Preserving Ireland's iconic railway viaducts

1 Pádraig McCarron MSc, CEng, MIEI IntPE(IRE), MSc, DipAPM, IPMA-C, CEng, MStructE
Lead Civil Engineer, United Nations World Food Programme, Ethiopia; formerly Senior Engineer, Iarnród Éireann, Structural Design Section, Dublin, Ireland

2 Aidan McAdam CEng, MICE, MIEI
Senior Engineer, Iarnród Éireann, Structural Design Section, Dublin, Ireland

Ireland's national railway system operator Iarnród Éireann has developed innovative repair and strengthening techniques to retain, preserve and extend the longevity of several of its aging bridges. Of those, three nineteenth-century structures rehabilitated in the past decade stand out as iconic structures in their own setting and will now continue to provide service into the foreseeable future. Innovative bespoke engineering, three-dimensional finite-element modelling techniques and laboratory research and testing were used to confirm the theoretical hypothesis. This achieved a significant increase in the lifespan of these structures that would otherwise have been deemed life-expired. This paper gives an overview of each of the projects, describing the existing structure and the challenges overcome in their design, repair and rehabilitation.

1. Introduction

At the end of the 1990s Ireland's railway infrastructure was generally in need of upgrade, having received little in the way of major investment over the previous two decades. Led by a railway safety programme, the following years in Ireland resulted in substantial funds from the Irish government being invested into the railway infrastructure to approximately €1.6 billion.

Bridge works were a substantial part of this investment, with railway system operator Iarnród Éireann demanding improvements in the quality of the track support zone across many of the structures and the removal of historical speed restrictions.

Typically many of the larger viaducts had remained unchanged from initial construction, but an increase in the desire of the operator to satisfy the needs of its customers for a modern quality train service, with reduced journey times and improved ride quality meant that upgrade works would be necessary. As a result, the bridge infrastructure was now being subjected to higher loads, larger dynamic factors from increased train speeds and a higher frequency of train movements across the structures.

To facilitate these improvements, it would be important that the works would use innovative and smart solutions to achieve

the necessary upgrades, while causing the minimum disruption to normal railway services. The bridge strengthening designs presented here were in part an evolution of this process to develop innovative bridge strengthening solutions. Figure 1 shows the location plan for the viaducts.

2. Cahir viaduct

2.1 Introduction

Cahir viaduct carries the Limerick to Waterford railway over the River Suir at Cahir in Co. Tipperary. The viaduct comprises an 85.7 m (18.5 m, 48.7 m and 18.5 m) three-span continuous half-through wrought iron Fairbairn box girder viaduct, and originally had an open deck consisting of a series of wrought iron and steel transverse members supporting timber waybeams and track. The original viaduct was supported on unique and ornate masonry piers and abutments that were designed to complement the heritage in the nearby town of Cahir.

2.2 History

The Limerick to Waterford line was opened in stages from 1848 to 1854. Cahir viaduct, part of the Tipperary to Clonmel section, was constructed between 1851 and May 1853. The

Figure 1. Location map Ordnance Survey mapping © Crown copyright. Media 042/12

viaduct was originally designed to carry double track; however, this was later reduced to single track with a siding on the Waterford approach in 1931 (Cox and Gould, 1998).

The main girders were designed by William Fairbairn, who patented the design in 1846. The design used the box shape to stabilise the girders in bending and was adopted for two viaducts in Ireland and several in the UK, most notably the Britannia and Conwy Railway bridge in the UK (Fairbairn, 1849). The abutments and piers were constructed in limestone masonry, and complemented the ancient architecture in the town of Cahir and in particular the nearby Cahir Castle.

The viaduct had been impacted as a result of a previous railway accident, when in 1955 a locomotive pulling 32 beet wagons passed through the home signal in the station onto the siding on the Waterford end of the viaduct and crashed through the buffer stop and the deck falling to the river below. Figure 2 illustrates the accident. The inquiry into the accident could 'find no explanation for this occurrence other than the driver failed to keep the train under control and to obey the

signals'. The viaduct deck was repaired with steel girders, and train services resumed within 3 weeks of the incident (JIRRS, 1985).

Figure 2. Damage from 1955 accident

2.3 Project background

At 6:15 a.m. on 7 October 2003, a train comprising two locomotives and 22 cement wagons derailed as it passed over the viaduct. After derailling, the rear 12 cement wagons broke through the viaduct deck on impact, and fell into the river below. Figure 3 shows the extent of the damage. While nobody was injured as a result of the accident, the deck of the existing viaduct was substantially damaged.

The cause of the derailment was eventually attributed to a number of factors, but the primary cause was the resulting interaction between the rigidity of the two-axle freight wagons and the deterioration in the timber waybeam track support system (RSC, 2003).

The damage to the viaduct was extensive. The wagons derailed at the rear of the train, while the engine and the front wagons continued and carried the derailed wagons along the open transverse deck for a short distance until eventually snagging on the transverse girders and then tearing each in turn from the connection to the main girders. This continued to such an extent that over half of the transverse girders were torn from the main box girders. The main box girders were also locally damaged at each of these connections, resulting in web distortion, yielding of rivets to the transverse connection and metal tearing of large holes in the web where the transverse connection was more substantial than the box web plate. In addition, there was also some minor damage to the existing bearings, abutments and piers.

Figure 3. Damage from the derailed train 2003

Immediately after the incident the operator's in-house design team of the Structural Design Section prepared a scheme for the re-opening of the Limerick–Waterford line.

2.4 Design features

An initial study into the most feasible options to allow re-opening of the railway included the option of retaining the existing main wrought iron box girders as well as a range of concrete and steel superstructure renewals. These options were reduced to the two most viable, with each option examined considering the merits based on the following primary objectives.

- Ongoing disruption to the public to be reduced to a minimum, that is the Limerick–Waterford line should be re-opened as early as possible.
- Provide a solution for single track with capacity for full RU design loadings.
- A sustainable solution respecting the heritage of the existing structure and the town of Cahir.
- Retention of the existing substructures as much as practicable.
- Economic viability of the solution based on capital and whole life costs.
- Constructability, with a focus on reducing the health and safety risks to a minimum.
- Robustness of the new structure and provision of derailment protection.
- Alignment of the track to remain as existing owing to the proximity of the adjacent Cahir railway station (this effectively meant that ballasted track over the bridge was not a feasible option as it would have increased the rail level above the allowable limits for both the station and platform levels).

The original Cahir viaduct was designed as a twin track structure, but has operated as a single track structure since 1931. There were no plans for installing a second track in the foreseeable future and as such the project requirement was to reconstruct and maintain the viaduct deck for single track only while retaining the existing substructure. This factor had a significant impact on the viability of retaining the existing box girders, as these were adequate to support the new design loading for one track only and the additional deck dead load if the main girders were effectively restrained. This factor also provided confidence that the fatigue capacity would not pose a long-term problem given the low yearly tonnage for the line and the expected stress ranges. Thus, under this option the structure could be designed far in excess of a 50-year operating life.

As such, the two options that best satisfied the design criteria were identified as follows.

- (a) Option 1. Replace the existing superstructure with a new economic steel design, consisting of half-through plate girders for the shorter spans and a truss for the longer central span while retaining the existing substructure, see Figure 4(a).
- (b) Option 2. Retain the existing Fairbairn box girders, repair the webs and the local damage caused by the accident and construct a new robust composite integrated deck system, see Figure 4(b) and inset.

Option 1 met the project objectives but had the following disadvantages.

- The elevation of the viaduct would be altered significantly.
- The construction programme would require an extended programme, so as to allow for fabrication lead times.

- The implementation costs would be prohibitive.

Option 2 also met all the objectives and had the following advantages.

- The elevation of the viaduct would only be changed slightly but the solution would respect the existing heritage.
- The solution provided the most sustainable solution by retaining the existing piers, abutments and main girders.
- There were significant cost savings when compared to option 1.
- The works could be carried out in a shorter programme period of 22 weeks.

Option 2 was selected because it presented better heritage, sustainability, economic and constructability advantages.

Figure 4. New superstructure options: (a) option 1 – new steel superstructure on existing abutments/piers; (b) option 2 – existing main girders strengthened and with new deck system on existing abutments/piers

Figure 5. Finite-element model

The selected design focused on two particular areas – first the strengthening and repair of the main box girders and second the new deck system. It was decided that given the level of damage that had occurred at the location of the original transverse girder connections it would be prudent to avoid reusing this location as part of the primary load path connection. The mid diaphragm location of the box was selected as the next most suitable location to form the new deck connection onto the box. However, this would alter the existing load path, and as such, it would be necessary to install additional bracing inside the box girder at the support locations. A three-dimensional (3D) linear-elastic finite-element model, see Figure 5, was created of the complete structure with second-order effects applied as external loads to the box girder to investigate in detail the effects of the new load path.

The new deck hanger design would also facilitate safer, easier and more rapid construction on site, as well as providing additional lateral torsional stiffness and thereby increasing the overall buckling capacity of the main box girders.

2.5 Construction

As well as conducting the design works for the project, the operator's structural design team led the overall project management and site supervision of the works. Constructions of the new works were awarded to an external civil engineering contractor John Mowlem Construction Ltd, with steelwork fabricators LMH Engineering Ltd. The sequence of the works is briefly detailed as follows.

- Repairs and strengthening of the Fairbairn boxes (external) – areas where the external webs were damaged were cut back to the location of the stiffener or flange riveted connection, or back to undistorted existing web plates. The new steel plate

Figure 6. Repairs conducted to the web panels and strengthening inside the box girder (inset)

sections were bolted in place using high-strength friction grip (HSFG) bolted connections, see Figure 6.

- Repairs and strengthening to the Fairbairn boxes (internal) – damaged internal diaphragms were replaced. In addition, due to the alteration of the load path, internal bracing and extra bearing stiffeners were installed, see Figure 6 inset. The internal box girder dimensions were 3.1×0.715 m providing a difficult confined-space working environment. This was mitigated to some extent by the provision of new access hatches at each end of the box girders and web entry hatches close to the centre of the main spans.
- Installation of the new deck panels – the deck panels were lifted into position in series, consisting of three transverse

Figure 7. New frames being installed

Figure 8. Frames in place: part concreting of deck in progress

beams and hangers, with the deck plate joints between the units being welded on site, see Figure 7.

- Concreting run-on slabs on the approaches.
- Repairs to the bearings and stitching of substructures.
- Concreting the composite deck – following the installation of all steel panels, the concrete deck was poured, with the upstands for the embedded rails cast separately, see Figure 8.
- Permanent way works – installation of the proprietary embedded rail system, derailment protection guardrails and track expansion devices on the viaduct approaches to alleviate the track/structure interaction effects.
- Load testing and verification of the theoretical model – prior to the re-opening of the viaduct a series of stress–strain and deflection–rotation checks was conducted using a test train to confirm the assumed parameters used during the design phase, see Figure 9. The results compared well

Figure 9. Completed works and inset load testing of structure

Figure 10. Completed project

with the model and confirmed the theoretical predictions and the alternative load path through the box.

- Opening of the Limerick to Waterford line, see Figure 10.

Reconstruction work began in February 2004 and was completed in early August. Thereafter, a series of safety checks was carried out on the line and on the viaduct, and it fully re-opened on 23 September 2004.

3. Thomastown viaduct

3.1 Introduction

Thomastown viaduct carries the Dublin to Waterford railway over the River Nore at Thomastown. The central span comprises a single track span of 67 m with the main superstructure forming a through bowstring arch girder. Prior to the strengthening works the deck consisted of a series of wrought iron transverse and longitudinal members supporting timber waybeams and the track rails. The viaduct is supported by rough-hewn coursed vaulted masonry abutments, which rise 24 m above river level.

3.2 History

The railway line between Kilkenny and Thomastown was opened by the Waterford and Kilkenny Railway Company in May 1848. It was later extended from Thomastown to Jerpont Hill, opening 2 years later. This section required the construction of a viaduct over the River Nore, which originally consisted of timber lattice girders supported on masonry abutments to a design by W. S. Moorsom. This viaduct at Thomastown was renowned as the longest single-span railway bridge in either Britain or Ireland at the time (Cox and Gould, 1998), and the *Illustrated London News* reported on the new viaduct in June 1850, describing it as ‘stupendous... the largest work of its kind in the three kingdoms’ (Barry, 1985).

However, from the start there was public unease about its safety. A series of tests was carried out and finally in June 1858 a report concluded that while the timber viaduct was safe, it needed constant monitoring and that the railway company should start preparing for its replacement with a more permanent structure. Finally, in 1875 it was decided to replace the timber structure with one of wrought iron, with construction starting in the autumn of 1876 under the supervision of engineer Charles Richard Galway (1840–1894) (NIAH, 2009). The bowstring arch replacement viaduct was completed in 1877, and is one of Iarnród Éireann's most prestigious viaducts, see Figure 11.

3.3 Project background

Following a request from the viaduct maintainer, the Structural Design Section of Iarnród Éireann initiated a study into the condition and load-carrying capacity of the central span, as observations from routine inspections indicated that there was a potential reduction in capacity due to deterioration of some of the deck members. The output of the study is summarised below.

- The load-carrying capacity of the deck and main girders was satisfactory for some passenger-only locomotives at line speed, while heavier trains were limited to lower speeds. The lack of critical capacity was due to the low strength in the deck system of longitudinal and transverse girders. The main bowstring arch girders themselves proved to be sufficient for all rolling stock at line speed.
- The deck of the span was not capable of supporting derailment loads and the main girders were susceptible to collision from trains during derailments.
- The track alignment on the approach warranted the installation of a concrete run-on slab and derailment protection.
- The inner vault within the northern abutment exhibited signs of deformation and required repairs.
- The roller bearings had seized, and the resulting thermal

Figure 11. Existing viaduct elevation

contractions and expansions were potentially causing cracking of the abutment.

The study recommended that to provide the operator with the rolling stock flexibility required to manage their business efficiently and to address safety and maintenance liabilities the viaduct should be strengthened or replaced, such that

- the viaduct could support all rolling stock at line speed
- the viaduct was capable of resisting derailment loading or the risk of derailment was mitigated
- the timber waybeam support system was replaced with a continuously welded rail ballasted track or embedded rail system
- the life expectancy of the viaduct was increased to a minimum of 50 years.

The Structural Design Section proceeded to prepare a scheme for the rehabilitation of the viaduct.

3.4 Design features

It was apparent from preliminary assessment calculations that it was not practicable to raise the track level and strengthen the central viaduct span to withstand the loading of a ballasted track with full RU design loading. It was therefore decided to proceed with the development of potential designs for the strengthening to provide for RU loading and an embedded rail system.

As part of any railway rehabilitation feasibility study, one of the key factors influencing the design is the allowable disruptive possession time. In the case of Thomastown there was already a 10-d disruptive possession allocated for re-signalling works on the line, which was an ideal opportunity also to use for the repair and strengthening works of the viaduct.

In studying the various options for the superstructure, the substructure works were common to all options – the repair and strengthening of the abutments and the replacement of the existing bearings. Both these sets of works were conducted prior to the 10-d possession.

The defects in the vaulted abutments were at least in part attributed to the seizure of the existing bearings and the thermal movements of the structure. The repair works involved masonry stitching and the installation of pattress plates and through-ties through the abutments where the defects were present. Also a shotcrete sprayed lining was installed to the inner vault within the northern abutment, which had initial deformations typical of progressive hinged mechanism failure. The bearing replacement works took place during a weekend possession, which involved jacking the central span vertically

and installing new bedstones and grouted anchors into the abutment walls. The new bearing system consisted of bespoke manufactured elastomeric bearings under the main girders, and the installation of both new fixed and guided pot bearings to form part of the new deck system. The pot bearings would then be accommodated in any new deck strengthening design so as to ensure sufficient transfer of longitudinal loads into the abutment.

Several options were studied to determine the best strengthening solution. The principal element needing strengthening was the main deck system. The main girders were already sufficient for all railway vehicles at line speed, and it was decided that there was no benefit in strengthening these for RU loading. Consideration was therefore given to strengthening the individual deck members and their connections. The most suitable options selected for the deck system works are detailed as follows.

- (a) Option 1. Remove the existing deck system, see Figure 12(a), and install two haunched HEB 600 beams either side of each existing transverse girder. Install longitudinal steel proprietary 'silent deck' embedded rail system, concrete run-on slabs and carry out maintenance painting, see Figure 12(b).
- (b) Option 2. Retain the existing deck system but increase the capacity of it by introducing a reinforced concrete composite deck (this type of system had previously been developed in strengthening bridges in the Iarnród Éireann stock and was further evolved for this project). Install an embedded rail system in troughs on the new concrete deck, concrete run-on slabs and carry out maintenance painting, see Figure 12(c).

Option 1 met the project objectives but had the following disadvantages over option 2.

- The original main span would be significantly altered, with the additional haunched transverse beams significantly impacting on the heritage of the structure and would be visually obtrusive from ground level.
- The works would be significantly more complicated and provided more health and safety risks with the installation of new fabricated haunched beams that required being lifted into place alongside the existing deck transverse beams and the follow-on removal of the existing deck.
- The new deck system would be less robust to the accidental situation.
- The cost would be greater than for option 2.

Option 2 also met all the objectives and had the following advantages.

- The original central span is retained, providing the most sustainable solution.

- The visual appearance of the structure would effectively remain unaltered in elevation.
- The works could be carried out in a shorter duration.
- The solution provided the safest construction environment.
- The cost would be more economical than option 1.
- There would be fewer unknowns, leading to fewer potential delay risks.

Option 2 was selected because it presented better aesthetic, sustainable, economic and constructability advantages.

The design was conducted in line with BS 5400 part 5 (BSI, 1997a) and utilised 3D linear finite-element modelling. The design of the works indicated that the deck system did not need to be propped in order to provide the capacity required for full RU design loading. Initially, it was decided to investigate the possibilities of using a precast concrete deck bolted down to the top flange of the existing beams through preformed holes in the slab and the existing rivet holes. This would allow the fastest construction time. The main difficulty would be in the provision of tolerances of the bolt holes cast into the concrete so that these would match with the existing rivet hole locations.

As an alternative, it was also decided to investigate the possibility of using in situ concrete for the new deck. As such, in order to provide composite action, HSFG bolts would act as shear studs. These would replace existing rivets at specified intervals in the top flange of the existing transverse and longitudinal beams. Each bolt assembly would consist of two nuts, a load-indicating washer, a standard flat washer and a bolt with an extended thread. The bolts would protrude 100 mm into the concrete deck and act similarly to a normal welded shear stud. This option would prove feasible for the possession time available and would avoid the constructability issues of using precast slabs.

3.5 Laboratory research and testing

The HSFG stud as a shear connector is an excellent solution for ease of constructability but was, however, untested in this capacity. As such, laboratory testing was conducted in line with the recommendations of BS 5400 part 5, which allows the use of different types of shear connector, provided shear tests are conducted on that particular type (BSI, 1997a). In addition, other alternatives were researched, including the use of resin injection bolts to reduce the slip characteristics associated with HSFG bolts and additional research into the part that existing protruding rivet heads would contribute in the overall ultimate limit state (ULS) capacity of the system.

Three sets of test specimens were constructed and tested to failure in the laboratory of Trinity College Dublin. One set consisted of steel beams with a smooth uncoated top flange. This was to represent the baseline between it and the second set, which consisted of a painted beam with rivet heads

Figure 12. Deck sections: (a) existing deck layout; (b) option 1 – new transverse beams with propriety 'silent deck' system; (c) option 2 – existing deck strengthened with concrete composite system

Figure 13. Test rig for shear stud tests

protruding as would be encountered on site with existing beams. The third set were resin injection HSFG studs in which an epoxy resin was injected into the void between the bolt and the bearing area of the flange holes. This was used to investigate what portion of the slip was related to slippage of the HSFG bolt and what portion was due to the deformation of the flexible connector within the concrete. Figure 13 illustrates the test rig and Figure 14 illustrates the failure modes observed.

The output from the tests were to ascertain the load–slip curve and the ultimate load resistance of each method of shear connection. The tests were designed with guidance taken from BS 5400-5:1979 5.3.2.4 (BSI, 1979). Each test unit consisted of a metallic I-beam section approximately 450 mm long. Two shear connectors, general grade 22 mm dia. HSFG bolts, were

Figure 14. Failure modes

7-d cube strength: N/mm ²	27	±3
Modulus of elasticity: N/mm ²	22 452	±0.723
28-d cube strength: N/mm ²	57	±6
Modulus of elasticity: N/mm ²	35 365	±1290
Aggregate size: mm	20	

Table 1. Concrete test results

attached to each flange, which were in turn encased in concrete slabs of approximate sizes 450 × 350 × 175 mm thick with B12 reinforcement throughout. The units were placed under a stiff loading frame and hydraulic jack, and loaded at a constant rate of 50 kN/min. Transducers were positioned at each side of the beam so as to measure any differential slippage between the sides. The units were tested at 7-d concrete strength in accordance with BS 5400, part 7 (BSI, 1997b) with the results indicated in Table 1.

3.6 Test results

The averages for each set of test results are illustrated in Figure 15 with the ULS results and failure modes as indicated in Table 2.

3.7 Test conclusion

There are significant differences in the initial load–slip characteristics of the three sets of tests, as can be seen from Figure 15. There are three distinct reasons for the variations.

- (a) Normal statistical variation – due to inherent variations in materials properties and in-built imperfections, eccentricities of the loading applied, transducer locations and dimensional differences in the sections used.
- (b) Surface protrusions on the wrought iron beam as opposed to the steel beams, which had varying degrees of rivet head protrusions.
- (c) Resin injection – the resin injection bolts had the least slip. This is because the bolts could not move as the annulus around the bolt shaft was filled with resin.

The ultimate failure mode of the units unfortunately did not provide a distinguishable pattern, with some failing by pull-through, some by shear and three by both. However, one significant finding that could be taken from this is that 7-d concrete strength of 27 N/mm² provided sufficient strength to anchor the studs to the extent that the majority of failure modes were by shear of the bolts. In applying statistical techniques based on a Gaussian distribution it was determined that a characteristic resistance value of 146 kN provided adequate structural strength. This value is comparable to a 22 mm dia. shear stud acting in concrete with a compressive strength of 45 N/mm² when interpolating from

Figure 15. Load-slip graphs for averaged results

table 7, BS 5400 part 5. In addition, the similarity between the relevant tests and the field of application provided assurance that typical recommended partial factors of safety were appropriate.

3.8 Construction

As well as conducting the design works for the project, the operator's structural design team led the overall project and construction management and site supervision of the works. The specialist wrought iron and shear stud installation construction works on site were led by the operator's in-house bridge maintenance teams, with the concreting of the deck

being awarded to an external specialist contractor, Clonmel Enterprises Ltd.

The replacement of the bearings, stitching and installation of the tie-bars and pattress plates took place prior to the main 10-d possession. In addition, pre-possession works during non-disruptive possessions involved installing the majority of the shear studs to replace the existing rivets, where accessible around the existing track layout, and the installation of formwork from a temporary slung scaffold.

The sequence of main possession works is briefly detailed as follows.

- Removal of the track in short sections – due to the limitations of site access and the through type structure the only feasible lifting equipment available were on-track road-rail vehicles (RRVs) with a limited lifting capacity. As such, the works were sequenced so that two RRVs worked in conjunction with each other to remove the track, starting at the centre of the main span and moving towards each end of the span.
- Following the removal of the central section of existing track, the bridge maintenance team installed the remaining shear studs.
- The RRVs then placed prefabricated steel reinforcement cages in short sections where the existing track had been removed, see Figure 16.

Test unit	Ultimate load: kN	Failure mode
Steel beam 1	176	Shear
Steel beam 2	160	Shear
Steel beam 3	145	Shear
Wrought iron beam 1	172	Pull through
Wrought iron beam 2	168	Shear
Wrought iron beam 3	178	Shear/pull through
Wrought iron resin beam 1	172	Shear
Wrought iron resin beam 2	150	Shear/pull through
Wrought iron resin beam 3	179	Shear/pull through

Table 2. Shear units test results

Figure 16. Existing track being removed, shear studs and the first reinforcement cage installed

- In parallel, the connection between the existing transverse girders, bottom boom and the new concrete slab were strengthened by installing additional shear studs and through reinforcement into the concrete deck.
- This sequence was repeated until all the steel reinforcement cages for the new deck had been installed, see Figure 17.
- Concreting then started on the cast in situ run-on slabs on the approaches.
- Concreting of the deck was undertaken by pumping small aggregate concrete from a nearby access point. The deck was poured with the upstands for the rails poured separately.
- Permanent way works – installation of the embedded rail system and derailment protection guardrails on the approaches.
- Completed works – see Figures 18 and 19.

3.9 Monitoring

Prior to the works, strain gauge measurements were taken at the longitudinal, transverse and bottom boom sections. This was to ascertain that the finite-element analysis model was calibrated correctly and to compare directly the results of measurements taken before and after strengthening. The pre-strengthening measurements deviated by 3% to the finite-element analysis predictions, and indicated a live load stress of 59 N/mm^2 on the

Figure 17. Steel reinforcement cages completed

bottom flange of the transverse members for a static Iarnród Éireann type 201 locomotive (110 t) and concrete freight wagons.

After strengthening of the main span, further measurements were taken and the theoretical predictions also closely correlated at 4% to the measured prediction of 34 N/mm^2 stress on the bottom flange of the transverse girder for the

Figure 18. Underside view of completed concrete deck

Figure 19. Monitoring

same live loading, indicating that there was full interaction of the composite section under live loading. This represented a reduction of live load stress in the bottom of the transverse beams of 41%. The effects of extra dead load from the concrete and the concrete creep/shrinkage stresses have the effect of reducing this improvement in strength to 32%. Another interesting feature of this particular design is that the global concrete shrinkage will produce relieving effects in the top and bottom booms, but these have been omitted from these calculations.

The concrete deck has improved the main span significantly. The bottom booms that acted independently previously, will now act along with the concrete deck and longitudinal beams, thus giving a stiffer cross-section of the span as a whole. The continuous deck will reduce the dynamic impact on the deck members by approximately 30%, and lastly the deck members themselves have increased section moduli of approximately 45%, while also providing derailment protection and significant additional robustness.

The reduced stress range and dynamic impact, in addition to some local strengthening for the connections, will provide an additional 50 years' fatigue life for the deck members, which previously may have become critical to the future operation of the viaduct with time.

4. Slatty viaduct

4.1 Introduction

Slatty viaduct carries the Cork to Cobh railway over the estuary of Slatty Waters in Cork harbour. The eight-span viaduct comprises two masonry arches on the approaches, and

six wrought iron half-through bowstring arch spans supported on cast-iron columns, giving a total overall length of 137 m.

The twin track was carried on timber waybeams supported by a combination of longitudinal and transverse wrought iron beams.

4.2 History

In 1855, the year that the Dublin–Cork line was completed, the Cork and Youghal Railway Company were granted authority to construct a new line from Glounthaune to Cobh. However, owing to navigation difficulties with the admiralty, construction did not begin on the railway line until 1859, but the single track line was finally opened in 1862 (Barry, 1985).

The line between Cork and Cobh was an immediate success, serving as access to Cork harbour and the trans-Atlantic trade, and owing to this success was later reconstructed as a twin track in 1882.

Slatty viaduct, see Figure 20, was built as part of the twin track construction carrying the Cork to Cobh railway over the Slatty Waters between Harpers Island and Fota Island.

4.3 Project background

In 2003, following a request from the viaduct maintainer, the operator's Structural Design Section carried out a study into the condition and load-carrying capacity of the viaduct. The output of the study is summarised below.

- The load-carrying capacity of the deck and main girders was satisfactory for passenger-only locomotives at a speed

Figure 20. Original deck system of existing viaduct spans

of 64 km/h. The lack of critical capacity was due to the low strength in both the main girders and the deck system of longitudinal and transverse beams.

- The deck of each span was not capable of supporting derailment loads and the main girders were susceptible to collision from train impacts during derailment.
- The track alignment on the approaches warranted the installation of concrete run-on slabs and track guardrails.
- The masonry arch spans required stitching works, some grouting and general repointing.
- The cast-iron bearings consisted of a stool type configuration with a voided centre. A number of the bearings had fractured; although the fractures were evident in the earliest inspection records, it was considered prudent to restrain and grout inject them.
- The complete structure would also require a new paint system to be applied.

The study recommended that, in order to meet the ongoing business needs of Iarnród Éireann, the viaduct should be strengthened or replaced, such that the following objectives could be met.

- As a minimum the viaduct was to support all rolling stock at line speed, but additional capacity for full RU design loading would be desirable.
- The viaduct was to be capable of resisting derailment loading or that the risk of derailment was mitigated.
- The timber waybeam track support system was to be replaced with a continuously welded rail ballasted track or embedded rail system.

- The design life of the viaduct was to be extended to a minimum of 50 years.

The Structural Design Section proceeded to prepare a scheme for the rehabilitation of the viaduct.

4.4 Design features

From a preliminary assessment of the existing deck system and the main girders it was apparent that it was not practicable to raise the track level and strengthen the viaduct to support the loading of a ballasted track with full RU design loading. It was therefore decided to proceed with the development of feasible designs for the strengthening for RU design loading and an embedded rail system.

Several options were considered to determine the most feasible design option for the superstructure strengthening. The two most feasible options selected are detailed as follows.

- (a) Replacing the existing superstructure with new steel half-through plate girder spans, and reusing the existing substructures, see Figure 21(a).
- (b) Retaining the existing wrought iron spans but increasing the capacity by installing a concrete composite deck similar to the design used for Thomastown viaduct. In addition, strengthen the top boom members of the bowstring arches by installing curved channel sections, installing an embedded rail system with the new concrete deck, constructing run-on slabs on the approaches, repairing the existing bearings and carrying out maintenance painting, see Figure 21(b).

Figure 21. Options: (a) option 1 – replace with a new half-through steel solution; (b) option 2 – existing deck strengthened with concrete composite system

Option 1 met the objectives but had the following disadvantages.

- The works could not be carried out within the scheduled possession, and would require a longer railway shutdown to facilitate the works.
- The works would be more extensive for the removal of the existing superstructure, necessitating the use of large floating plant.
- The projected cost would be economically prohibitive.

Option 2 also met the project objectives and had the following advantages.

- The original viaduct would be retained providing a sustainable solution.
- The visual appearance of the structure would essentially remain unaltered respecting the original heritage.
- The impact on operations would be considerably less than option 1.
- This solution did not require the use of large floating plant, thus reducing risk.
- The cost would be less than for option 1.
- There were fewer project unknowns that could lead to potential delay risks.

Option 2 was selected because it presented the best aesthetic, sustainable, economic, risk averse and constructability advantages.

In addition, another advantage of option 2 was the possibility of temporarily closing one of the lines and using bi-directional operations on the other line, while allowing works to be undertaken during normal railway operations. The design was also further prioritised by a bridge renewal on the same line, which required a 4-week closure and provided a significant possession for the works.

The assessment and design was conducted in line with BS 5400 part 5 and utilised 3D linear finite-element modelling.

With the successful implementation of the Thomastown rehabilitation works, a similar deck strengthening design was conducted at Slatty. The existing wrought iron deck plates of each span provided suitable permanent formwork. HSFG bolts were installed to act as shear studs along the existing transverse and longitudinal beams. The main girders needed to be strengthened, and the most appropriate approach selected was to involve bending parallel flange channel sections to the inner radius of the bowstring arch and then bolting on with predrilled holes on site.

4.5 Construction

As well as conducting the design works for the project, the operator's structural design team led the overall project and

construction management and site supervision of the works. The wrought iron and shear stud works on site were led by the Iarnród Éireann in-house bridge maintenance teams, with the concreting of the deck and painting being awarded to specialist contractors Clonmel Enterprises Ltd and J. S. McCarthy Ltd, respectively.

The pre-possession works involved installing the majority of the shear studs to replace the existing rivets, where accessible around the existing track layout, and the installation of a temporary hanging scaffold, see Figure 22. It also involved installing the channel strengthening works to the central main bowstring arch girder of each span. The sequence of main possession works is briefly detailed as follows.

- Removal of the track in short sections – due to the limitations of site access the only feasible lifting equipment available were on-track RRVs with a limited lifting capacity. As such the works were sequenced, so that two RRVs worked in conjunction with each other to remove the track, starting at the centre of the span and moving towards each end of the span.
- Following the removal of the central section of existing track, the bridge maintenance team installed the remaining shear studs.
- The RRVs then placed prefabricated reinforcement cages in short sections where the existing track had been removed.
- This sequence was repeated until all the steel reinforcement cages for the new deck had been installed.
- Concreting then started on the cast in situ run-on slabs on the approaches.
- The track was then over-ballasted to provide a level access for a concrete pump and concrete wagons to gain access along the causeway from Fota railway station. Concreting

Figure 22. Shear studs being installed

Figure 23. Steel reinforcement cages installed and concrete pumping commencing

of the deck was undertaken by pumping small aggregate concrete, see Figure 23. The deck was poured, with the upstands for the rails poured separately.

- Permanent way works – installation of the embedded rail system was undertaken using a fully encapsulated scaffold in order to provide the required environmental controls for the curing of the proprietary epoxy compound, see Figure 24.
- Permanent way works – installation of the derailment protection guardrails and track expansion devices on the viaduct approaches to alleviate the track/structure interaction effects.

The post-possession works involved installing the remaining channel strengthening to the outer bowstring arch girders of each span and the painting of the structure. Figures 25 and 26 show the completed works.

Some of the statistics of the works for Slattery viaduct are detailed, as follows

- 19 872 rivets replaced (shear studs, repair and strengthening)
- 563 mortice taper drill bits used
- 16.4 t of wrought iron repairs completed
- 36 bearings strengthened.

The works were completed between 2005 and 2006 and the significant effort taken to retain this historical structure was recognised when the project received first place in the first group skills award from the British National Heritage Awards 2006.

5. Conclusion

The use of innovative bespoke engineering, 3D finite-element modelling techniques, laboratory research and on-site calibration

Figure 24. Installation of the embedded rail system

and testing were used to develop, verify and confirm the effectiveness of the strengthening measures to these three iconic structures. In addition, over 50 years of usable life span had been achieved when the structures would otherwise have been deemed life-expired.

As such the integrity of these iconic structures will remain serviceable for at least another generation while also providing

Figure 25. Project completed: deck details

Figure 26. Project completed: elevation

a better economic and sustainable solution to the context of their environment. The strengthening techniques used were specific to the circumstances and configuration of each structure but are principally repeatable and are presented here to illustrate examples of how existing infrastructure can be further preserved to extend its working life and provide improved sustainability.

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