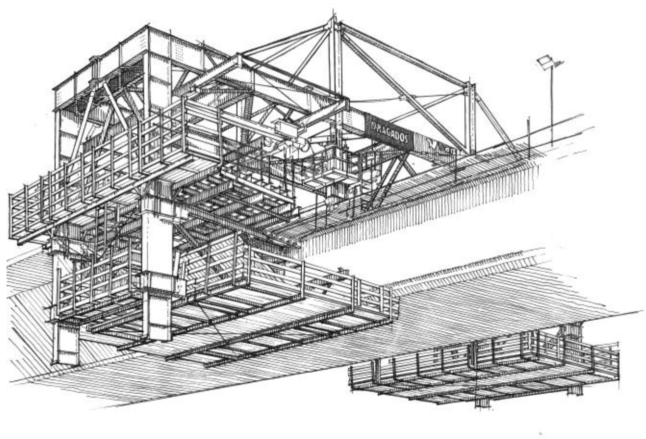
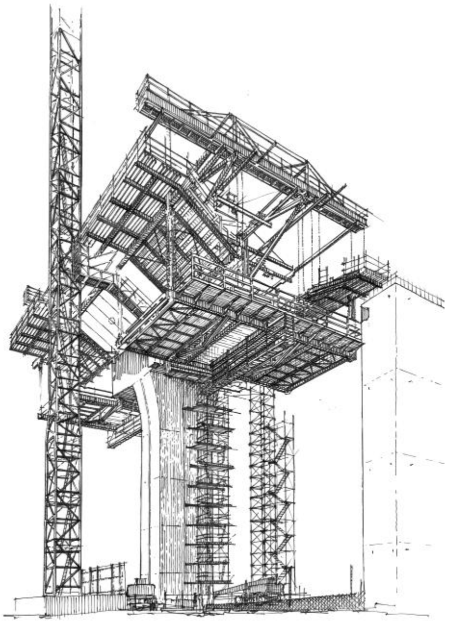


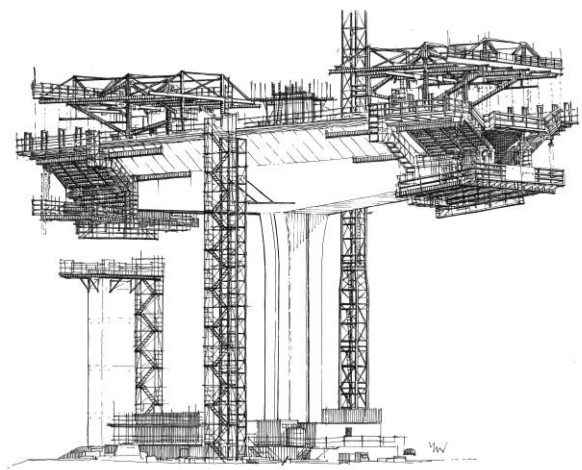
Figure 6: 16th July 2018. Progress sketch showing travelling formwork for the construction of the cantilever wings and sloping section of the deck on the western approach spans.

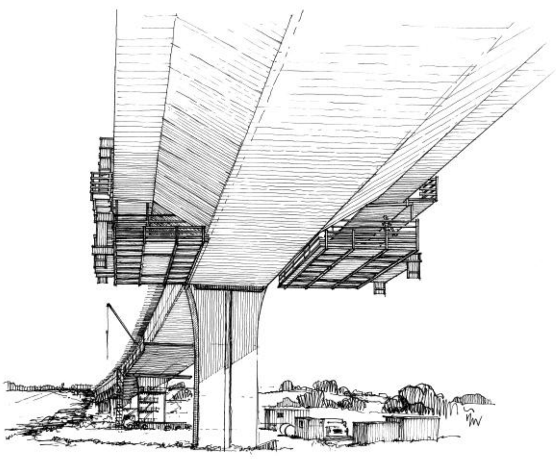


← Figure 7: 16th July 2018. Progress sketch of the main tower pier (P4) construction showing temporary pier on the right.
The splayed head of the main piers P3, P4 & P5 include part of the deck, and had to be poured in four sections.
Using a travelling formwork supported by steel support structure attached to previously completed segments, consecutive cantilevers are cast symmetrically either side of the pier stem so that the structure remains in balance.

↙ Figure 8: 15th August 2018. To stabilise and support the bridge deck during construction, temporary piers (TP) were constructed, seen on the left on the sketch. TP1 and TP2 were constructed to support the western and eastern approach spans respectively, whilst TP3 helped to stabilise the main spans until the stressing cables took the load.
Upon completion of the deck construction and the final stressing of the stay cables, the temporary piers became redundant and were removed since the bridge is now self-supporting.

↓ Figure 9: 15th August 2018. Casting the edge cantilevers on the Eastern approach near pier P6.





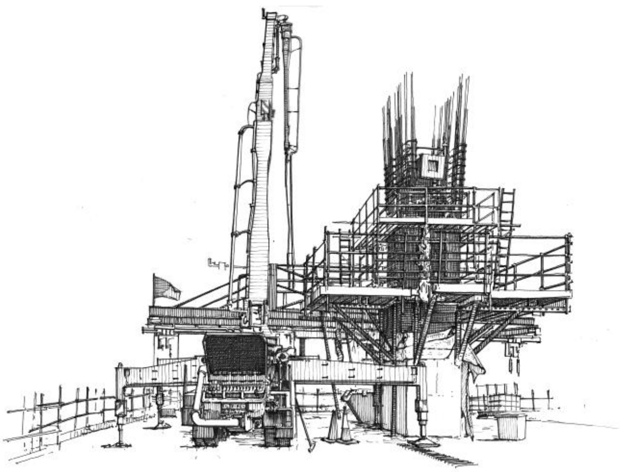
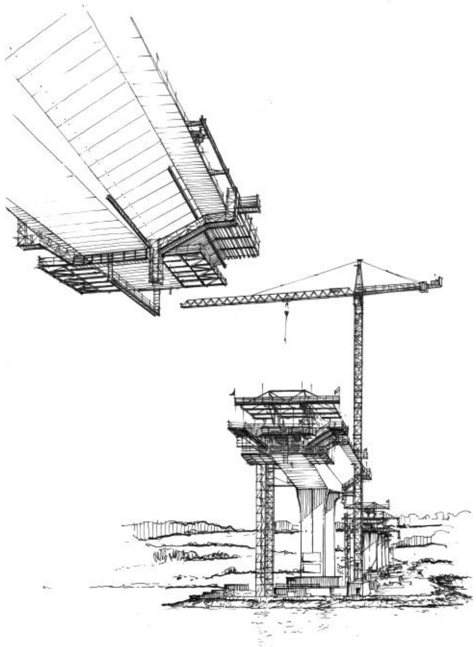


Figure 10: 11th September 2018. Construction of the main tower stem with the first cable anchorage installed ready for concreting in.

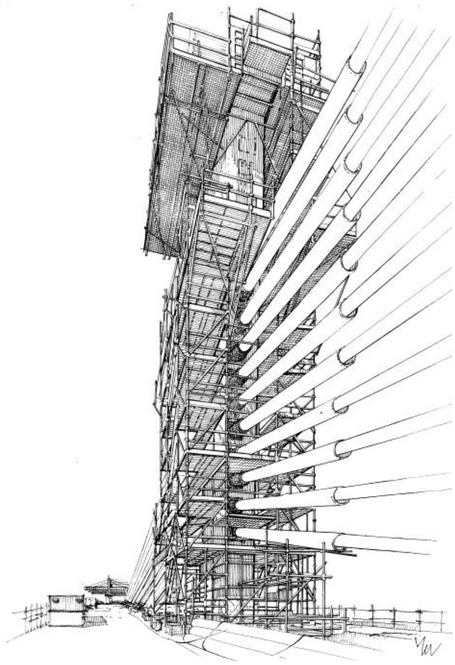
Figure 11: 23rd January 2019. View of the main bridge construction from beyond the Eastern Abutment.

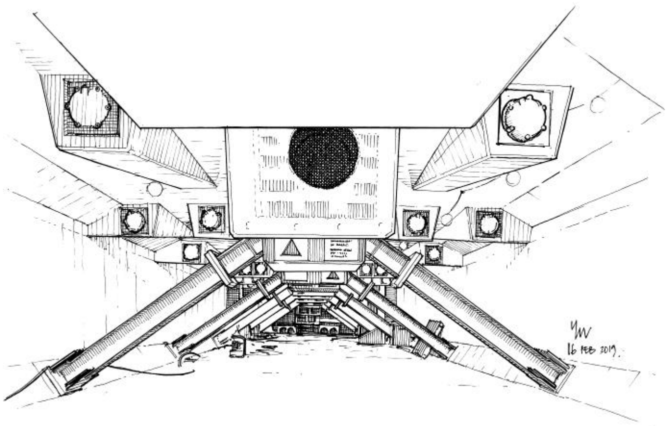


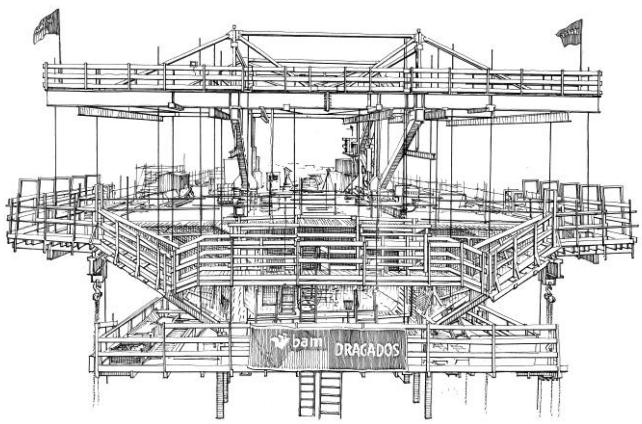
← Figure 12: 16th February 2019. Sketch showing the balanced cantilever construction of the main spans. Tower cranes are used to assist in the construction of the bridge. Each crane has a 50m jib capable of lifting 13tonnes at a 20m radius (normal working radius) 2.5 tonnes at a 50m radius and has a working height of 60m under the hook block.

→ Figure 13: 8th May 2019. Eight of the nine sections of the main tower stem have been concreted and eleven of the nineteen cables have been installed.

↘ Figure 14: 12th March 2019. A cold and misty morning sketch of the travelling form for the main river span looking from the West.







↑ Figure 15: 8th May 2019. Internal sketch of the hollow main span showing the sealed ends of the longitudinal pre-stressing tendons on either side. Here the structure is still awaiting the installation of the next main stay cable in the central socket. Constructed using metal formwork, the internal concrete finish is extremely smooth and is of a quality that a plasterer would be proud of! The deck is designed to allow internal access for maintenance and doors are provided within the abutments at each end.

DESIGN

OF THE ROSE FITZGERALD KENNEDY BRIDGE

Miguel Angel Astiz Suarez[1] – Marcos Sanchez Sanchez[2] – Lucia Blanco Martín[3]

INTRODUCTION

The design of the River Barrow crossing was developed in two stages as it is conventionally done in Irish PPP Motorway scheme.

During the Tender Stage, which took place in 2014, and in a competitive dialogue format, alternatives to the specimen design within the constraints set by the Tender documents were developed.

This process usually lasts around 20 weeks and the level of detail achieved in the design is limited.

The awarded team consisted of Dragados + BAM Ireland as contractors and ARUP and Carlos Fernandez Casado S.L. as Designers (ARUP being the designer for the whole scheme, including other structures and the rest of disciplines in the 12 km long scheme of N25-PPP New Ross bypass, and Carlos Fernandez Casado S.L. as sole designers during the Tender stage and in partnership with ARUP during the detailed design stage for the main bridge).

CONSTRAINTS

As part of the EIS and Construction Requirements, critical documents in the Irish planning and tendering process, the following constraints, amongst others, were established as fixed:

- The exact position of the three towers (thus fixing the main spans to 230m).
- The height of the pylons (forcing the bridge to be an extrados structure) and limiting the cable angle to less than 12 degrees.
- The clear envelope for the navigational channel (117m wide and 36m high over Mean High Water Spring).
- The requirement for a full concrete section for the deck and pylons (at least the outside surfaces) and the requirements of a “closed” section with inclined webs without props or ribs.
- The maximum depth at the central pylon of 8m and at midspan of 3.5m.
- The position of a central pylon and plane of cables in cross section.
- The maximum height of the abutments over ground level of 10m.

With all the above constraints, the number of variables to optimize the design was limited to the cable spacing, number and size, along with the cross section configuration for the main spans.

Also, there was room to tweak the road design both in plan and elevation on the approaches and also the configuration of the side spans.

VALUE ENGINEERING

With the constraints given above, the design was optimized as detailed design from the specimen design with the following changes:

- The cross section was modified from inclined outer webs (see Figures 1 and 2) to two vertical webs 8m apart, substituting the outer webs with precast panels to maintain the appearance of a closed section. The precast panels contribute in the transversal behaviour but there is a gap

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of 20mm between each panel longitudinally so they do not contribute in the longitudinal direction.

- The initial proposal of three parallel cables was substituted by a single cable, spaced 6.5m longitudinally and with a maximum size of 127 strands. Also, saddles were proposed for the cable detail passing on the pylons, looking for the minimum possible deck width. This allowed the pylon width to be reduced from 2.6m to 1.6m.

- In order to maintain a relatively light deck, the web and slab thickness were minimized by the use of high strength concrete where required. C80/95 concrete was used in the main spans and C60/75 in the side spans where the compression required this strength while the approach spans were designed as C50/60.

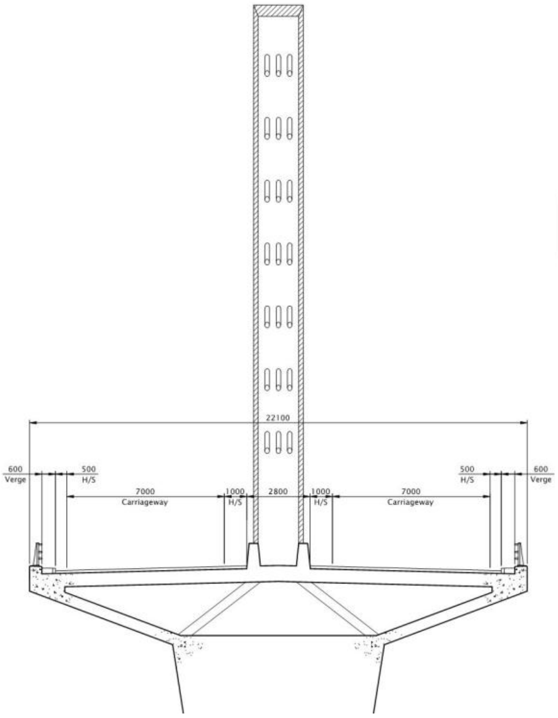


Figure 1: River Barrow bridge, cross section shown in the specimen design (pre-tender)

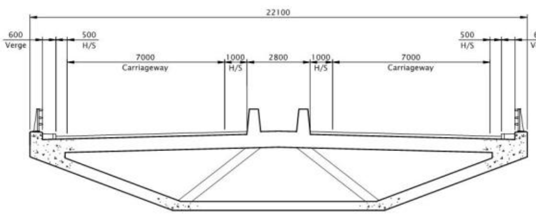
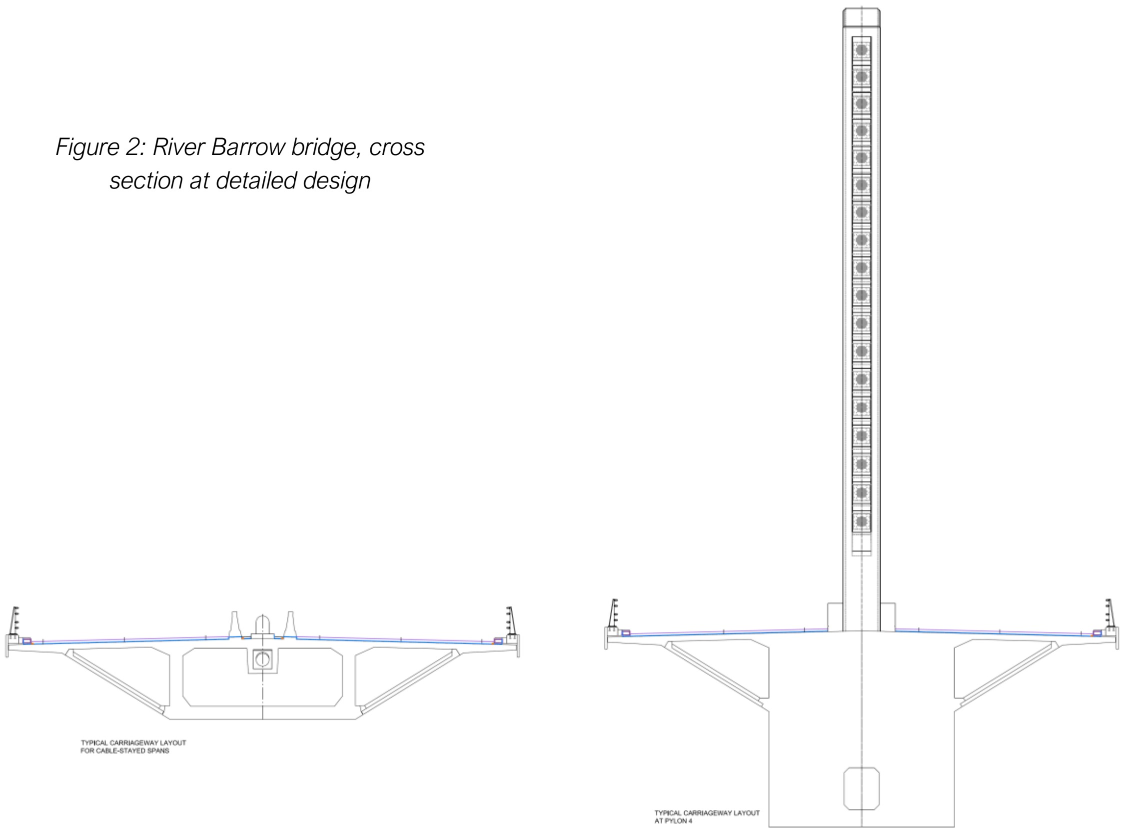


Figure 2: River Barrow bridge, cross section at detailed design



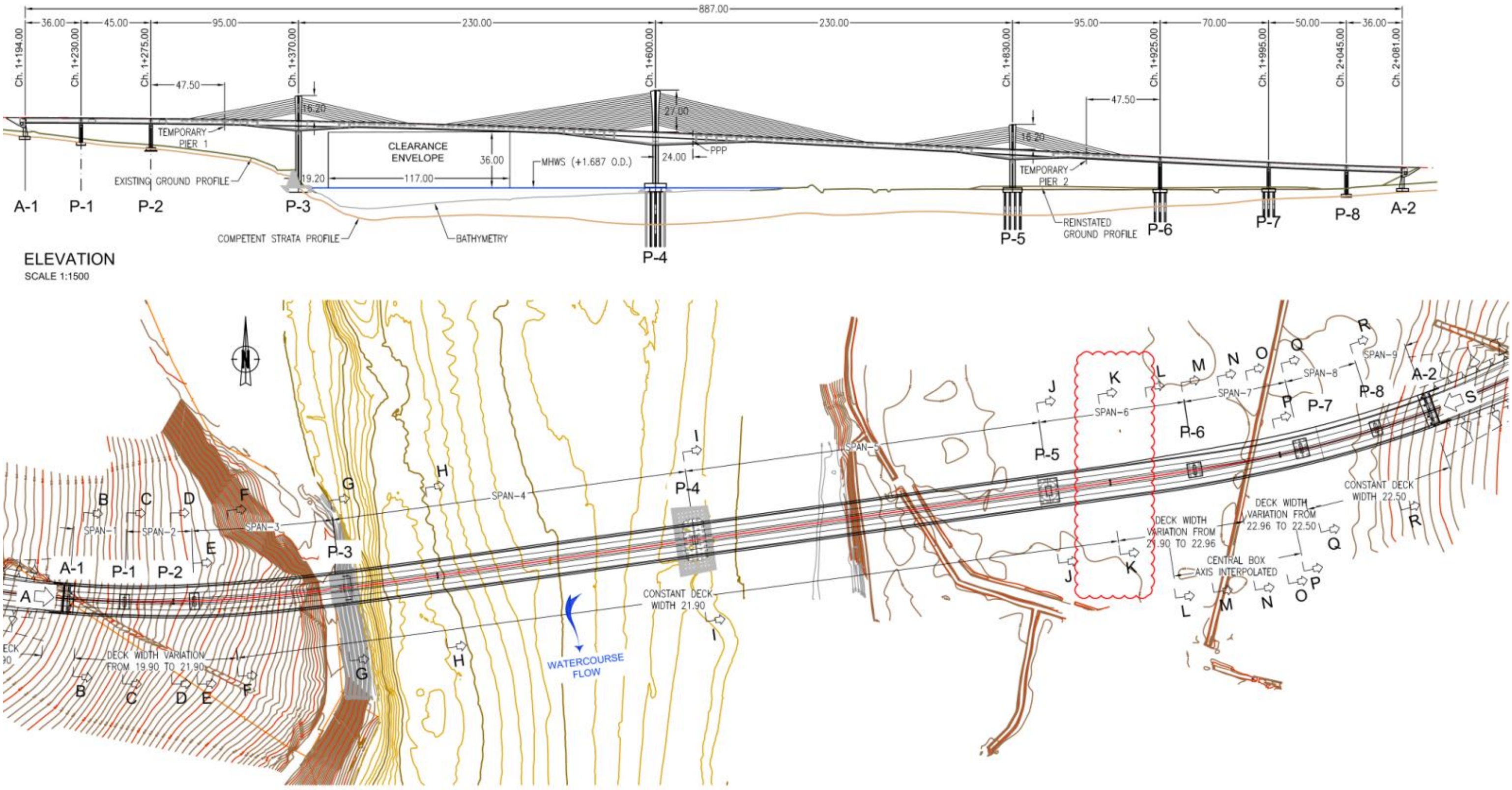


Figure 3: RFK Bridge, elevation and plan view at detailed design

(click on the image to see it in full)

Finally, minor adjustments to the side spans were implemented to optimize the longitudinal behaviour.

The road alignment was also modified to reduce the bridge width at both ends trying to achieve a constant width cross section where possible and reducing the bridge length from 905m to 887m by changes in the vertical alignment (see Figure 3).

DETAILED DESIGN. Main parameters.

The bridge final configuration, after the minor span changes during tender, resulted in a total length of 887m as already indicated, with an arrangement of 36 + 45 + 95 + 230 + 230 + 95 + 70 + 50 + 36m, as depicted in Figure 3.

In this way, the structure is characterized by 9 spans with 8 intermediate piers – P1 to P8 – and the 2 abutments – A1 and A2.

The plan alignment is straight along 440 m located approximately in the central part of the bridge and then curved with a transition from a radius of 720m to the straight alignment at both ends.

The height of the deck above the ground or over the river reaches 40m and the height of the towers above the deck is 27.0m for the central tower (P4) and 16.2m for the two lateral ones (P3 and P5).

These values imply tower height to span rations of 0.07L for the side towers and 0.117L for the central tower (with L being the central span length), which are low values, and lead to a classic extrados arrangement cable.

In addition the deck is only 3.5m deep at midspan L/65 and 8.5m at the central tower (L/27) and 6.5m at the side towers (L/35) which are quite slender parameters.

It is also important to highlight the implication of the different height in the towers, which leads to an asymmetric distribution of the cables along the main spans (8 from the side towers and 18 from the main tower), of approximately 145m from the central tower which would equate to a span of 2×145 = 290m.

LONGITUDINAL BEHAVIOUR

The longitudinal behaviour of the bridge and, more notably, the main cable dimensions and longitudinal post-tensioning are governed by the requisite of full compression under the frequent load combination in service as stipulated in the Irish National Annex of the Eurocode, see [1], [2].

Although there are different studies regarding the optimal design of the cable stay system in extrados bridges ([3], [4]), for the specific case of this bridge there were material and geometric constraints which left little room for optimization.

A single cable harp arrangement was chosen from the start for economic and aesthetic reasons, making it possible to reduce the width of the towers and, consequently, the width of the entire bridge.

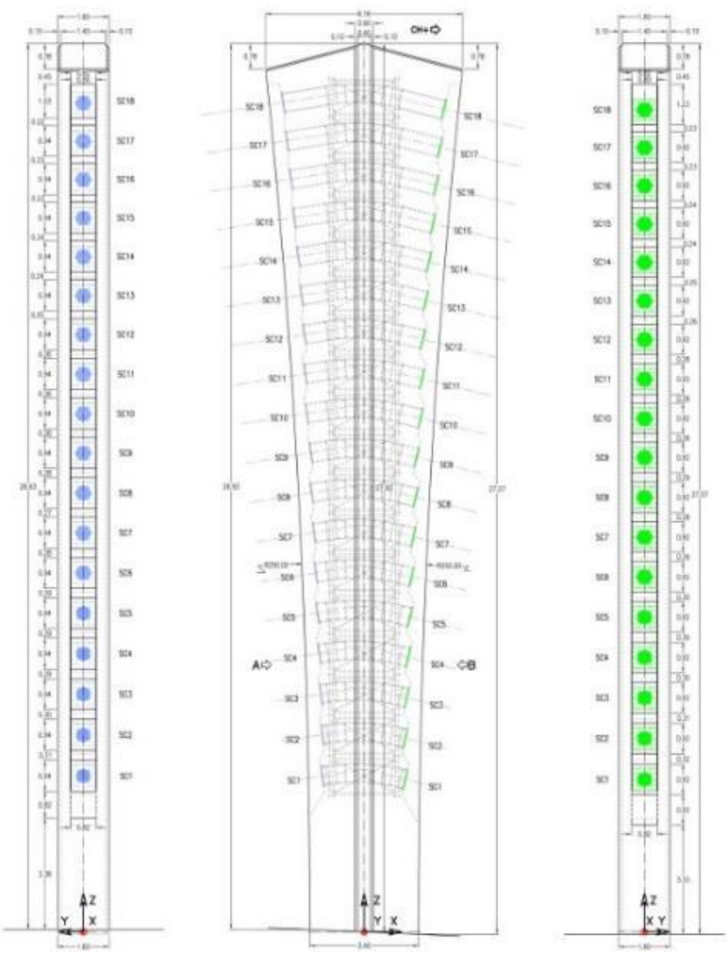


Figure 4: Cross-section and elevation of the central tower

(click on the image to see it in full)

The cables are practically parallel with a deck horizontal spacing of 6.5m between them and an average vertical distance of approximately 1.1m at the towers, resulting in 18 and 8 cables in each side of the central and lateral towers respectively.

The central tower is linked to the deck through a monolithic connection as it is longitudinally located approximately at the bridge mid-point and as it is characterized by a distinctly larger vertical load.

The remaining support points present a configuration of four pot bearings at the abutments and a pair of pot bearings for the remaining support locations.

The cable stay system supports approximately 50% of the weight of the deck, which is a low value for this type of bridge, implying a greater amount of prestressing.

These cables are continuous when passing through the towers thanks to the use of saddles. In this way, the dimensions of the towers are reduced to a minimum.

In addition to the different inferences due to the aforementioned constraints of the tower heights and main span lengths conditioning the shallowness of the cable action in this bridge typology, an added challenge was faced in the light of its vertical alignment.

Most of the bridge deck has a 5% slope, which produces a considerable lack of symmetry in the vertical load supported by these cables, Figure 5.

This results in the prestress distribution on the deck being greatly affected by this lack of symmetry.

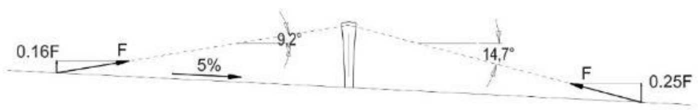


Figure 5: Longitudinal slope effects in typical cable configuration

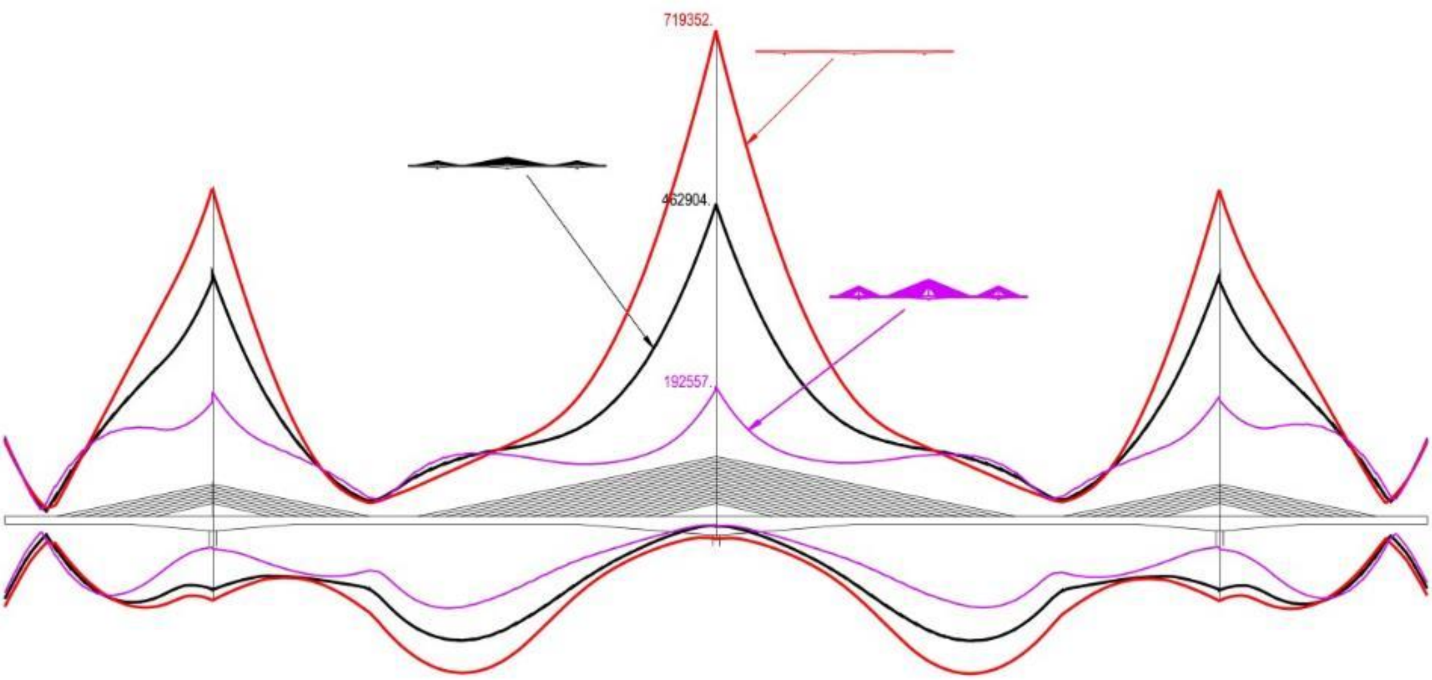


Figure 6: Bending moment law of the deck under live load model LM1 (EN 1991-2)

The most relevant aspect of extrados bridges in terms of its structural behaviour resides in the relative contribution of two superimposed structural systems: the deck as a conventional post-tensioned box, and the cable system.

The relationship between both is heavily influenced by the structural arrangement and dimensions chosen for the structure (pylon-deck connection, span/depth ratios at midspan and supports, number of spans and cable inclination, etc).

Figure 6 shows the bending moment law of the deck for the live load LM1 model of the Eurocode (Irish Annex) for River Barrow Bridge in comparison with the bending laws for the same load in the case of the deck being a continuous and a cable-stayed bridge with a conventional cable angle.

As expected, the behaviour of the extrados bridge is intermediate between that of the continuous beam and of a conventional cable-stayed system.

And in this case, the bending moment laws are a bit closer to those of the continuous bridge than to the cable-stayed bridge in terms of negative bending moments.

For positive moments, the values obtained can be considered halfway between those of the continuous bridge and those of the cable stayed.

This is a result of the particular slenderness of the bridge deck - within what is common in extrados bridges - and the shallowness of the cable arrangement.

According to current codes, ([5]–[7]) the stress range in extrados bridges is expected to be around the 50N/mm^2, with some granting a permissible higher maximum stress above the 0.45/0.5 GUTS when the stress range due to live load is sufficiently low, i.e. 50 to 100N/mm^2.

In the RFK bridge case, most of the cables are over the 100N/mm^2 range under the frequent live load combination, which again is a sign of the relative slenderness of the deck and the span arrangement.

The long main twin spans and the integral pylon deck connection add flexibility to the structural system, particularly on single span loading scenarios.

From a cable stay vs. post-tensioning relative distribution the decision was to use a main cable size with a maximum of 127 strands to ensure that the saddle and anchor sizes where within the range of those available for fatigue testing in existing laboratories, addressing the rest of compression needed in the deck with conventional post-tensioning cables of 27, 15 and 12 strands depending on their location.

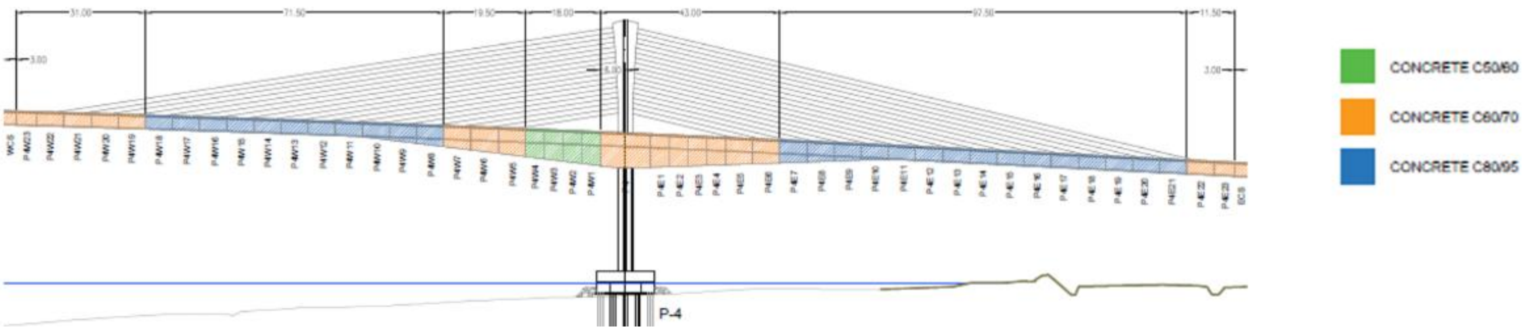


Figure 7: Distribution of concrete strength along the main spans from the central support

(click on the image to see it in full)

As a result of this, high strength concrete was required in a large part of the deck, going from C50/60 in the approaches, to C60/75 in part of the back spans and finally C80/95 in the main spans.

The distribution of concrete strength, as shown in Figure 7 was optimized for every segment in order to reduce the cost of the structure.

Due to the effects already described, the distribution is not symmetric along the central support.

It is also important to highlight that the sections that have the highest concrete strength are not located where the maximum bending moment occurs (at support) but around the areas where the section changes from constant depth to variable depth where the ratio between the axial load produced by the internal post-tensioning and the shallowness of the main cable combined with a relatively small concrete section produces the maximum stresses in the SLS (Serviceability Limit State) Envelopes.

This is another particularity of extrados bridges which is clearly reflected on this particular bridge due to its specific structural behaviour.

TRANSVERSAL BEHAVIOUR

At the main spans the deck is composed by an 8m wide single box and lateral 6.95m long cantilevers performing a total deck width of 21.90m that accommodates the dual carriageway and the central plane of cables.

The cantilevers are supported by precast panels in order to reduce its transversal bending.

The connection between the central plane of stays and the deck is naturally one of the most crucial elements of this structure.

Due to the shallow angle of the cable, and for the cable spacing chosen, 6.5m, the form tube of the cable interrupts the top slab in half its length between cables (3.3m), this forces the continuity of the top slab to be transferred in a relatively small area, leading to a distribution of forces and stresses both transversally and longitudinally.

In addition, considerable transverse bending is induced by main cable anchor locations due to the vertical component of the cable.

Despite the presence of diagonal props, the relative stiffness of the props and top slab in bending results in the load being shared between both structural systems.

Finally, the horizontal component of the stay force needs to be distributed transversally to the whole cross section resulting in equilibrium transversal forces (see Figure 8).

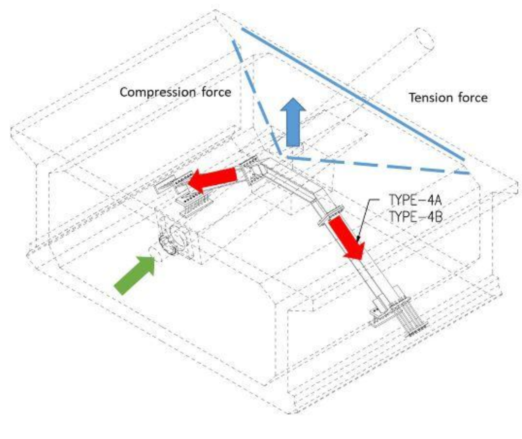


Figure 8: Schematics of the deck- stay central anchor

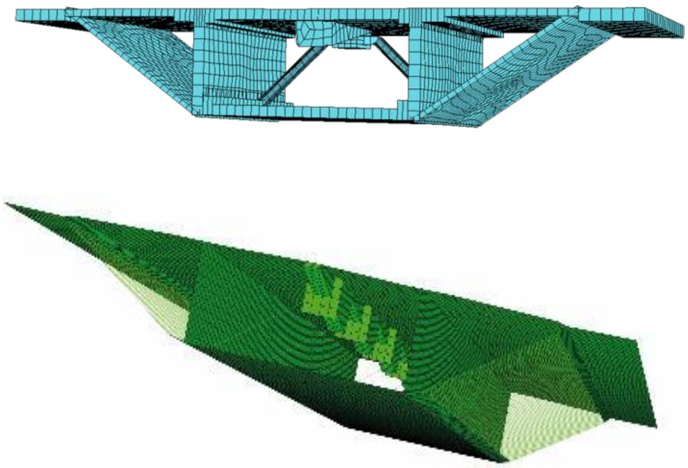


Figure 9: Sofistik (above) and Abaqus (below) FEM models for transversal behaviour analysis (click on the image to see it in full)

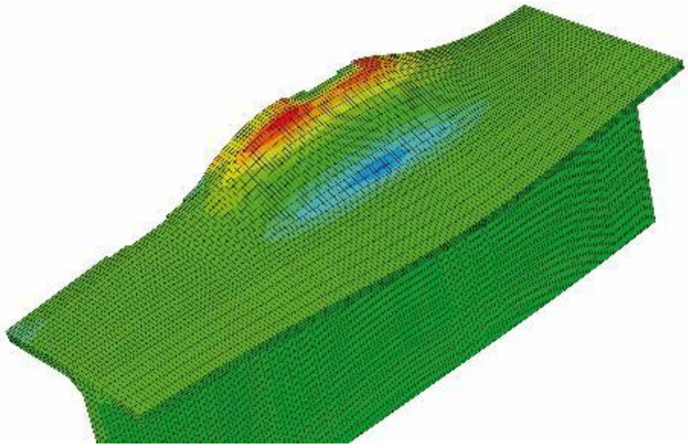


Figure 10: Transversal tension forces in top slab (3d brick Abaqus model - click on the image to see it in full)

The internal steel props are anchored to the bottom corners in the areas of constant deck height and to the webs at the anchor's locations closer to the towers where the section is deeper, which results in a different stiffness depending on their position in the web and required specific analysis of all the different configurations.

In order to analyse this complex element, different three dimensional finite element models using plate and 3d elements were developed on Sofistik and Abaqus, see Figures 9 and 10.

In addition, a further challenge in the dimensioning of the top slab was precisely related with the already congested slab and limited space between cables with the steel tube interrupting the slab around half this distance.

This implied that the aforementioned tension forces induced in the slab had to be transferred within the 3.6m continuous slab available in the centre line between cables in between form tubes, Figure 11.

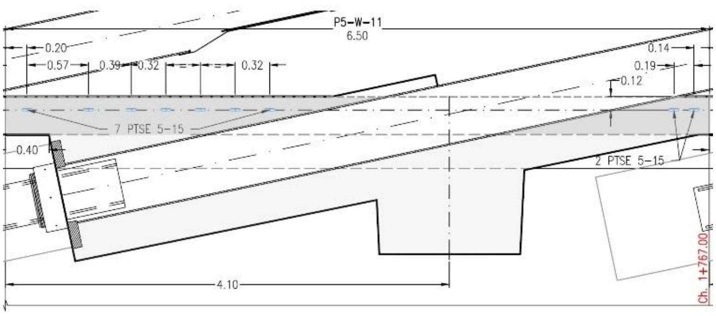


Figure 11: Longitudinal cross section at the central line for a typical segment in Span 4 and 5

SPECIAL STUDIES

Due to the particular nature of this project and contract requirements, different detailed studies were conducted in addition to the conventional designed scenarios, regarding wind, fire, ship impact and hydrodynamic modelling of the river caused by the central support.

The main features of these studies are described below.

WIND STUDIES

In addition to the conventional wind studies to demonstrate the stability of the bridge during service, the contract required to demonstrate that the bridge could remain open to traffic under high winds resulting from a 5 year return period.

Specific studies to demonstrate that this requirement was satisfied while optimizing the necessity of wind shields on the deck edges were carried out.

Having this in mind, experimental and numerical models were developed.

Here, different types of parapets and windshields were considered, and their effects in a model of the maximum height vehicle allowed in Ireland which corresponds to 4.65m were incorporated in the wind tunnel study, Figure 12.

This allowed the calculation of the wind coefficients to apply to both the deck and the truck in the analytical models.

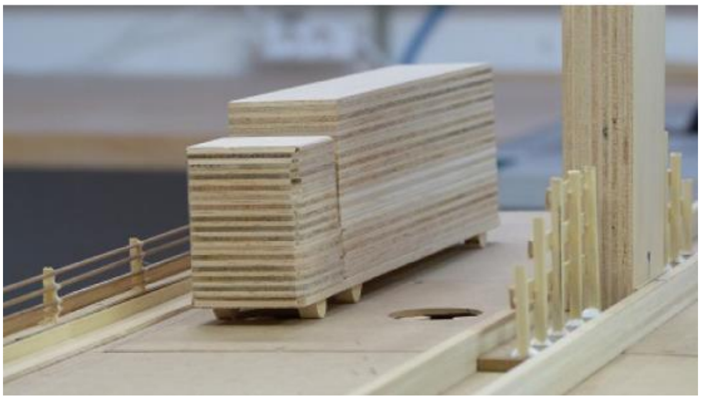


Figure 12: Truck at pylon location in the wind tunnel model

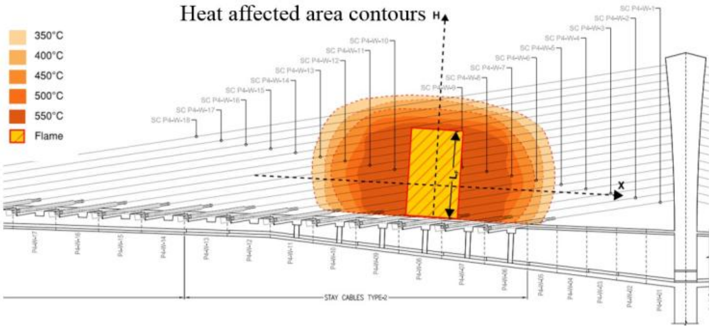


Figure 14: Isothermal contours on the deck under the fire scenario (click on the image to see it in full)

In this way, different runs were performed using these parameters for wind loads both on the structure as the vehicles which confirmed their stability under the required wind scenario.

From this analysis it was observed that no particular wind shield protection was required at the edges of the bridge in order to satisfy the 5 year return period requirement.

Notwithstanding this, in the case of the tower locations it was concluded that the installation of a small wind shield panel would be necessary due to the effects created locally by the pylons. This is depicted in Figure 13.

FIRE STUDIES

The particular specification of the project as part of the contractual requirements laid out by the Client also required the study of a potential scenario of a progressive collapse under the action of a fire occurring in the bridge.

Therefore a 50 MW fire source had to be modelled at any location of the bridge deck. Isothermal distribution curves were obtained for this scenario as depicted in Figure 14.

These results were then employed in a non-linear analysis of its produced effects in the structure.

As a result of this study, it was concluded that through the use of thermal blankets in the interior of the cables and only at their lower sections, the bridge would be able to sustain the considered fire scenario for 60 minutes.

This duration had been established as necessary in order to allow for the required mitigation services to be deployed considering their response time and the location of the structure.

SHIP IMPACT

The Barrow River is a navigable route and although this structure is situated 25km inland, it was also a contractual requirement to consider the verification of the substructure against the impact of a ship collision.

This was due to New Ross Port having significant activity. For this exercise, the established boat size was of 6000 DWT travelling at 8knots.

Initially, it was observed how the direct application of simplified approaches based in the Eurocode and AASHTO normative resulted in significant static loads between 38 and 71.3MN.

Therefore, a more detailed analysis was required taking into account the dissipation of energy of the

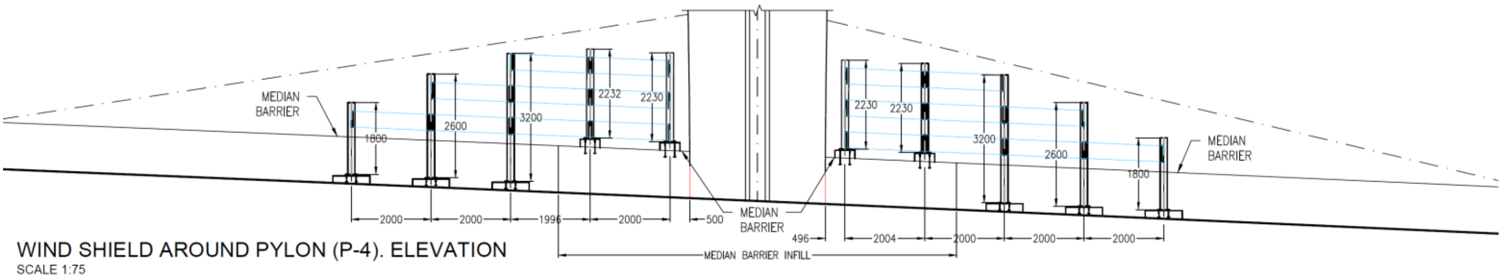


Figure 13: Wind barrier at I tower locations

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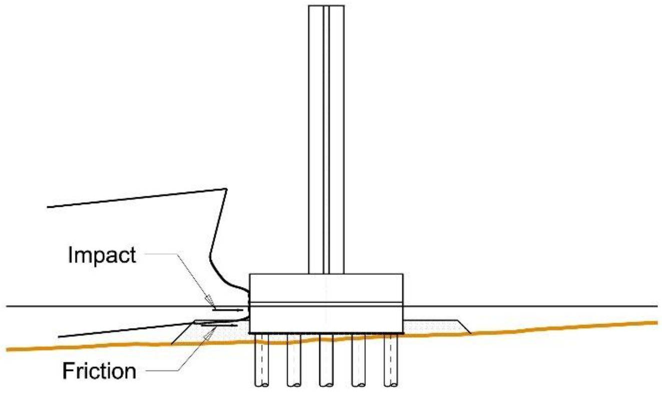

Figure 15: Elevation schematic of ship impact

ship while approaching the substructure, the non linear behaviour of the soil structure displacements during such scenario and considering a real representation of the ground configuration.

In this analysis and in function of the boat size and its load, two scenarios were envisioned.

The first one is where the ship would come to a full stop before reaching the substructure due to the friction between the bottom of the ship and river bottom and a second where the ship would eventually reach and impact the structure, see Figure 15.

For this second scenario it was necessary to ensure that the pile cap was able to absorb the kinetic energy of the ship.

In this analysis a 2d model of the pile cap with non linear springs was used, (Figure 16). This analysis was repeated using deterministic values of tonnage, ship speeds and angles of impact.

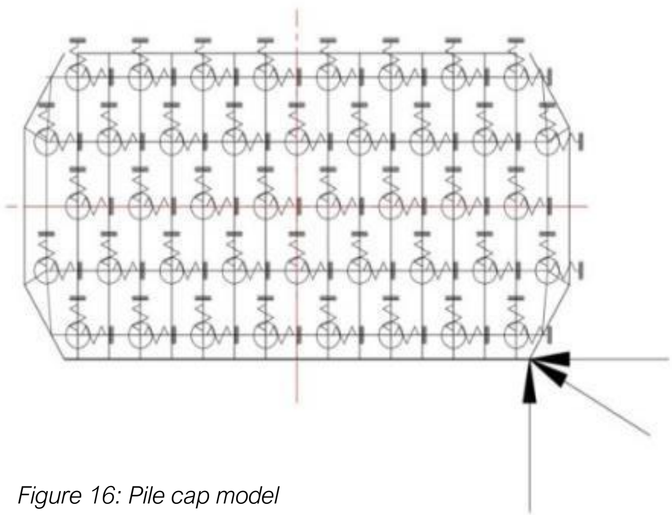

Figure 16: Pile cap model

In this way, it was possible to obtain the moment displacement diagram curves for the pile cap. It was concluded that in order to verify the safety conditions for this scenario, a small rock revetment, which also serves as scour protection proved to be sufficient to reduce the forces in the piles to acceptable values.

HYDRODYNAMIC STUDIES

The central pier P4 is located within the river tidal range, in an environmentally sensitive area.

As part of the environmental constraints it was required to demonstrate that both under the construction stages, which required a road access to the foundation of P4 and in the permanent situation, where the pile cap and rock revetment are in the tidal range, the hydrodynamic flow did not change significantly the sedimentation patterns in the river bed affecting the navigational channel and the environmentally sensitive mood flats in eastern river bed in particular.

As shown in Figure 17, this required an iterative process and comparative studies between different pile cap and revetments form and sizes.

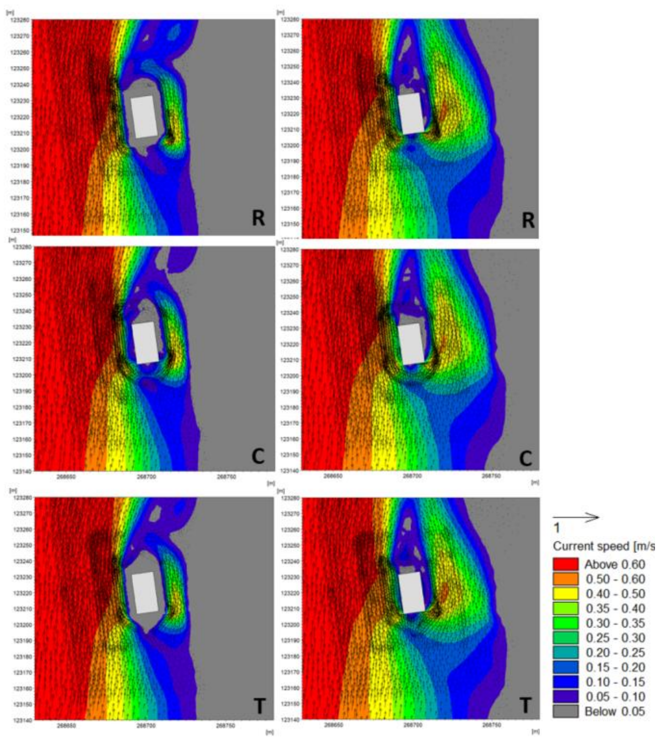

Figure 17: Hydrodynamic model at different river levels and for different pile cap and rock revetment forms

The study, which was supported by bathymetric studies carried out before and after the construction, demonstrated that the riverbed was not significantly affected by the solution finally chosen.

ERECTION ENGINEERING

In this section the main details regarding the adopted construction process is described for the different elements of the structure. Different methodologies were implemented due to different reasons which are further outlined in the sections below.

SUBSTRUCTURE

The construction methods deployed for the substructure were mainly conventional. The spread footings in the west side of the structure, i.e. from A1 to P2 were able to be developed in situ using local excavations.

Pier P3 foundation, a spread footing as well, due to its location and the bedrock at river level within the tidal range required an auxiliary wall and provisional protection during construction.

For the main central support P4, in order to carry out the pile cap as well as the provisional cantilever balancing tower (necessary for the erection of the superstructure and detailed below), a temporary peninsula was created in the middle of the river with an access road. This allowed the execution of the foundation without the need for sheet piles or a cofferdam at this support.

The remaining foundations to the east side of the structure were located in dry terrain outside of the riverbed. Therefore, depending on each pier particular condition, deep or superficial foundations were constructed through conventional means with localized excavations. From a design point of view there were no specific requirements for the substructure.

SUPERSTRUCTURE

After a detailed study to optimize the construction method solution, mainly aimed at simplifying the overall process and reducing deadlines, it was decided to execute the superstructure with different construction methods for the approach and side spans and those deployed for the two main spans.

This resulted in the use of four form travellers, two of them in P4 progressing in balanced cantilever and two starting from P3 and P5 with a single cantilever front after the side span is built.

The approach spans were built using scaffold for the main box and a wing traveller supported in the section already built for the side cantilevers.

Three temporary towers (in yellow in the Figure 18 below) were required.

Two of them in the side spans, located half way along the side spans, allow the deck on the approaches to be built on scaffold prior to the installation of the main cables and the cantilevers of the main spans.

These two temporary supports work only in compression and they will lose contact with the deck a certain stage in the construction process.

On the other hand a temporary tower located 24m away from the central tower was used to reduce the bending moments on the central pier (P4) during construction (minimizing its size).

This temporary tower required vertical prestressing in order to guarantee that it will remain under compression during the cantilever stages under all load cases.

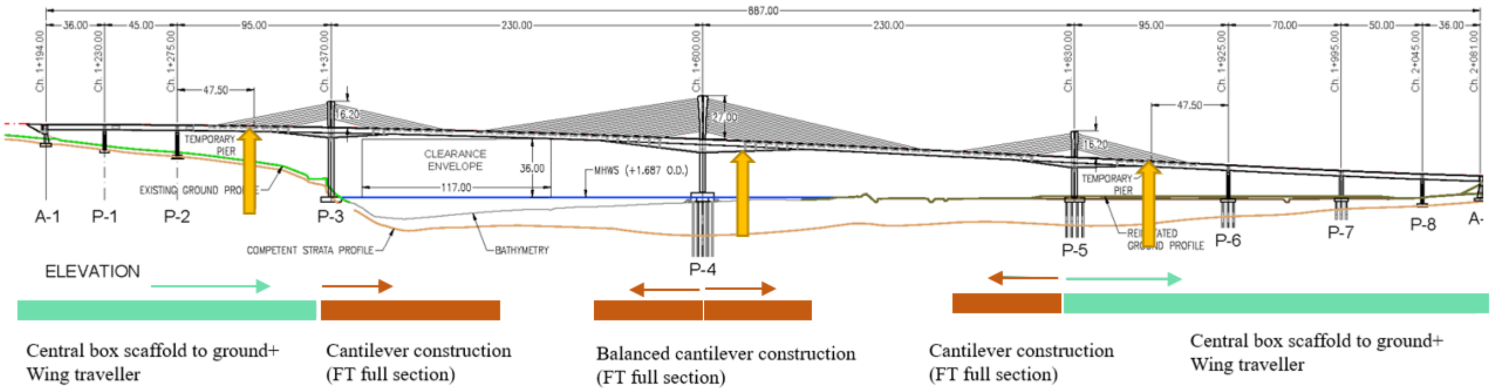

Figure 18: Superstructure construction methods schematic (click on the image to see it in full)

Figure 19: Construction of central box at Spans 1 and 2 in the west area of bridge using scaffold to the ground

Figure 20: Approach spans construction of precast slabs and cantilevers using a wing traveller

Approach viaducts spans

As indicated, the approach spans were built in two stages. Taking advantage of the already built central box, an independent wing traveller was used for the installation of the precast slabs and casting of the cantilevers, see Figures 19 and 20.

The use of this process method associated with the decompression requirement along the entire width of the section and not only in the proximity of the tendons imposed by the Irish National Annex to the Eurocode [2], enforced a cautious consideration of the longitudinal post-tensioning due to the significant tension states generated at every stage of the construction process, having a critical impact in the final layout of this element, the cross section was cast in three stages (bottom U, top slab and wing and precast panels, requiring a multi stage post-tensioning at cross section level).

This construction process resulted in the requirement for detailed calculations for SLS verification of the main box and the internal prestressing to be applied in several stages in order to guarantee the decompression under Service, which resulted in a significant increase in the post-tensioning quantities resulting from a single stage construction.

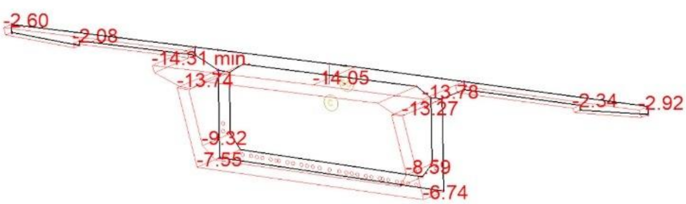

Figure 21: Stress check on the side span under construction

Main Central Spans

As seen in Figure 18, the two central spans of 230m length were built using the cantilever method with form travellers.

Due to the aforementioned asymmetry of the towers, the number of segments was different on the central pier P4 and the side towers.

Also as indicated, the side towers were built after the side span was built on scaffold so only one cantilever front was required.

This asymmetry also implied different cantilever lengths, of 140m coming from P4 and 90m from P3 and P5.

Figure 22: Balance cantilever method deployed at P4 prior to connection to the Push Pull prop

A full section traveller was used on each of the four fronts of the cantilever's construction.

The longitudinal post-tensioning was introduced in every cycle of this process by means of tendons in the first sections closer to the towers and then advancing to DYWIDAG bars of 47mm once the constant depth section was reached in order to speed up the construction times.

The construction sequence which affected this longitudinal post-tensioning, and which was agreed with the construction team, consisted of the following steps:

- Post-tensioning of the longitudinal PT on segment n-1.
- Forward movement and setting out of the form traveller in segment n, including geometry corrections.
- Placement of the precast slabs and reinforcement in segment n.
- Transversal post-tensioning of segment n-2.
- Stressing of main cable in segment n-2 to the construction target load.
- Pouring of concrete in segment n.

In advance of the cantilever construction, specific creep and shrinkage tests were performed in order to more realistically predict the produced and measured deflections of the deck during construction.

At this stage it was predicted a duration for each cantilever production cycle between 7 and 15 days.

However, the schedule during construction was far more variable as seen in Figure 23 which further complicated the geometric control and required a continuous update and verification of the models.

Two different models on different software packages were used by ARUP and CFC to ensure that the information provided to site during the stage of setting out the form traveller levels was as accurately as possible.

Although the deflections measurements obtained in the first segments were within the expected tolerances, from the segment 12 of the central pier, significant discrepancies began to be found between the deformations predicted in the model and the results provided by topography.

This led, particularly in segments 14 to 20, to a more exhaustive control of the geometry and the deformations.

The conclusion of this specific analysis carried out in segments 12 to 14 was that these discrepancies were due to the elastic and short term properties of the high strength concrete in early hours (these segments were cast in cycles of 12 to 15 days with the form traveller strike and segment post-tensioning taking place after 36 hours).

This required an adjustment to the geometry control models and to extend the time to strike the formwork to 60 hours in the following segments.

Finally, and as demonstration of the different flexibility of each cantilever, a maximum deflection

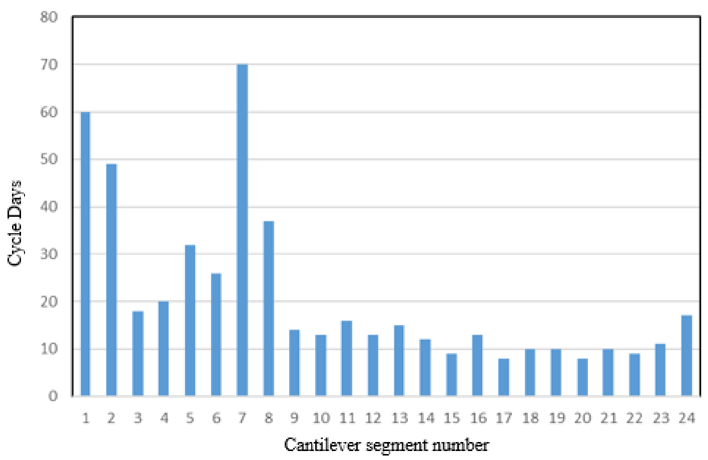

Figure 23: Duration of cantilever segment construction in west front of P4

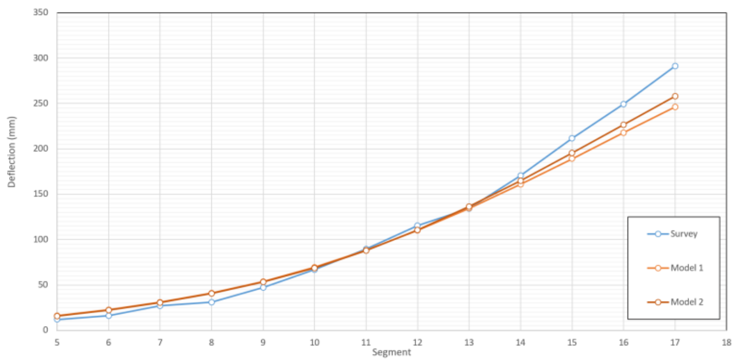

Figure 24: Deflections due to the 50% stressing of cable 12 in the central Pier. Analytical models vs survey

difference between cantilevers, as shown in Figure 25 below, was achieved at the time of casting segment 24 and before stressing cable 17 (out of 18) in the central cantilever.

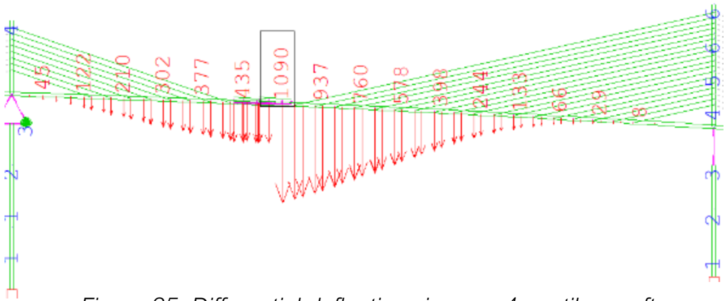

Figure 25: Differential deflections in span 4 cantilever after casting segment 23 (and before stressing cable 17)

Due to the asymmetry of the cantilevers, a specific locking system designed to ensure that no differential deflections or rotations will occur during the casting of the central segment was designed.

The locking element consisted of two steel beams 1.5m deep located in the top of the deck. Each beam had 4 rows of two pairs of 40mm diameter McAlloy bars that, in addition to ensure that no relative displacements between cantilevers could take place during the casting of the central segment, were also designed to allow a correction in differential level of the cantilevers up to 400mm. This was much higher than the predicted and achieved differential deflection values of 150mm and 120mm in the west and east cantilevers respectively. Figure 26 shows the central segment being cast.

Figure 26: Cantilever blocking beams during the pouring of the closing segment

CONCLUSIONS

The Rose Fitzgerald Kennedy Bridge over the Barrow River is a milestone in the design and construction of bridges of this typology. As a world record breaker span with a full concrete deck, its design and construction represented a significant challenge for the design and construction team. This being the case not only due to its size but also the slenderness achieved, and the geometrical constraints derived from the Environmental Impact Statement.

The fact that this structure presents a very slender deck affects the load distribution between this element and the cable system leading to a behaviour more closely related with those of cable stayed bridges in comparison with other extrados bridges.

From an aesthetic point of view, this bridge is also unique due to the difference in height between the central tower and the side towers which also creates an asymmetry in the cable arrangement in relation to the central spans.

Because of the aforementioned slenderness of the deck, 3.5m deep at the tip with a maximum cantilever of 140m and extremely shallow cables angles (10 degrees with the deck) the geometric deflection control during construction was especially complicated, with the added difficulties of early age properties of the high strength concrete mix used in the project.

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DESIGN & CONSTRUCTION

OF THE ROSE FITZGERALD KENNEDY BRIDGE

Keith Tully, BAM Civil

Declan Roche, BAM Civil

INTRODUCTION

In January 2020, BAM Dragados completed the construction of the N25 New Ross Bypass, including the construction of the Rose Fitzgerald Kennedy Bridge, allowing traffic to cross the River Barrow and Bypass the heavily congested townland of New Ross.

The completion of the project and the Rose Fitzgerald Kennedy Bridge marked an important milestone for the people of New Ross and those who regularly frequent the busy N25 and N30 national primary routes in the area.

It also marked an important milestone for bridge engineering in Ireland with the completion of what is now Ireland's longest bridge at 887m long along with a world record concrete extradosed span of 230m.

In 2014, after extensive route selection, planning and preliminary design, the N25 New Ross Bypass was issued for tender by Transport Infrastructure Ireland (TII) under the form of a Public Private Partnership (PPP) Contract. The project comprised of:

- 4km of Type 1 Dual Carriageway, linking the existing N25 at Glenmore to the R733 at Landscape via the new Rose Fitzgerald Kennedy Bridge
- 9.6km of Type 2 Dual Carriageway, linking the R733 at Landscape to the existing N25 at Ballymacar Bridge and continuing to a roundabout southeast of Corcoran's Cross on the existing N30.
- 1.2km of Single Carriageway road, connecting the new roundabout southeast of Corcoran's Cross to the existing N30 to the east of Corcoran's Cross.
- 3 at-grade roundabouts, at Glenmore (N25), Ballymacar Bridge (N25) and Corcoran's Cross (N30)
- A compact grade separated junction at Landscape (R733).
- Multiple overbridge and underbridge structures
- An 80m long 3 span post tensioned railway structure
- An 887m long 3 tower extradosed bridge crossing the River Barrow

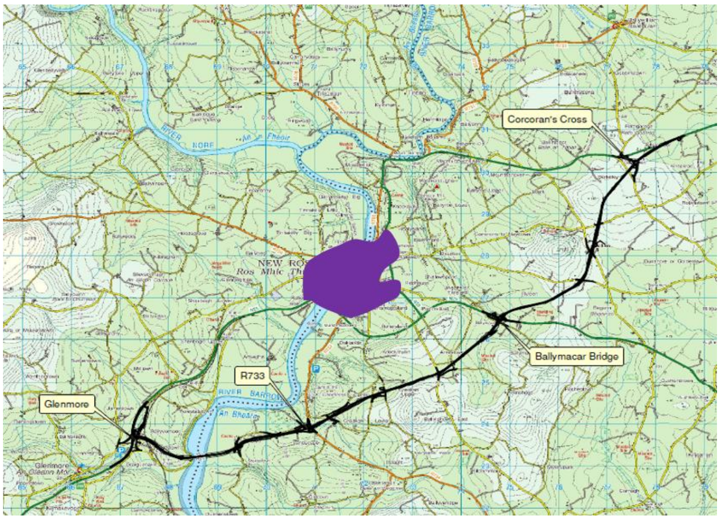


Figure 1: Route Alignment of the N25 at New Ross

TENDER STAGE

BAM Ireland and Dragados, having previously worked successfully together on the Dundalk Western Bypass PPP, N25 Waterford Bypass PPP and M7/M8 Portlaoise Bypass PPP, formed a joint venture together with their parent companies to tender for this iconic project.

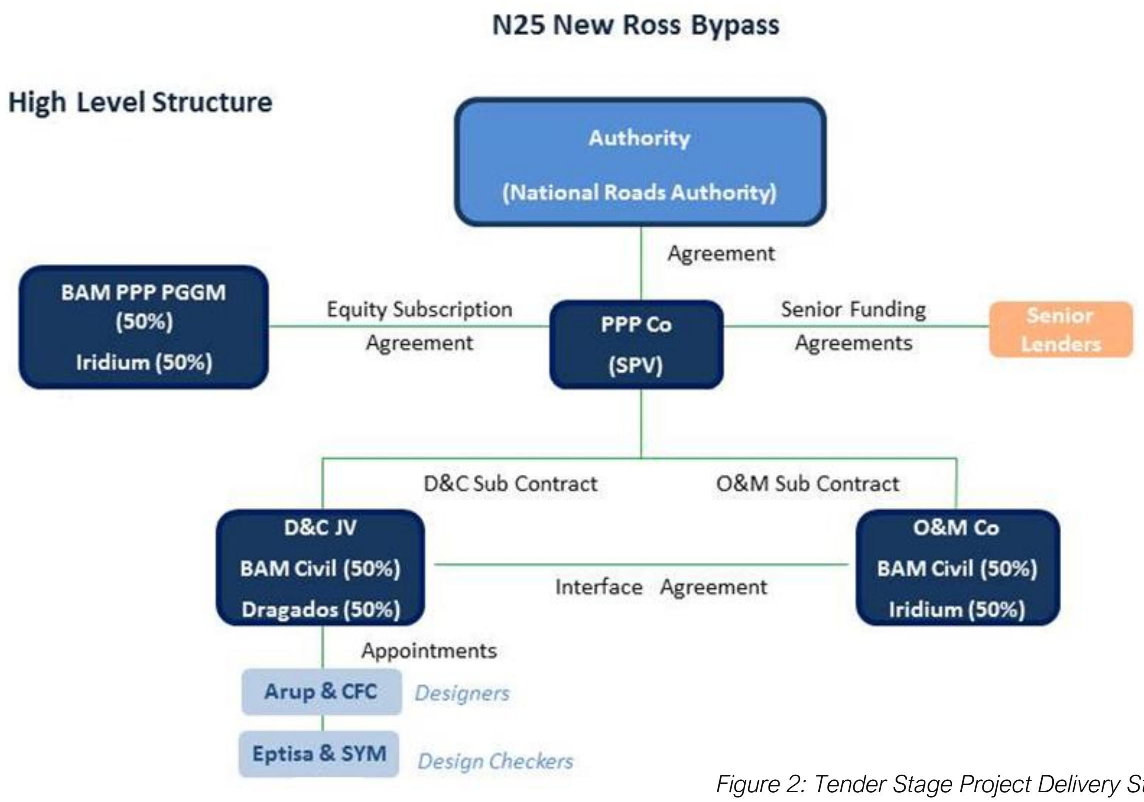


Figure 2: Tender Stage Project Delivery Structure

A Special Purpose Vehicle (SPV) was established to represent the PPP Co which comprised of BAM PPP PGGM and Iridium.

BAM and Dragados formed an integrated joint venture to design and construct the project with BAM and Iridium forming a joint venture to deliver the Operational and Maintenance (O&M) phase of the contract for the 25 year concession period.

Following a competitive tender process, in January 2016 BAM PPP PGGM and Iridium were awarded the PPP contract to design, construct, operate and maintain the project.

DETAILED DESIGN

Following the contract award in January 2016 BAM-Dragados commenced the detailed design stage of the project and with particular focus on the Rose Fitzgerald Kennedy Bridge.

The detailed design for the bridge was undertaken by Carlos Fernandez Casado S.L. and Arup, with Eptisa and Siegrist y Moreno undertaking the role of Category 3 checker.

As a follow on to the dialogue with TII during the tender process various elements of the structure were prescribed within the contract including the span arrangement, the pier locations and tower heights.

During the tender stage Carlos Fernandez Casado S.L. had identified a number of modifications to the Mott MacDonald specimen design and these were taken forward in the detailed design of the bridge.

The key aspects of the bridge design included:

- A span arrangement of 36m, 45m, 95m, 230m, 230m, 95m, 70m, 50m, 36m.
- Piled foundations at 4 of the 10 bridge supports. The piles at the central pier, pier 4, comprised of 43 number 1200mm diameter reinforced concrete piles bored to a depth of over 40m to reach, and socket into, the underlying bedrock.
- 18 number stay cables in the centre pylon 4 with a maximum size of 125 strands
- 8 number stay cables at pylons 3 and 5 with a maximum size of 109 strands.
- A central deck concrete box varying in depth from a nominal height of 3.5m to 8.5m at the deepest haunch at pier 4.
- Inclined precast panels forming a façade to the central box

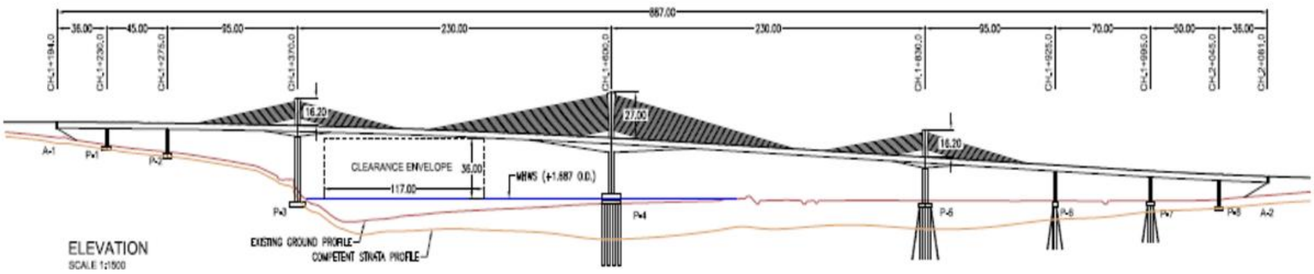


Figure 3: Bridge Span Arrangement – Click on the image to see it in full

The detailed design of the bridge presented various technical challenges and required a number of specific technical studies, including:

- *Wind tunnel testing & analysis* to ascertain wind loading on the bridge, the aerodynamic performance of the precast concrete parapet edge beam and the overall effects on traffic stability.
- *Hydrodynamic modelling and analysis* to demonstrate that both the temporary and permanent works at pier 4 had no adverse impact on the channel hydrodynamics and long-term sediment transportation. This modelling was also used to define the requirements for scour protection around the pier base.
- *Ship impact analysis* to determine all realistic collision scenarios and their effect on the bridge substructure at both pier 3 and pier 4. The analysis looked at a range of vessels up to the specified 6,000 DWT vessel.
- *Fire Analysis* to establish the effects on the structure of a 50MW fire in order that the structure be designed to prevent disproportionate collapse in such a scenario.
- *Application of dynamic crowd loading* to obtain the corresponding acceleration values.

OUTLINE CONSTRUCTION SEQUENCE

One of the early engineering aspects of the detailed design and construction stage was to establish the outline construction sequence, given its important relationship to the design of the bridge and also in order to align the design and construction programmes.

Consideration had been given to sequence during the tender phase and this was later finessed during the detailed design. The first aspect of this was to finalise the method of construction for the different parts of the structure. This comprised of:

- A falsework system for constructing the central box of the deck in spans 1, 2 and 3 on the west side of the river and spans 6, 7, 8 and 9 on the east side.
- A wing traveller system to complete the deck construction for these spans.
- A form traveller system for the cantilever construction of spans 4 and 5.

Two temporary piers were also required in the centre of spans 3 and 6 along with a temporary Push Pull Prop in span 5 adjacent to the central pier.

A number of distinct sequential phases were subsequently identified for the construction sequence:

1. Construction of pier and abutment foundations including piling works.
2. Construction of abutments and pier stems, including temporary piers and the Push Pull Prop at pier 4.

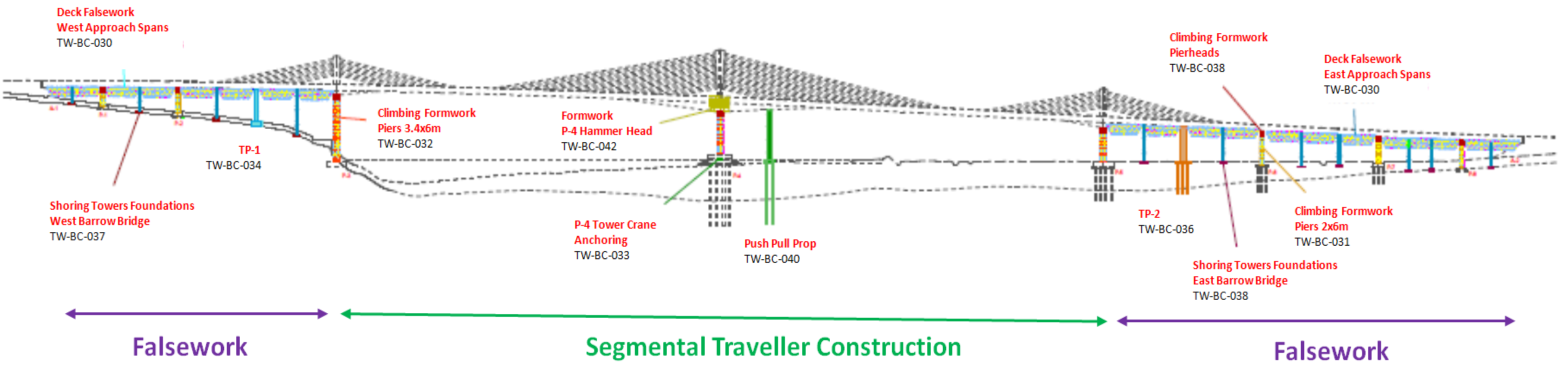


Figure 4: Methods of Construction – Click on the image to see it in full

3. Construction of the deck on a falsework system commencing at span 1 on the west side and span 6 on the east side as illustrated in Figure 5. The bridge deck bearings were locked for the construction phase at abutment 1 and pier 5 on the west and east sides respectively.

Figure 5: Falsework in Span 2

4. Construction of deck spans on the east and west sides continuing towards pier 3 and abutment 2 respectively.
5. Construction of the hammerhead at pier 4 to enable the traveller system, as illustrated in Figure 6, to be erected at the top of the pier.

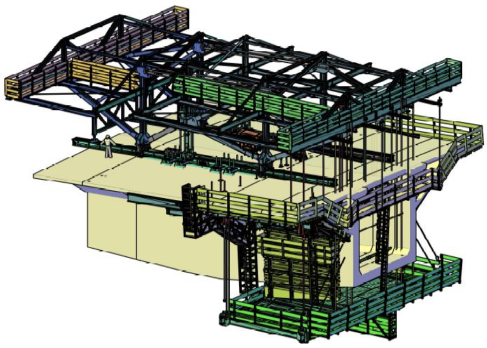


Figure 6: Form Traveller System

6. Construction of the balanced cantilever deck at pier 4 in parallel with the pylon construction.
   Early in this stage, the temporary Push Pull Prop was also engaged.
7. Construction of the hammerheads at pier 3 and pier 5 to enable the traveller systems to be erected.
8. Completion of the central box construction on the falsework systems on both the east and west sides of the bridge.
9. Construction of the wings on spans 1, 2 and 3 on the west side and spans 6, 7, 8 and 9 on the east side utilising a wing traveller system as illustrated in Figure 7.

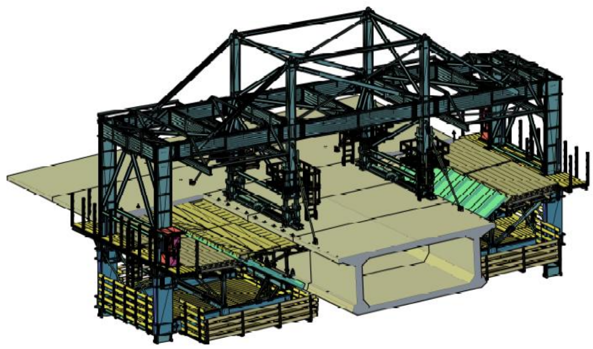


Figure 7: Wing Traveller System

10. Construction of the deck cantilevers at piers 3 and 5 in parallel with the associated pylon construction.
11. Locking of the adjoining cantilevers in spans 4 and 5 along with release of the temporary bearing locks at abutment 1 and pier 5.
12. Construction of the closure segments in spans 4 and 5 followed by the release of the temporary Push Pull Prop.

From these distinct outline phases, further sequencing methodology was developed between the design and construction teams. A 3 stage pour sequence was defined for the box section on falsework construction as illustrated in Figure 8.

The 3 stages comprised of:

1. Pouring the u-section of the box
2. Pouring the roof section over the support elements
3. Pouring the remainder of the roof section

Figure 8: Approach Span Pour Sequence

A detailed sequencing methodology was identified for the balanced cantilever construction. This sequence included;

- Launching of the form traveller exterior followed by setting out and level adjustment
- Installation of prefabricated web-wall reinforcement
- Installation of insitu bottom slab reinforcement
- Installation of pre-cast inclined wings
- Launching of the interior formwork web-walls and deck soffit
- Installation of post tensioning blisters, form tube for cables and the anchor block
- Installation of sleeves for next traveller position
- Installation of insitu deck reinforcement
- Casting of the concrete for the segment
- Concrete strength gain >37MPa and minimum specified curing duration achieved
- Installation of the post tensioning strand/bar to tie segment to previous deck cast
- Survey of the deck including the newly cast segment.
- Launching of the form traveller exterior followed by setting out and level adjustment
- Cable installation takes place in tandem, 2 segments behind

Using these principal tasks, a detailed overall construction sequence was determined with over 450 steps inter-linking the construction of the approach spans box with their connecting wings, the segmental construction of the main spans, the post-tensioning, cable stressing and finishing works.

TEMPORARY WORKS DESIGN

The bridge construction necessitated substantial temporary works, primarily the falsework system, the main traveller system, the wing traveller system and the impermeable sealed area within the river channel, which was necessary to construct pier 4, the push-pull prop and their associated foundations.

Pier 4 Foundation & Temporary Working Platform

The central pier on the bridge was located within the river channel, approximately 70m from the river bank.

The foundation for this pier comprised of a large concrete pilecap / pedestal measuring 14m by 27.4m by up to 5.4m deep, supported by 43 number 1,200mm diameter reinforced concrete piles over 40m in length. The piled solution was necessitated to support the substantial pier loads and found the bridge on the competent underlying bedrock.

In advance of finalising the design of the piles, additional ground investigation was also undertaken. A jack-up barge was used, as illustrated in Figure 9, to undertake rotary coreholes, Menard pressuremeter tests and piezocone penetration tests (CPTu) at the location of the foundation. While previous ground investigation information was available, the additional investigation was necessitated to provide a greater level of certainty.

Figure 9: Jack-up Barge for GI Works

To facilitate the construction of Pier 4, its foundations and indeed the adjacent Push-Pull Prop, a substantial temporary works solution was required.

This comprised of a temporary working platform constructed in the river channel which was connected to land by a raised jetty structure supported on 508mm diameter tubular steel piles.

Both the platform and the jetty structure were designed to support the construction loading for the bored piling, pier and hammerhead works. This included Bauer BG 28 & 42 Piling rigs, a Sumitomo SC1500 crawler crane and a Liebherr LTM1750 mobile crane.

As illustrated in Figures 10 and 11, the footprint of the extensive working platform was approximately 78m in length and up to 74m in width.

A detailed temporary works design was undertaken for the platform which was made more challenging by the very soft alluvial soils on which the platform was to be constructed and indeed the heavy equipment that would subsequently use the platform over the course of the construction phase.

The temporary works design for the platform comprised of:

1. A washed quarry run material sourced from an adjacent rock cutting on the project.
2. A separation geotextile with a CBR punching resistance of 9000 and a tensile strength of 50kN/m.
3. Longitudinal and transverse basal geosynthetic reinforcement with a design tensile strength of 520kN/m with washed coarse sand friction layers between the layers of geosynthetic material.
4. An external impermeable liner to prevent water from seeping into the pilecap excavation which was in the centre of the working platform.
5. An additional internal 3m deep impermeable clay barrier around the excavation for the foundation.

Travellers

Form traveller systems are ideally suited to this type of construction due to their practical and cost-effective methodology for construction over spans where ground bearing is not possible.

They are commonly used for the free cantilever construction of post-tensioned box-girder and cable-stayed concrete bridges.

They provide for a rigid formwork system, with minimum deflections at the leading edge due to the tie-back structure.

They are considered relatively lightweight for the load they are capable of carrying, often between 250t - 450t, versatile and easy to operate by rolling / pushed forward on a rail system.

In this way the system can be reset quickly and easily without being dismantled, a key attribute for a linear construction programme where cycle time is key.

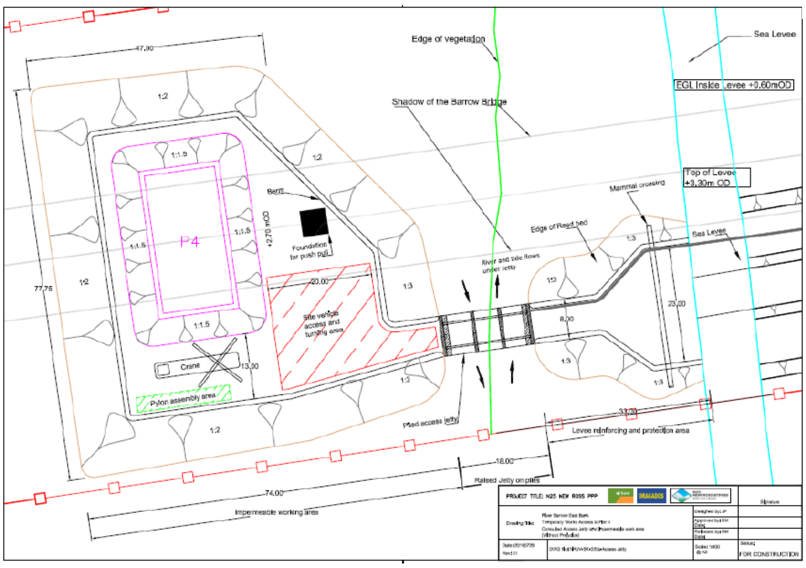


Figure 10: Plan of Temporary Platform

Figure 11: Piling Works on the Platform

In the case of the Rose Fitzgerald Kennedy Bridge, consideration was given to both a “full-width” and a “central-box only” traveller.

The latter only allowing for the construction of the post-tensioned box-girder, with the wing construction following afterwards.

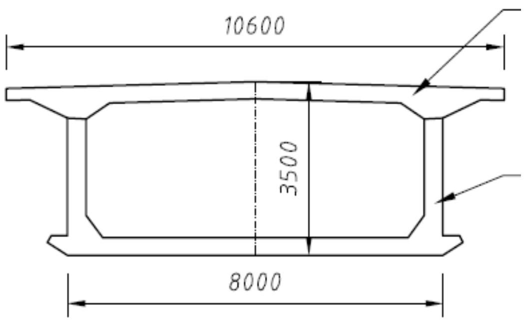


Figure 12: Box Only Option

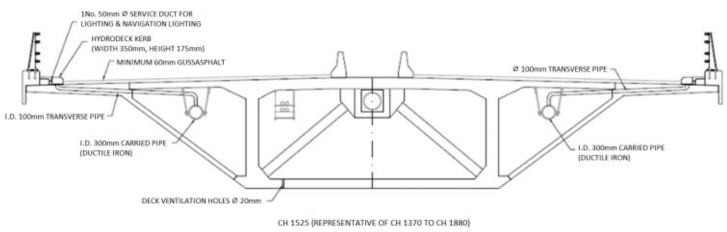


Figure 13: Full Width Option
Click on the image to see it in full

The maximum width of the deck under consideration was 22m with a segment length of 6.5m.

A detailed review of the two options was carried out by the construction team, giving consideration to a number of factors including, but not limited to: cycle times, traveller cost, follow on construction of wings, working space and access arrangements, geometry control, temporary works, material storage, cable installation and plant constraints.

While the “full-width” traveller was more costly to procure, the “central-box only” solution provided other complexities and challenges for the follow-on construction of the wings.

Therefore, following an extensive review of the cost benefit analysis, the construction team selected the full-width traveller system from *Rubrica Bridges* of Castellón in Spain.

Four travellers were procured in order to meet the programme with all spans being constructed in parallel.

One traveller was erected at each of piers 3 and 5, where the adjacent approach span deck had already been constructed.

Two travellers were erected back to back at pier 4 on top of the hammer-head.

Due to the deeper segments adjacent to pier 4, these two travellers were larger at circa 155t each, with the heaviest segment weighing circa 430t following the concrete pour.

DECK CLOSURE SYSTEM

On completion of the cantilever construction at piers 3, 4 and 5, connecting closure segments were required to structurally connect the bridge deck in spans 4 and 5.

A bespoke temporary solution was developed for this operation to lock the adjoining deck cantilevers together and allow the closure segment works, namely concreting and post-tensioning works, be completed.

In addition, the temporary works were designed to correct minor level discrepancies between the adjoining cantilevers.

This was all the more prevalent given the differing length of the cantilevers and the added complexity this creates to the predicted deflections and the associated geometrical control.

The locking system in each span consists of two steel plate girders (locking beams) with a hollow box cross-section.

The height of these beams was 1200mm with a width of 530mm and an overall length of 11305mm.

The beams were supported on concrete pads above the webs of the central deck box and were then fixed to the deck using 50mm diameter Macalloy bars with a pre-stressing force of 1,090kN in each bar.

As illustrated in Figure 15 below, the beam was first secured to the higher cantilever using hydraulic stressing jacks.

The jacks were then used to load the bars on the lower cantilever and slowly remove the level difference.

Figure 14: Locking System

Once the level difference was removed between the adjoining cantilevers the beams were then fully secured, following an exact pre-determined stressing sequence for the each of the bars.

Once this operation was complete, the temporary restraints were removed from the bearings at abutment 1 and pier 5.

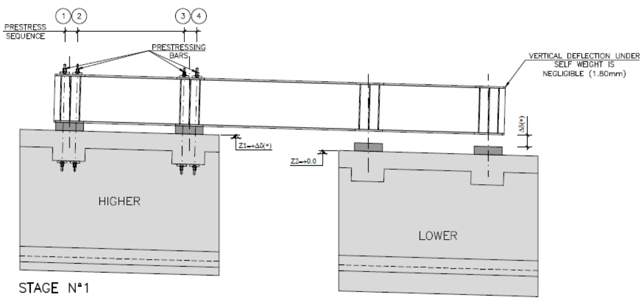


Figure 15: Locking System (Before Locking)

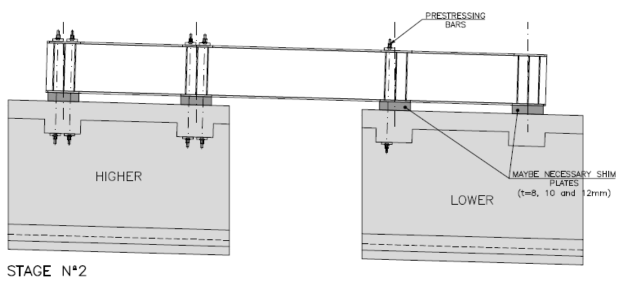


Figure 16: Locking System (After Locking)

Reinforcement and concreting works could then proceed in the 2 closure segments.

The closure segments in spans 4 and 5 were not only a significant engineering challenge but they also marked one of the most important milestones in the construction of the bridge.

Following over 20 years of planning and design, the townland of New Ross now had a new river crossing over the River Barrow.

CABLE STAY SYSTEM

As noted earlier the bridge is supported by cables in four of the spans with lengths of 95m, 230m, 230m and 95m. A cable-system was designed along a single vertical plane coincident with the box central axis.

The cables were designed with the same size anchorage for each cable. The external sheaths were also the same size, irrespective of the variation in cable size.

The maximum number of strands per cable was 125 for the central spans at pier 4, and 109 strands for the approach spans at piers 3 and 5.

Following an extensive review of a number of stay cable solutions by the construction team, the system selected was the 127TSR15 stay cable system from Tensa in Milan.

Tensa's system was one of only a few that could cater for the large cable size required, combined with their commitment for full scale cable system testing clinched the deal.

The stay cables were formed using up to 125 number of individual parallel strands enclosed within an outer HDPE (high density polyethylene) protective pipe. The strands were formed using 7 number 1860 MPa wires twisted to form a 15.7mm diameter strand.

The individual strands were coated with a black HDPE sheath and provided with a petroleum wax protective filler in the interstices between the wires comprising the strands.

The strands were delivered to site in coils weighing circa 3,000kg.

The outer HDPE pipe was selected to be of a light colour giving the structure an unobtrusive appearance. This pipe was formed using the bi-extrusion method whereby which a thin coating of light colour UV resistance HDPE material was extruded over a black HDPE pipe.

The pipes were delivered to site in 12m lengths and fusion welded to the required lengths.

The design of the bridge required that bespoke saddles were used in the pylons to deviate the cables' varying geometry from the deck through the pylons and back to the anchorage at deck level.

They also had the specific purpose of transferring loads from the stay-cables vertically into the pylon.

This was achieved by means of series of steel tubes formed into a single continuous "saddle" arrangement embedded within the concrete of the pylon and through which the cables were passed and back down through the deck where the stay cables were anchored by cable anchorages located under the reinforced concrete bridge deck.

The HDPE sheath over the length of strand that passed through the saddle was removed during installation and the resulting friction between the strand and the inside of the tube guaranteed that each strand was "fixed" within the saddle.

This ensured that the correct tension could be applied to the cables each side of the pylon and that the vertical load component was transferred effectively into the pylon through the saddles.

Figure 17: Typical Saddle Arrangement

Other elements of the cable system included the anti-vandalism tubes, wax-box system, dampers and deviators.

The deviators were provided at entry into the saddles and the anchorages in order to control the deviation angle of the strands from the bundle within the cable pipe into saddle and anchorages.

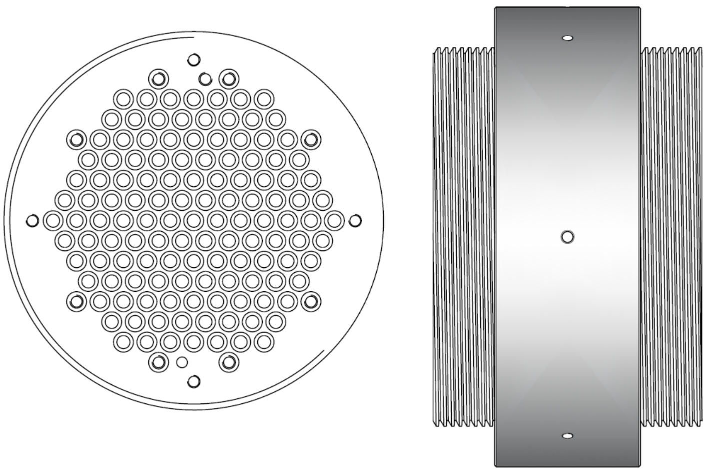


Figure 18: Typical Adjustable Anchorage Assembly

The dampers were provided to dampen the vibrations from the cable and prevent unwanted oscillations of the cable.

They were located within the steel form tube above deck level before the cable entered the deviator and anchorage.

The dampers were installed at the same time as the cables with the strands passing through the dampers.

The overall system was designed and tested in accordance with FIB Bulletin 30 and project specific, full scale, fatigue and tensile tests were carried out at laboratories in Chicago and Milan. An important aspect of the saddle test was the determination of the friction coefficient, noting that the system relies on friction to carry any imbalance in loads on either side of the pylon.

Furthermore, all the components of the system were also the subject of production testing to ensure that the cable system as whole meets the required standard.

HIGH STRENGTH CONCRETE

As outlined earlier, and in further detail in the Arup & CFC article, the design for the bridge deck required different concrete mixes with varying compressive strength requirements.

While much of the approach spans comprised of concrete with a compressive strength classification of C50/60, the main spans predominantly comprised of C60/70 and C80/95.

These compressive strength requirements posed unique challenges in the design and specification of the concrete mixes.

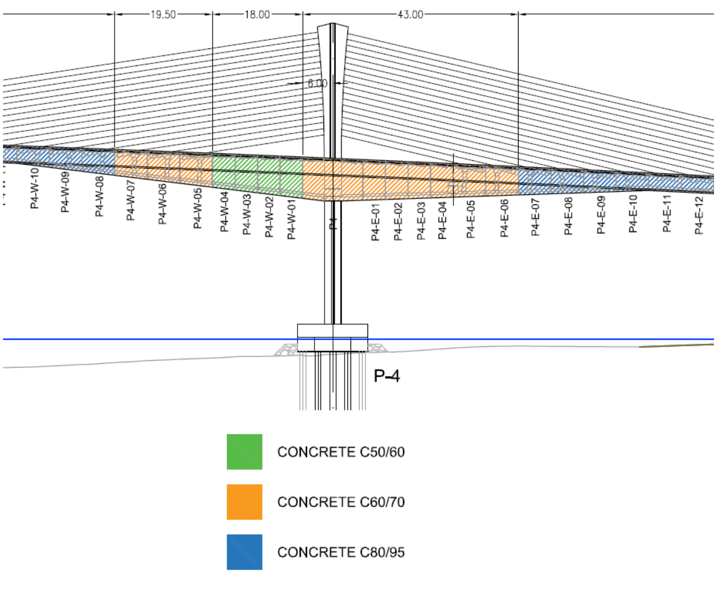


Figure 19: Concrete Grade Specification

Click on the image to see it in full

In the early stages of the project, NRJV worked with our concrete supplier to prepare trail mixes. Several important factors had to be considered in the mix design including:

- Compressive strength performance
- Early strength gain with a particular focus on the minimum strength requirements to launch the traveller systems
- Workability of the concrete to achieve the required level of compaction and surface finish
- Pumpability of the concrete, noting the long distances that concrete would be pumped up to 115m up the piers and over the river spans
- Aesthetical appearance, in particular in relation to consistent colour

H4A MEDIAN CONCRETE BARRIER

One of the predominant driver safety features on the bridge is the central median barrier and in particular the high containment barrier adjacent to the bridge stay cables.

On the approach to the bridge from both the east and west and indeed on the approach spans, the eastbound and westbound carriageways are separated by a slip formed concrete barrier with a containment classification of H2.

However, in accordance with the contract requirements, a higher containment classification of H4A was provided adjacent to the stay cables.

Notwithstanding the fact that the overall bridge design can accommodate 1 redundant stay cable, the higher containment barrier increases the level of protection afforded to the bridge structure.

Following a review of the various median concrete barrier systems, NRJV selected the Linetech LT104H4B barrier system.

The barrier system had previously been EN1317 crash tested up to a containment class of H4B. As the project requirements were slightly less onerous in that H4A containment was required, a simulation was undertaken by a specialist consultant to analyse the performance of the barrier system under a H4A test.

The simulation considered the performance of the barrier for a 30t vehicle travelling at 65km/hr with an approach angle of 20 degrees, all as required by EN1317.

As illustrated in Figure 20, all aspects of the barrier system were part of the simulation model including the exact profile, the properties of the concrete and the reinforcing steel.

Following the successful completion of the simulation, the barrier system was procured and constructed by NRJV under licence.

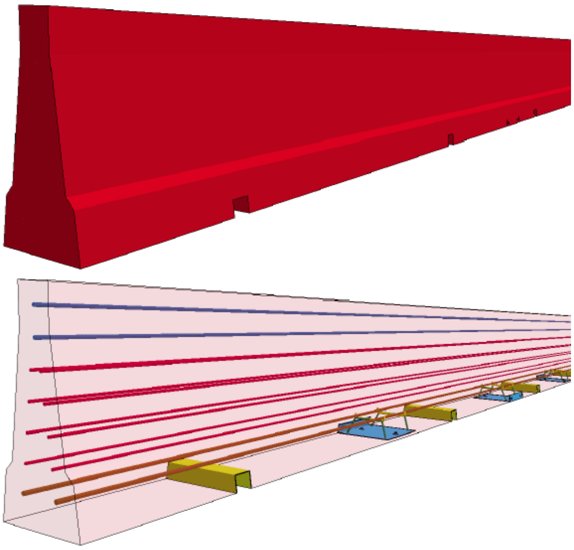


Figure 20: Extracts from Simulation Model

The barrier installation was also CE marked by an independent certified testing company.

In total approximately 1,220m of the Lintec barrier were cast. The concrete mix specification was a C32/40, however in order to meet the exposure classification of XD3, a C35/45 mix design was proposed.

Figure 21: Barrier Slip-forming Operation

Furthermore, and unlike the standard H2 concrete barrier used on the road network throughout Ireland, the LT104 H4b barrier is reinforced with high tensile steel with welded joints. EN ISO 9606-1 was used for qualification testing of the welders.

ARCHITECTURAL & AVIATION LIGHTING

One of the most prominent features of the Rose Fitzgerald Kennedy Bridge is the architectural and aviation lighting.

The primary objectives of the architectural lighting were to:

1. Provide the structure with an iconic appearance by night
2. Illuminate the stay cables
3. Illuminate the pylons
4. Illuminate the parapet edge beam

A specialist architectural lighting consultant was engaged and various options considered.

The design of the lighting had to take cognisance of various factors, including:

- Who can see the bridge and from what vantage points
- Developing a lighting solution that deliver the objectives, enhance the bridge structure at night but at the same time be sympathetic to the rural location
- Ensuring safe levels of glare for road users
- Avoiding unwanted light spillage
- Power consumption
- Anti-vandalism measures
- Durability and maintenance

A 3D render model was produced to simulate the proposed lighting performance and to ensure that the final design selected delivered on the primary objectives and indeed the design factors outlined above.

In advance of finalising the design, a site trial was also undertaken to both validate the render model and also to gain a full appreciation of the proposals.

The final design comprised of:

- A pair of cannon lights, on the deck at each stay cable each with an illumination intensity of approximately 70,000cd.

These lights provided the illumination required for the stay cables. The cannons were also orientated and angled to align with the cables and further illuminated the pylon in the distance.

- A continuous strip light on both parapets for the entire length of the bridge with an illumination intensity of approximately 4cd/m2. This strip lighting was housed in a bespoke aluminium extrusion as illustrated in Figures 21 and 22. The extrusion was slightly offset from the concrete parapet with the strip lighting shining downwards, in effect providing an indirect light source.

The aviation lighting at the top of the 3 pylons is also a predominant night-time feature.

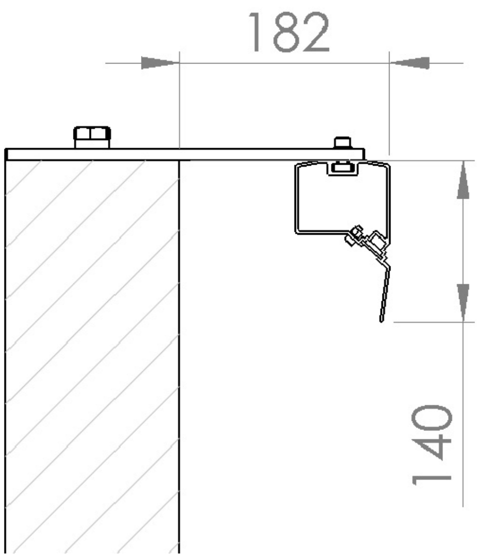


Figure 23: Strip Lighting Extrusion Housing and Support Bracket

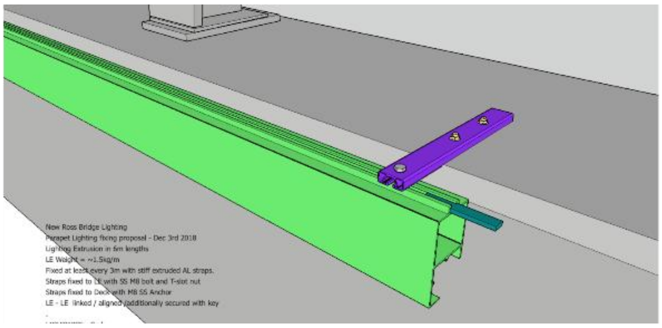


Figure 22: Aluminium Extrusion for Strip Lighting

Following consultation with the Irish Aviation Authority, aviation lights were procured and installed to the following specification;

- A red continuous (steady) light
- Intensity of 2,000cd
- Infra-red emission to aid pilots using night vision equipment

Figure 24: Architectural & Aviation Lighting

THE OPERATIONAL & MAINTENANCE PHASE

As outlined earlier, the project was awarded as a PPP Contract with a 25 year concession period with an operation and maintenance contract running from the end of the construction phase to 2045. With this in mind, whole life costing was very much the focus of the design and construction phase.

Early in the design stage, an inspection and maintenance strategy was developed. The purpose of this was to identify opportunities associated with inspections and maintenance with subsequent positive intervention in the design phase. This strategy included:

- Access methodology for the central box, the adjacent wings and the abutment galleries
- Bearing inspection, repair and replacement
- Stay cable system inspection and replacement
- Replacement methodology for the fire protection system

Ventilation of the box and wings was also one of the primary considerations of the strategy. The 20mm joint spacing between the precast panels provided the required ventilation in the wings. To negate the need for mechanical ventilation in the central box, ventilation holes were included in the design.

Access and egress through the abutment galleries were also a primary focus in the early stages of the design. All post construction access for routine inspections, repairs or even cable replacement in years to come would be solely through the abutment galleries. To verify and validate the design assumptions, a site trial was executed whereby the gallery access was replicated with a timber frame.

Figure 25: Bearing Access Site Trial

Access and egress checks were then undertaken including a mock emergency evacuation by the emergency services.

Similar site trials were also undertaken to assess the access arrangements for the inspection and potential replacement of the bearings as illustrated in Figure 25.

CONCLUSION

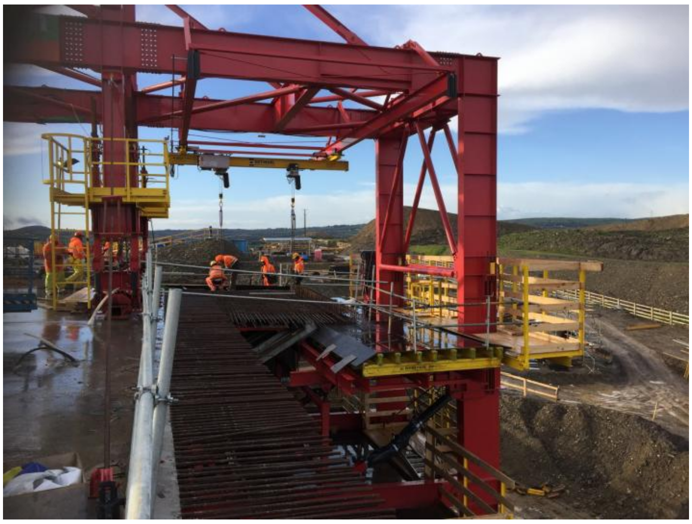
The design and construction of the Rose Fitzgerald Kennedy Bridge has earmarked a milestone for bridge engineering in Ireland.

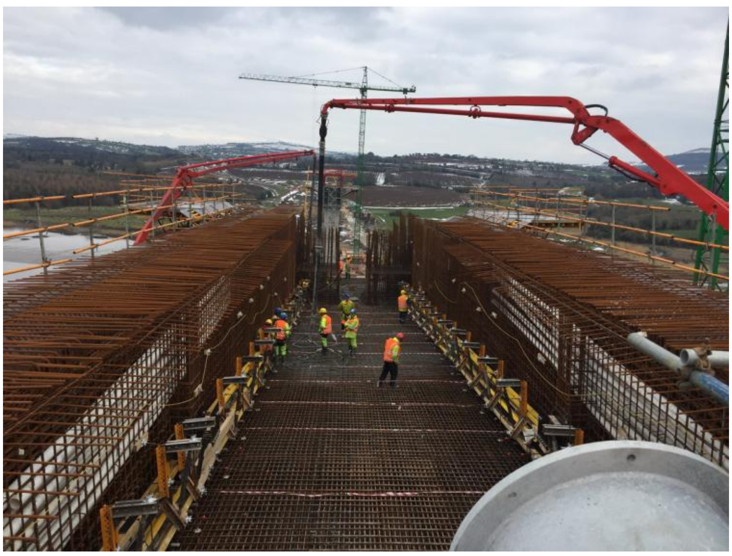
In January 2020 the townland of New Ross had a new Barrow bridge, one which would alleviate the local traffic congestion, reduce journey times on the national N25 and N30 routes and moreover, one that would set a new record for extradosed bridge construction.

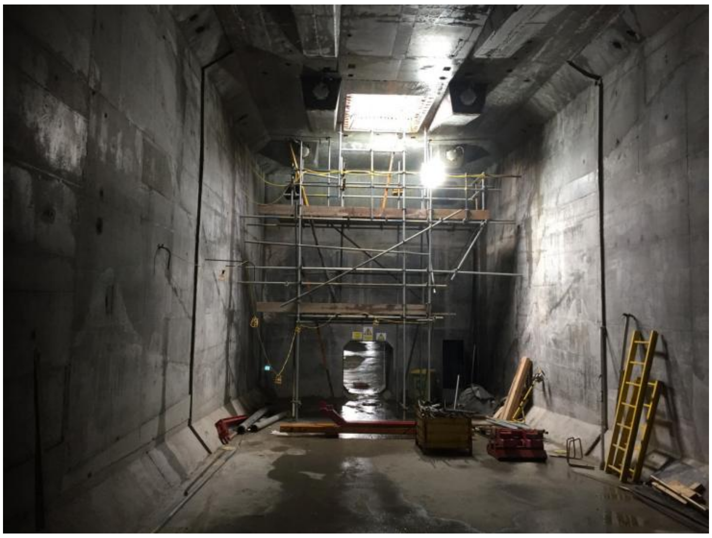
BAM, Dragados, BAM PPP and Iridium have played a vital role in delivering this piece of iconic infrastructure.

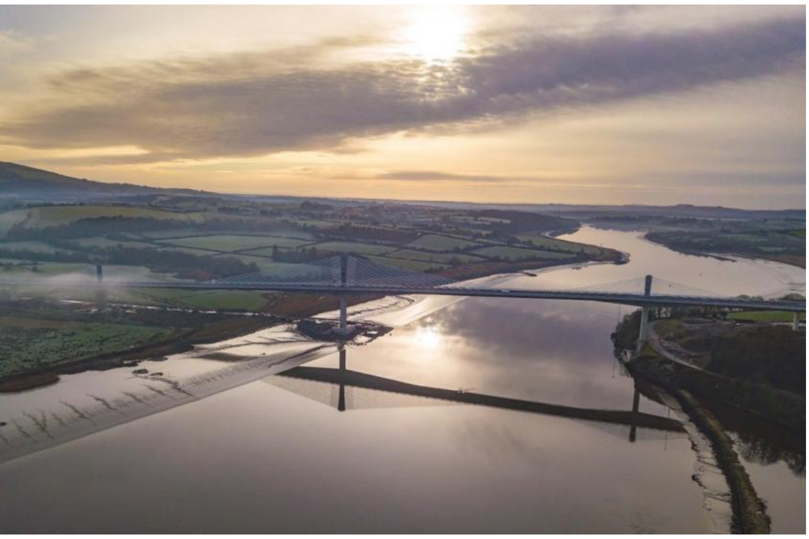
↓ *Figure 26: The Completed Rose Fitzgerald Kennedy Bridge*

CONSTRUCTION PHOTOS





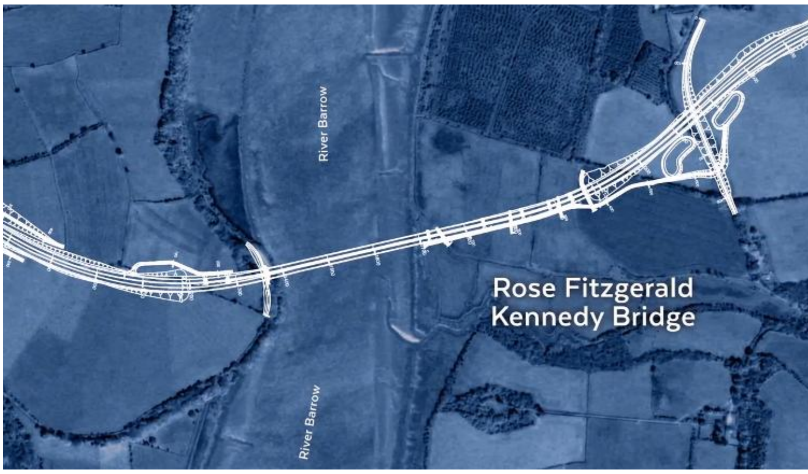




Photos Credit: BAM PPP

VIDEOS

Video 1: Aerial Video of N25 New Ross Bypass PPP Scheme



Video 2: N25 Presentation Progress September 2019

Video 3: The Rose Fitzgerald Kennedy Bridge Opening Ceremony

Virtual Reality Video: 360° View of the Bridge

This 360° video is best viewed on your mobile phone using the YouTube App and a cardboard viewer. You must use the YouTube app to get a fully immersive experience.
- Load the YouTube app on your phone
- Search for this 360° video in YouTube using search tag '#N25VR'
- Press Play to start the video
- Select the 'goggles' icon to show the split screen view
- Place your mobile phone in your viewer
- Look around and enjoy a truly immersive experience

Desktop users can navigate this 360° experience by using their mouse cursor to 'drag' around the screen.

Credit: Wexford County Council

Click on the image to play the video

WING AND FORMWORK TRAVELLERS

FOR THE ROSE FITZGERALD KENNEDY BRIDGE

Santiago Andrés Sales, Project Coordinator ´New Ross´, MSc Civil Engineer, Rúbrica Engineering

Figure 1: N25 Bypass over River Barrow

GEOGRAPHICAL AND DEMOGRAPHICAL SITUATION

The project is located in South Eastern Ireland which is typically rural in nature with local roads connecting towns and local communities.

With increasing traffic serving the growing town of Wexford, and with the plan to upgrading strategic connections westwards to Waterford and Cork, an improvement of the N25 at New Ross offers major benefits.

The major geographic obstacle on the preferred route comprises the tidal River Barrow which serves the town's historic harbour.

The river is about 200km long and 200m wide, with the southernmost fixed crossing being the old O'Hanrahan bridge in the town centre. The two lane bridge creates severe congestion with typical delays of up to 30 minutes at peak times.

To improve the situation, the Transport Infrastructure Ireland (formerly the National Roads Authority) approved a bypass to the town of New Ross to be built about five kilometres downstream from the O'Hanrahan bridge.

At this particular location the River Barrow is 312m wide and is still an important navigation channel, so the chosen bridge arrangement was selected as described in preceding articles in this issue of e-mosty.

GENERAL DESCRIPTION OF THE BRIDGE

The Bridge over the Barrow River solution, taking into account the constraints described previously, is a concrete extrados bridge with 2 main spans of 230m. These are extended from a central pylon 65m high.

The complete crossing consists of 8 piers and 2 abutments joined by a road almost 1km long that makes this bridge the longest in Ireland and the longest of its kind in the entire world.

DESCRIPTION OF THE SOLUTION ADOPTED AND SCOPE OF THE SUPPLY

The main reason for the award of the Travellers contract to Rúbrica Engineering was the operation of the system, coupled with price and delivery time.

Rúbrica has full commitment with its solution, guaranteeing the requirements by a constant communication with the designers and the construction company as well as constant support during the works duration.

The contract required 2 types of equipment, the Wing Traveller and the Formwork Traveller, both of which are specialisms provided by Rúbrica.

The solution adopted included the design, manufacturing and supply of each equipment set, including all documentation required complying with EU and Irish regulations.

Moreover, as the JV specified plywood formwork sheets (rather than using steel formwork), Rúbrica also coordinated with the JV to enable the installation of Plywood in our designs, giving an optimised solution for its installation and use.

One of the main challenges of this project was the design of the 'Inner form' system for the Main deck. The complication was to fit all the forms inside the cellular deck and enable a system capable of being moved "automatized" and without clashing with the different parts of the deck (Post-tensioning anchorage blocks, Stay cable anchorages and Delta frames between other elements). The solution adopted satisfied all criteria required.

Figure 2: Traveller in P5 side during construction

MAIN SPAN BRIDGE

The bridge had a series of conditions that had to be met during the design phase. These conditions were essential for the award of the contract. The list of main requirements is as follows:

- Variable Transverse cross-fall.
- Variable Longitudinal gradient.
- Variable height segments.
- Variable length segments.
- Limited weight of the equipment to ensure dimension control of the deck.
- Access to previous segments for finishing works.
- Precast concrete strut positioning and pouring together with main segment.
- One phase pouring and symmetrical works operations.

Geometrically speaking, the main span is a single box structure combined with strut supported wings.

The length of the segments varies from 4.5m for the first 8 segments (varying from 8.5m in height to 7m), then progressing to 26 segments of 6.5m length (varying from 7m to 3.5m) and 2 segments of 5m length that are the final segments prior the closing segment, of 3m length.

Segment 0 (zero), which was a hammerhead 12m in length, was executed prior the installation of the travellers at the top.

This served as the starter segment for the Form Travellers to be supported and begin its run (using a special connections structure to enable the balanced pouring between sides).

The longitudinal and transverse slopes, as well as the changing height of each segment, added difficulty to the design, as the pieces that formed the Form Travellers had to be adaptable for these conditions.

The panel configuration in each segment was different until reaching the constant height segments in the middle of the spans.

These along with the placement of the precast concrete struts (3 or 4 pieces each side per segment, with the added difficulty of the struts comprising slabs, which necessitated precise positioning), made the operation of the traveller a challenge in itself.

It is also necessary to consider that these changes in configuration had to be done approximately 50m above ground or water without any support from below, so the operation of the Formwork Sets were quite a challenge.

Figure 3: Constant depth main deck, P3-P4 span

Due to the described difficulties the solution adopted for the form traveller was an upper cantilever system where the lateral web panels and the bottom slab formwork could be reduced as the segment gradually became shorter in height.

The solution for placing the struts was to provide a supporting fix structure capable of adapting to the different horizontal radius, where the slabs were placed using gantries with remote control.

The transmission of loads to the deck was through anchor bars for the hanging points (and negative rear forces ie uplift) to the deck.

The front support was designed with locking nut jack, a double system that Rúbrica Engineering uses for high transverse and longitudinal slopes on decks to ensure stability and safety.

During the 'advance' phase the weight of the upper structure was transmitted to the upper deck via a set of rails used to move the structure forward.

The weight of the bottom slab formwork was transmitted to the web forms and those, at the same time were hanging from the upper structure.

This solution was adopted as a special measure due to the access requested by the JV.

Once the main structure of the travellers was defined, the operational and functional aspects of the system needed definition. In this phase the hydraulic systems, the forms, safety elements, accesses and working platforms were defined.

Description of the solution adopted for Main span

For the construction of the main spans of the bridge four Form Travellers were supplied, one assembled on P3 and P5, and a pair of travellers assembled on P4.

Each traveller has four clearly identified parts that composed the complete set of equipment.

This comprises:

- The Upper Structure;
- Bottom Structure;
- External Form Structure;
- Inner Form structure.

The decision made for the upper slung structure was mainly due to the price and adaptability to the different heights. These travellers are capable to support segments exerting a moment of 410Tm, and a maximum length of 6.5m, which are one of the biggest structures of its kind.

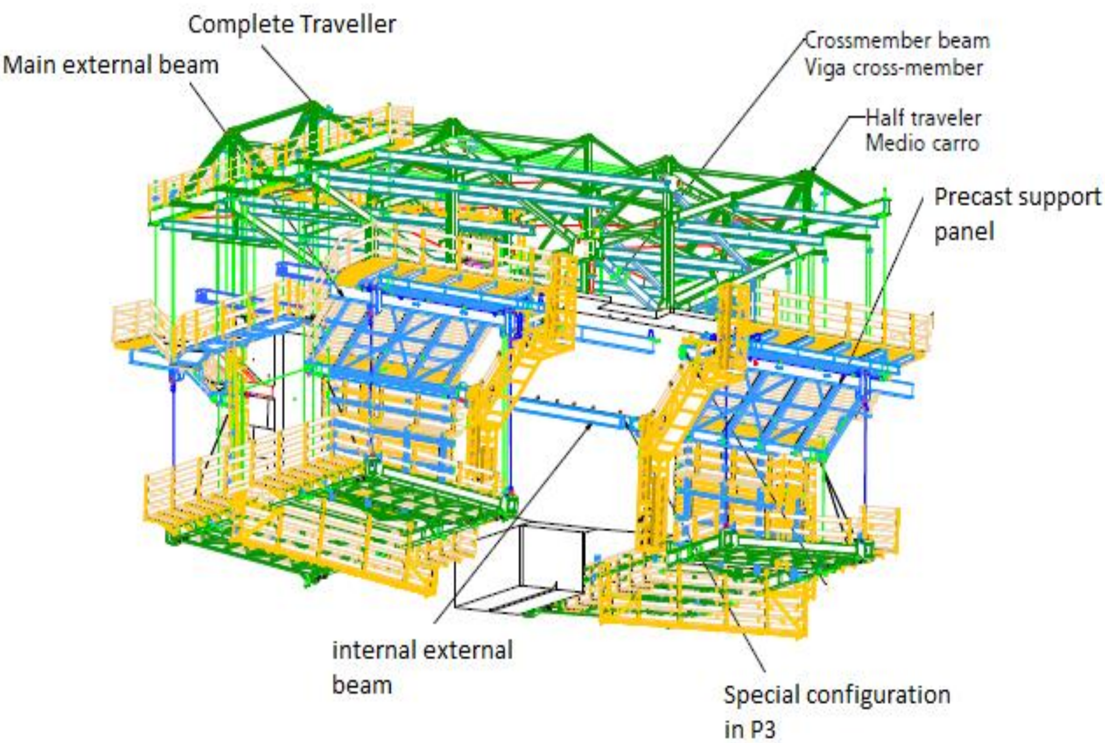


Figure 4: General description for pier table assembly at P4

Upper Structure

The function of the Upper Structure is holding together the rest of the associated parts of the system, transmitting the loads to the upper deck, manage the position and precamber, stripping the forms and carrying the structure during the 'advance' phase.

The Upper Structure's main job consists in supporting half of the freshly poured concrete weight of the segment when casting and transmitting it to the upper deck.

The load of other parts was transferred to the upper structure through a built-up steel member located at the front of the cantilever.

This huge load, corresponding to the structure´s self-weight and the weight of the freshly poured concrete was then transferred through the main trusses to the rear support where 4 high-strength bars (8 in total) guaranteed the equilibrium. Each bar was prestressed to 620kN.

The necessity of such a high pre-stress force is governed by the need of the furthest point of the traveller to remain fixed even when the loads of all the elements were applied so the deflection on the next segment could be controlled, as well as a safety measure, because the bars are checked with the pre-stressing prior the loading.

In reality, the fresh concrete reduces the pre-stressing value, being the critical situation for the bars during pre-stressing work.

The advance of the structure was performed via hydraulic jacks that moved the structure over a rail system firmly anchored to the deck, so that when the forms were detached from the previously cast segment and in an 'advance 'position, the uplift reaction at the rear supports of the upper structure is transferred to the deck through the rails due to the position of the centre of gravity.

External web forms

This part of the traveller was composed of a series of beams that make the "skeleton" to support the precast concrete struts and the web panels.

This structure had the job of providing support to the bottom formwork during the advancing stage. This was done through the built-up suspension beams. This form moved simultaneously with the upper structure as the latter moved to the next position. It moved below the deck.

From the Rear Platform of this part the advance movement was performed, as the upper structure of the traveller hydraulically moved, this form had to accompany the movement from below the deck.

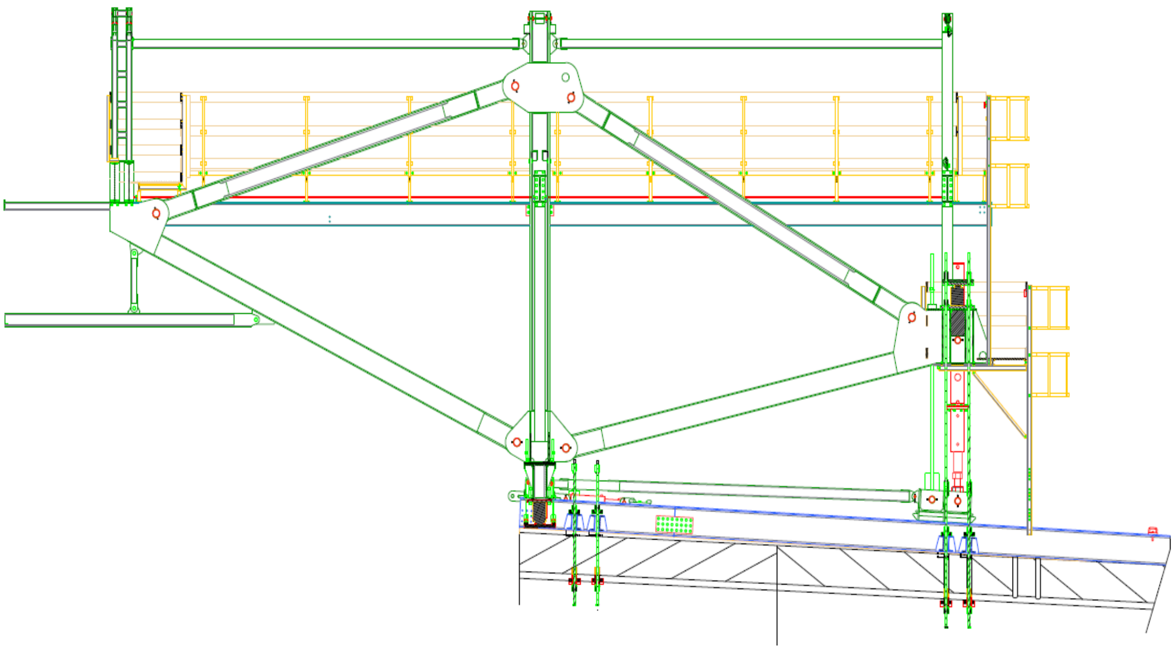


Figure 5: Scheme of the Upper Structure

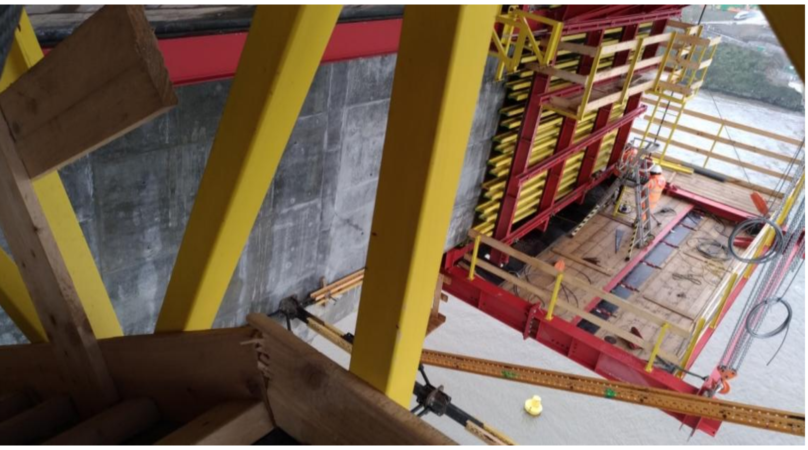


Figure 6: View of bottom platform from the access platform of the opposite traveller

Figure 7: Inner form view from external form

Additionally, it was requested to have access to the already performed segment, and the beam hanging from the bottom structure was also used as rail system for the movable access platform, designed for the finishing works.

Bottom slab forms

This is the main resistive part of the system along with the upper structure, as it holds most of the weight during the pouring phases.

A series of huge steel profiles positioned meticulously allowed for the distribution of the weight to 2 No. M64 threaded bars anchored to the previous segment with a pre-stressing force of 1500kN each.

The bottom platform was capable of being adjusted in height to adapt to the variability of the deck.

The hanging system incorporates a "safety fail" approach, and all elements were duplicated to ensure the safety of workers.

Inner forms

Due to the cross section of the bridge, the Inner form had to be divided into 3 parts; one for the central core section and 2 No. forms situated between the precast strut and the wing, which - due to their form - were called triangular forms.

The inner core forms consisted of 2 suspension beams hanging from 2 hangers; the front and the back hangers.

The front hanger transmitted the load to the main front beam in the upper structure as the rear hanger was attached to the previously constructed segment.

These 2 double armoured beams over 12m long held the whole weight of the inner core forms: the web panels and the forms between them which held the anchor block for the post-tensioned cables of the bridge.

The beams could move horizontally and vertically to allow the forms to strip and fold to advance to the next pouring position.

As the segment became shallower, changes in the configuration of the panels had to be made making the advance movement more challenging due to the congestion at the end.

The triangular forms were laid over the precast concrete struts.

They were formed by a main beam that transmitted the load to the main front beam in the upper structure and to the previously constructed segment in the back.

From these main beams 2 extensible panels were welded, one in the roof and one in the web that formed the triangle.

These extensible panels, hydraulically operated, allowed the stripping of the forms and, the vertical section on the webs, gave room for a system of foldable wheels to deploy and advance.

SECONDARY SPANS BRIDGE

In the secondary spans the conditions to be met were less restrictive in terms of access and loads, but equally challenging as the variability was higher and the cycle required was shorter, namely:

- Variable longitudinal gradient
- Variable transverse slope
- Variable width
- Precast concrete strut positioning (variable due to previous variable parameters)
- High cycle (less than 5 days per launching)
- Avoid manual adjustment on each segment

In this case, as the secondary spans were inland, the core of the section was done with falsework, but, due to the change of width of the section along the longitudinal axis (from 19m to 22.5m) along with the slopes already mentioned and a steep terrain, the solution proposed was a form traveller that could cast only the wings of the cross section, thus the name Wing Traveller.

The Wing Traveller run was composed of 36+45+95m from Pier 3 to Abutment 1 and 36+50+70+95m from Pier 5 to Abutment 2.

A starting cantilever of 20cm was needed for the kickstart of the formworks as the sealing must be done to a solid element.

The formwork had to adapt in each segment to a changing transverse crossfall (from 0% to 5%) and longitudinal gradient (from -5% to -0.2%) so to solve this variability the system necessitated a hydraulic adjustment of height and slope to meet the conditions, which were different after each launching.

The transmission of the loads from the concrete to the previously constructed box deck was through two hanging anchors, placed in the joint between the precast strut support and the bottom slab combined with the launching frame itself laying over the concrete/rail (it was designed as a system for changing the support from deck to rail and vice versa) which also serves as the advance system. In the advance situation the loads will go directly from the legs of the main frames to the rails and into the deck without need for anchorages, operating as a cantilever for all wings of the system.

Description of the solution adopted for Secondary span

For the construction of the wings of almost 500m in length only one Wing Traveller was supplied. The structure could be divided into number of elements: Main Frame, Vertical guides and Bottom casting platforms.

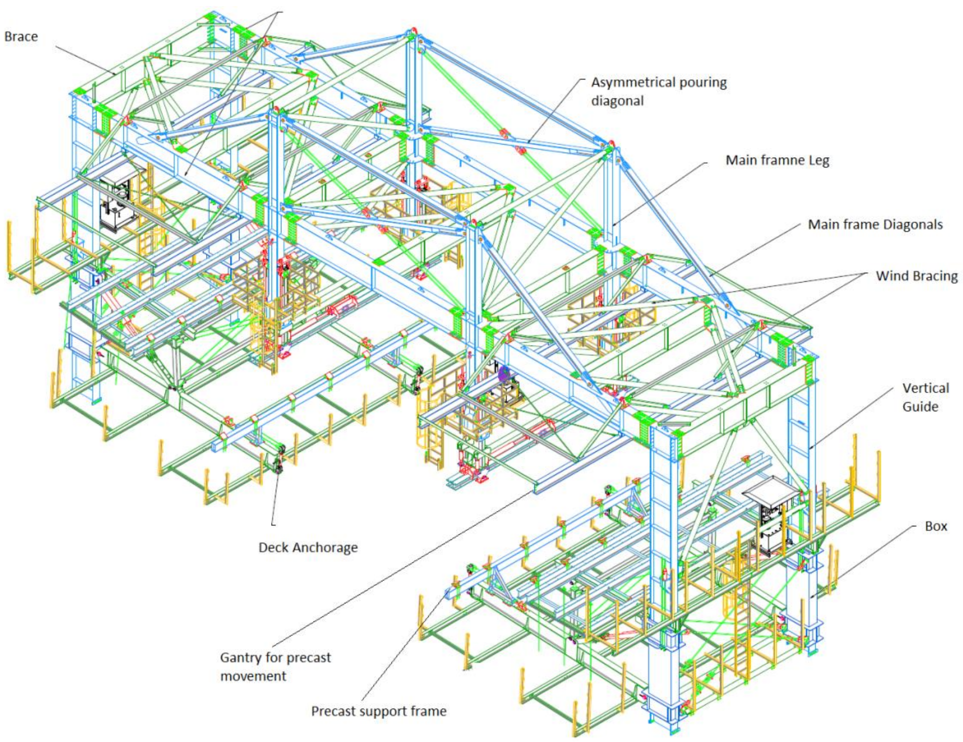


Figure 8: Wing Traveller general layout

The solution designed enables the passage of vehicles underneath the structure, as well as picking up precast elements for positioning.

Main Frame

The main frame transmitted the forces carried by the bottom platform and the vertical guides to the deck core via 4 legs over a rail that acted both as a load spreader and as a guide during the advance.

The 4 legs that composed the supports are independent and operated via high pressure hydraulics.

The system was conceived to maintain the main frame of the Wing traveller in a horizontal position at all times regardless of the transverse or longitudinal slope that could occur.

A series of beams were installed in the lateral frames to allow gantry cranes to position the precast concrete struts, as well as being able to pick them on the back section of the traveller.

Vertical guides

These guides, as the name suggests, are vertical movable props that adapt the height of the bottom platform to the casting segment to adapt each lateral to the correct height due to the variable transverse slope.

They are completely independent from each other and are hydraulically operated, being easy and quick to position.

Bottom casting platforms

This platform carries the greatest weight of both the fresh concrete and the precast props.

It is formed by 2 main frames that are anchored to the core deck via 2 push-pull props and to the vertical guides.

Using these frames the corbel connection to support the precast concrete struts is formed. The frames also support the movable formwork used to form the cantilever wing similar to the arrangement used on the Form Travellers.

Having the precast concrete strut was a handicap because the two triangular inner formwork sections had to be designed to support the wing part of the deck above the precast slabs.

Figure 9: Wing Traveller

PRESTRESSING TECHNOLOGIES
OF THE ROSE FITZGERALD KENNEDY BRIDGE

Cosimo Longo, Andrea Castiglioni di Caronno, Tommaso Ciccone, TENSA

Figure 1 – Main spans of Rose Fitzgerald Kennedy Bridge over River Barrow

INTRODUCTION

As the latest state-of-the-art Irish river crossing, with a new world record set for the longest extradosed concrete spans, the Rose Fitzgerald Kennedy Bridge relies on an extensive use of both parallel strand stay cable (SC) and internal bonded post-tensioning (PT) systems installed by Tensa.

For the PT systems, three families of CE (Conformité Européene) marked products were used:

- Multi-strand type 27MTAI and 12MTAI, for longitudinal, cantilever/cap and continuity tendons;
- Multi-strand flat type 5PTSE, for transverse tendons aimed to resist spalling forces over SC blisters; and
- Threaded bars type 47WR, to allow segmental launching of the form-travellers (FT).

A total of thirty-four continuous SCs were installed: eight type 113TSR in the two short pylons and eighteen in the main central pylon, type 113TSR up to the sixth cable from the bottom and type 127TSR for the rest.

Each SC was equipped with adjustable anchorages and a multi-tube saddle type TSS-T,

Figures 2 and 3: Multi-tube saddles assembly for lifting and installation

an innovative structural component which allows each strand of the bundle to run into a dedicated pipe and deviate continuously through the pylon.

By using this technology, no traditional anchoring of a SC was needed at the pylons, since the saddle's friction was sufficient to avoid strand slippages and resist the unbalanced forces transmitted from both sides.

TESTING

According to the Project Specification (PS) issued by the National Authority - Transport Infrastructure Ireland (TII) - no pre-qualification tests were required for ETA (European Technical Assessments) approved PT systems, as well as for the materials produced and supplied under the UK CARES certification scheme which, by the way, also regulated the qualification of the personnel involved in the PT works.

On the other hand, all the PT provisions and site operations were requested to be detailed in advance in a PT method statement (MS), covering proposed materials, personnel, equipment and quality controls as well as arrangements for storing, ducts installation, strands threading, stressing and grouting.

The MS was first assessed, then formally approved after inspecting the results of a dedicated on-site full-scale trial, with special reference to grout mix and injection operations, in order to demonstrate that the proposed procedures ensured proper embedment and protection of the prestressing steel.

In more detail, the full-scale trial consisted of testing a representative 50 metre long RC beam sample provided with a draped PT tendon.

First, anchorages, ducts and grouting accessories were assembled according to the proposed MS, then proved to be air-tight as demanded by the PS.

Subsequently, threading and stressing operations were simulated with the equipment intended to be used for site works.

Finally, the PT tendon was injected and inspected after three days, by reviewing the grout filling in several transverse sections cut along the profile.

Regarding the SC system qualification, a full-scale testing campaign was carried out to prove its suitability for the use on the Project.

A tensile fatigue and static test over a 127-strand SC was carried out at CTL, Illinois, USA, and a tensile fatigue and static test over a 37-strand SC system equipped with a multi-tube saddle, equivalent to the one to be used in the project, was performed at LPM of Politecnico di Milano, Italy.

Both tests successfully met acceptance criteria demanded by PS and Fib bulletin 30, proving the high performance of the SC system in terms of endurance, as well as in terms of static efficiency and ductility at ultimate load.

Figure 4: Tensile fatigue and static test set-up over 127-strand SC – CTL, Illinois - USA

Figure 5: Tensile fatigue and static test set-up over a multi-tube saddle for 37-strand SC – LPM, Politecnico di Milano - Italy

In order to confirm the saddle friction capacity, a dedicated testing rig was assembled in the TENSA facilities, simulating a saddle with the same configuration of the one to be used on site.

Friction tests were performed varying the initial prestressing level in the strand.

Several trials were also requested for SC works before the installation could commence.

A first site trial was performed to evaluate the procedure for butt-welding HDPE pipe sections to be used for SC pipes (OD 315 mm) and telescopic pipes (OD 450 mm), respectively.

A total of three welds were done for each pipe diameter. Welds were proved to achieve and exceed the full yield strength of the pipe section, by performing tensile tests at the LPM of Politecnico di Milano, Italy, over three machined 25 mm wide and 300 mm long samples.

Further trials regarded anchorages and accessories injections, with the aim to prove complete wax filling.

For the situation where no vents at the highest points were allowed on site, an improved procedure was proposed and accepted.

After wax set, the specimens were inspected and found completely filled without any leakage.

The wax mass was homogeneous, and no voids or defects were observed.

INSTALLATION

Regarding the site activities, the installation works of the prestressing systems needed to fit the bridge deck construction schedule, which was split in several phases activated and run at the same time.

The access spans S1, S2, S3.1 + S3.2 and S6.1 + S6.2 , S7, S8, S9 were sequentially built by full span cast-in-place method on gantry shoring, while the two main spans - S4 and S5 - were built by cast-in-place segments by means of the Form Travellers: two working in balanced cantilever from P4 (S4.2 and S5.1) - for a total of 23+23 segments - two spanning from P3 (S4.1) and P5 (S5.2) - for a total of 13 segments each.

The design of the access spans was optimized to include longitudinal sets of 27-strand internal bonded PT tendons only, having typical draped profiles, anchored on blisters and staggered to stitch up the construction joints.

Some of the longer spans required additional continuity cap tendons of the same type.

Stressing operations followed a standard scheme on each span: once the concrete strength was achieved, a single end stressing of some of the tendons was carried out in conjunction with the gantry shoring supports releasing.

After completion of the wings and the subsequent span, the remaining tendons were stressed from both ends.

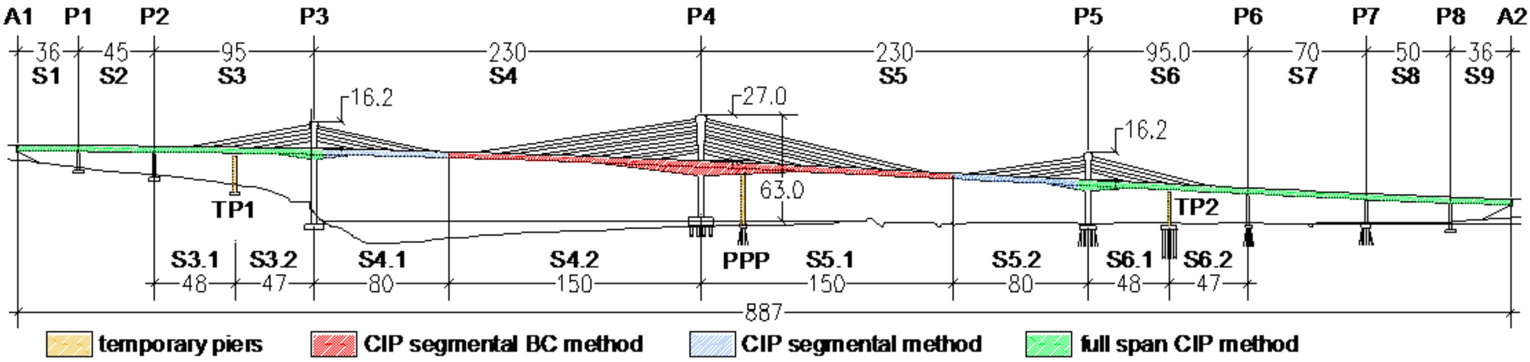


Figure 6: RFK Bridge general construction scheme

Finally, the tendons stressed in the first phase were restressed from the second end.

Some further prestressing provisions were needed in S3 and S6, since these were designed to work as back-spans for the cantilever construction of S4.1 and S5.2, respectively.

In more detail, due to their remarkable length, S3 and S6 were built into two stages, i.e. S3.1 + S3.2 and S6.1 + S6.2, by means of temporary reinforced concrete piers, TP1 between P2 and P3 and TP2 between P5 and P6.

The main features of S3 and S6 with reference to the prestressing technologies were the cantilever cap tendons sets - to be continued and stressed in the following construction phases - and the form tubes, each provided with groups of ten 5-strand transverse flat PT tendons, arranged in the upper slab to prestress the SC blister and counteract its vertical force component.

The construction of the main spans required to install and stress sets of symmetric cantilever tendons at each FT cycle: specifically, type 12MTAI or 27MTAI on P4 - from the hammerhead up to Segment 6 - and type 27MTAI on P3 and P5 - running from the corresponding back-span up to Segment 6.

From Segment 5, symmetric pairs of PT bars type 47WR were installed in the top slab, coupled at each segment and stressed alternatively every two FT cycle from Segment 7, where no additional cantilever tendons were provided.

The stressing operations of cantilever tendons and bars allowed the FT launching.

A temporary push-pull prop (PPP) with unbonded vertical tendons type 27MTAI was erected next to pylon P4 to cope with unbalanced loading on the cantilevers.

Eighteen form tubes per side were installed from Segments 5 to 22 in P4; similarly, eight form tubes were installed from Segments 5 to 12 spanning in P3 and P5.

In general, about 36 to 48 hours after pouring, the cantilever tendons or the PT bars in the top slab of segment “N” were stressed.

Then, the FT was launched and locked in the new position on Segment “N+1”, ready for the installation of bottom slab and webs reinforcements.

At this stage, a PT team joined steel fixers for installing PT ducts in the new segment.

Concurrently, a second team dealt with the stressing of transverse tendons on Segment “N-1”, to prestress the blister of the SC to be installed.

Soon after, the SC installation started. Depending on the cable length, this phase took three days in average to be completed while other works kept going on the FT, such as installation of lateral precast panels, fixing of reinforcement in the top slab, placing of transversal ducts and strands threading before pouring.

After SC installation and FT assembly completion, the Segment “N+1” was poured closing a typical construction cycle.

Afterward, out of the FT cycle, PT tendons were grouted to provide permanent protection.

Figure 7: PT works on a typical access span

Figure 8: A view of RFK Bridge under construction during cantilever working phases on P3 and P4

The SC installation and its coordination with the form traveller activity were undoubtedly the most outstanding, and challenging, aspects related to the bridge erection.

To fit the tight schedule of the segmental construction, some preparation activities, just like strand cutting and stay pipe welding, were moved up. For SC works in P3 and P5, the accessible back-spans S3 and S6 allowed strands to be prepared directly over the bridge deck most of the time, by uncoiling, cutting and laying them on long trays.

The cutting length of strands was defined based on the actual distance between the bearing plate, at deck level and the saddle exit, measured through a topographic survey.

Then, the HDPE coating of each strand was removed over three specific portions: at both ends, to allow wedge gripping, and in the central part passing through the saddle.

Preparation activities were usually performed by the SC team while not involved in the installation. For SC works in P4, the bridge deck was too short and too crowded to prepare strands safely and effectively.

Moreover, the FT cycle was quicker on this pylon and, subsequently, the available time window for preparation was not compatible with the pouring cycle.

Sometimes, even on P3 and P5 the schedule did not allow the strands to be prepared over the bridge deck.

Hence, a different solution was implemented: an additional team was full time dedicated on strands prefabrication in a specifically designated site area.

Basically, each strand was uncoiled on a bench, marked, uncoated where needed and then rolled up, without any intermediate cutting.

In this way, during installation, strands were threaded directly from the coils instead of from the storage trays.

If compared with the first method, the second one required a longer preparation time but allowed to bypass the critical path of the SC cycle.

In addition, with a correct planning of the activity, the preparation team provided coils in parallel with their installation, so to increase the installation rate.

Another preparation activity to be carried out before starting the SC installation was the welding of HDPE pipes and telescopic tubes with the relevant accessories.

HDPE pipes were supplied in 11.8 m long sections, to be butt-welded by means of dedicated welding machines in temporary protected shelters located at each pylon.

Figure 9: Strand preparation and site inspection

Figure 10: Stay pipe hoisting on P4

For the weld bead to develop the requested tensile strength, the operations had to be carried out strictly in compliance with the procedure assessed and approved in the site trial.

Parameters like welding time, welding temperature and contact pressure, needed to be carefully respected based on duct diameter and thickness.

Two pipes per SC were pre-assembled with the telescopic tubes and accessories, then moved on rollers over the deck while progressively welded.

The welding process usually started just before the FTs launching and lasted about one day.

The SC installation started once the written “load permit” was received.

The two pre-assembled pipes were hoisted one by one with a crane, from the end closer to the pylon, then secured with chain-blocks at a certain distance from the saddle exit.

At deck side, each pipe was pulled with a ‘tirfor’ toward the corresponding form tube.

To cope with the slack of stay pipes, mainly in the case of the long cables, the first strands to be installed were proportionally longer than the others.

The installation of cables equipped with a multi-tube saddle required a good coordination of the SC crew distributed as follows:

- two teams inside the box girder for the stressing operations, to be carried out at the same time from both anchorages;
- two teams over the deck, one at each form tube exit, for threading strands and passing them throughout the bottom deviator and form tube; one last team at the pylon, for pushing each strand in the corresponding saddle pipe and coordinating the stressing.

One of the teams over the deck operated the strand pushing machine, placed behind the form tube of the SC to be threaded, and fed with strands coming either from the trays or from the coils.

Each strand was pushed in the first pipe up to the pylon, then inserted in the corresponding hole of the saddle and pushed again into the second pipe.

When the strand reached the second form tube, on both ends tips were linked to coupling devices and pulled down from the anchorages all through the form tubes.

Once the correct length of strand tails was achieved on both anchorages and the pylon team confirmed the strand centring within the saddle, the stressing teams installed the wedge and commenced the tensioning with the balanced force stressing method, based on the use of mono-strand jacks and a load cells permanently installed on each anchorage.

Figure 11: Stressing of a continuity tendon

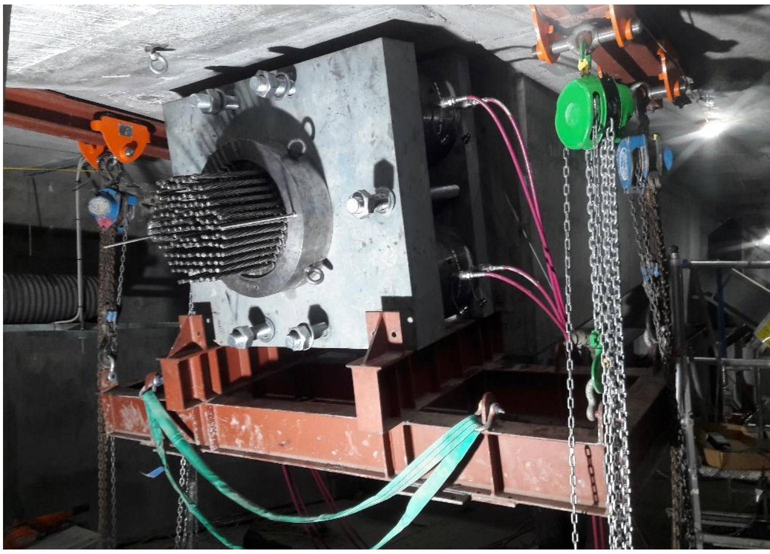


Figure 12: Multi-strand jack system

By progressively stressing each strand to the force pointed out by the load cells, a correct equalization of strand forces was achieved.

In this phase, tensioning operations were carried out in two successive steps to tackle with different load maps over the bridge deck: in the first (strand threading), the SCs were stressed to a target load of 40-50% the force to be provided at the end of the stressing phase and required for pouring the new segment.

In the second (strand tuning), the SC was stressed to the final force, i.e. 100% of the first stressing phase.

After joining the main spans and completing the stressing operations of the continuity tendons, a load adjustment was needed for all the SCs.

Typically, such an operation required to apply a small increment or decrement to the SC force by acting on the whole bundle, so to avoid any double biting of strands, as demanded by PS.

In case of restressing, whenever the elongation to be provided exceeded the wedge grip length, mono-strand jacks were used as already done during the SC installation.

Otherwise, whenever the elongation to be provided was smaller than the wedge grip length or even negative (destressing), the load adjustment was carried out with custom-made multi-strand jacks.

Basically, they consisted in two pre-assembled systems of four lifting jacks 500 ton each, used in conjunction with special lifting frames that allowed the jacks to be hoisted and installed in place.

A total of 27 SCs were adjusted in this way, acting either on both ends simultaneously or just on one anchorage per time.

Then, the SC finishing works took place, consisting of installation of fire protection blankets up to a vertical height of 2.5 m from the deck, setting up of anti-vandalism pipes, saddles and anchorages wax injection for permanent protection.

CONCLUSION

The RFK Bridge is an outstanding milestone for bridge engineering and sets a new world record for the longest extradosed concrete spans. Prestressing technologies played a key role for such an impressive achievement, which required important R&D efforts as well as detailed study of working procedure and accurate planning of site activities to cope with tight construction schedule and high quality standard for the works execution.

↖ *Figure 13: East spans and S.5.2 under construction*

↗ *Figure 14: Main spans under construction*

← *Figure 15: Main spans just before completing the closure pours*

Figure 16: RFK Bridge, SC and PT teams

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THE ROSE FITZGERALD KENNEDY BRIDGE

OVERVIEW DESIGN CONSTRUCTION TRAVELLERS PRESTRESSING