The Metrolink Finback Bridge, Manchester

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**ABSTRACT:** The Manchester to Oldham Line of the proposed Metrolink Phase 3 tram system passes through the new Central Park Business Park utilising the existing track bed of a disused and dismantled heavy rail route. To the west of Thorp Road Bridge the new Metrolink route crosses the existing four tracks of the Manchester to Leeds Trans-Pennine railway. A post-tensioned concrete finback bridge has been provided to carry the new twin Metrolink tracks over the existing heavy rail route. In order to minimise disruption to the operational railway, the Finback Bridge was constructed in a position alongside to and roughly parallel to the existing railway. Once the construction was substantially complete, the 6,250 tonne bridge was rotated in plan through 21° to its final position during a weekend possession.

1 INTRODUCTION

In the Moston area of northeast Manchester, the route of the proposed Phase 3 of the Manchester Metrolink tram system crosses the four tracks of the Trans-Pennine Manchester to Leeds. The original heavy rail configuration had two lines merging utilising level points; however the safety requirement was for the new Metrolink tram line to be grade separated from the existing operational heavy railway.

Initial designs for crossing the railway considered both tunnelling below the operational railway with a series of jacked box structures and alternatively providing a new bridge to carry the tramline over the heavy rail lines. The tunnelling option was not preferred because of the disruption it would cause to the operational railway and the extensive modifications that would be required to the adjacent Thorp Road Bridge. The option selected was a two span post-tensioned prestressed concrete box-girder finback design.

The Finback Bridge was designed and constructed as part of an enabling contract to create a new entrance into the Central Park Business Park being created on the 200 hectare site of the former Monsall Hospital. This project also involved the design and construction of the structures required for a new Metrolink stop for the new business park along with the crossing of the heavy rail lines.

The ground conditions consist of 1.5 m of Made Ground comprising 300 mm of ballast, overlying varying thicknesses of medium gravel, cobbles, gravelly sand and soft to firm gravelly clay. Directly underlying the Made Ground is Glacial Till to a depth of approximately 33 m below existing ground level.

2 CONCEPTUAL DESIGN

The form of the Finback Bridge developed in response to the many constraints of the site.

The vertical alignment of the tram lines carried by the bridge had to have vertical gradients no greater than 5.5%, and when combined with the horizontal alignment had to allow theoretical tram transit times between the new Central Park stop and the subsequent Dean Lane stop of less than 96 seconds. The bridge had to provide a minimum headroom over the existing heavy rail lines of 4.80 m. Therefore the vertical alignment had to rise to clear the railway and then drop to pass through the southwest span of the existing Thorp Road Bridge to achieve a headroom of 4.35 m to the soffit.

The close proximity to the operational railway also meant there were significant constraints for construction clearances. A lateral clearance of 3.0 m was adopted for construction and the central pier position was required to be more than 4.5 m from the nearest rail to be outside the zone of train impact.

To satisfy all these constraints the resulting alignment of the bridge is a compound ‘S’ shaped curve in plan (Fig. 1). A consequence of the curved horizontal alignment is that significant torsions are induced within the deck from both permanent and transient loading.
In order to minimise the gradients it was clear that a ‘through girder’ type solution would be required, where the tram tracks could be supported on the bottom flange. Also a box section was an efficient structural form for transmitting the torsions induced in the deck as a result of the curved alignment to the torsional restraints at the supports. This cross sectional shape would be further tuned by varying the sidewall thicknesses to control torsional shear stresses in the webs.

The resulting arrangement of the bridge is a 131.8 m long two span, post-tensioned prestressed concrete finback design. The tram tracks are supported on cantilever extensions of the bottom flange. The main span is 87.9 m and the backspan is 43.9 m (Fig. 2). The fin is 2.5 m wide at its base and rises from 4.5 m high at the two abutments to 7.0 m high over the central support.

In order to minimise construction work over the operational railway, it was decided to construct the deck in a temporary position rotated by 21° about the central support so that the deck was roughly parallel and beyond a 3.0 m clearance line for the nearest live heavy rail line. This was then rotated into its permanent position once construction of the substructure and deck was complete.

3 FOUNDATIONS AND SUBSTRUCTURE

A significant proportion of the weight of the bridge is carried by the central support. The bearings at the underside of the central support downstand also provide torsional restraint to the deck. The reactions to these loads are provided by an arrangement of 28 No 1050 mm diameter reinforced concrete CFA piles. An asymmetrical pile arrangement with a centroid offset to the south of the group was selected to correspond
to the centroid of the imposed permanent loading. The pile group is connected by a stepped reinforced concrete pilecap to minimise excavation depths adjacent to the railway. These piles are founded in the Glacial Till.

The east abutment is also supported on a group of 10 No 1050 mm diameter reinforced concrete CFA piles. The bearings on this abutment are offset to the south primarily to provide clearance to the adjacent railway but also to correspond to the centroid of the vertical loads and offer torsional restraint.

The short backspan induces uplift reactions at the west abutment. Therefore the west abutment is a pad foundation with an arrangement of two bearings and four short near-vertical post-tensioning tendons to resist uplift.

4 SUPERSTRUCTURE

4.1 Description of structural form

A typical cross section of the superstructure is indicated in Fig. 3. The superstructure has constant external dimensions with horizontal cantilever slabs and is ‘extruded’ along the design alignment with some variations. The height of the fin varies, being a maximum over the intermediate support and the internal dimensions of the fin vary to allow thickening of webs, base slab and roof. The external width of the fin was minimised within the requirements for access and structural adequacy. The transverse slab thickness was minimised to mitigate the headroom and associated track gradients. Track cant is provided by varying height rail plinths; detailed geometric checks being made to ensure that clearances to the fin were maintained. Subsequent checks on the deflected shape confirmed that the track cant in service remains within acceptable tolerances.

Distortion of the fin is controlled by internal diaphragms, which are also used to provide anchorages for the external tendons.

The superstructure is post-tensioned longitudinally with external tendons, within the fin, over the intermediate support and internal grouted tendons in the slab of the main span. Transversely the cantilever wings of the deck act as normally reinforced concrete members.

4.2 Design loading and thermal stress distributions

In general, design loading is in accordance with BS 5400 Part 2, but with additional interpretation applied to better suit the structural configuration and method of analysis. The bridge is intended to carry a light tram system, however is designed for standard RL railway loading.

The ultimate limit state load factors given in BS 5400 Part 4, Clause 4.2.2 for creep and shrinkage are not appropriate to an analysis that explicitly incorporates time dependent effects and instead global load factors to be applied to the SLS time dependent results (of 1.2 and 1.1 for \(\gamma_{fl}\) and \(\gamma_{f3}\), respectively) were agreed with the Client.

SLS stress and crack width limitations were based on BS 5400 Part 4 Clause 4.2.2 (a). The application of the requirements to a post-tensioned railway structure with a mixed tendon arrangement is not fully defined and it was agreed with the Client that in tensile zones containing only internal grouted tendons, the structure would be checked for Class 1 stresses under permanent loads only and Class 3 stresses with a 0.25 mm crack width, for all other combinations. In tensile zones containing external tendons, the structure would be checked for Class 1 stresses under permanent loads only and as a reinforced concrete section for all other combinations.

More appropriate differential temperature profiles were agreed with the Client based on winter and summer sun, positive and negative difference and the orientation of the structure.

4.3 Method of analysis

Analysis of the superstructure was carried out in two stages.

Preliminary design, section sizing and post-tensioning requirements were determined using a ‘line beam’ and the computer package Sofistik, a 3D time dependent analysis program. The simplicity of the line beam model and the use of a simplified construction sequence enabled the various design parameters to be optimised fairly rapidly, and, as it transpired, fairly accurately. During this preliminary work it became clear that the overriding design effect was to be the web shear stresses under the worst combination of vertical shear and torsion and at this stage various mitigating measures were considered such as an internal composite steel lining. It was determined, however, to retain
the simplicity of the concrete structure and although departures for the use of higher strength concretes were agreed with the Client, these were not, in the final event, implemented. This preliminary work was carried out by Bilfinger Berger and enabled an early start to be made on the design of the temporary works.

The final design was carried out using a 3D shell element model of the full superstructure. This enabled section sizes and post-tensioning requirements to be optimised and included full time dependent effects based on the proposed construction programme. All construction stages were modelled, including rigid body movements, enabling falsework loads and temporary works effects to be investigated. Some reductions were made in the numbers of external tendons but more significantly in those in the slab. Fin web thicknesses were finalised on the basis of limiting combined shear stresses with the discreet loading from the external tendon anchorages being incorporated. Transverse reinforcement requirements were determined directly from the model and the facility of the Sofistik software to section the model to report stress summaries enabled the ultimate strength to be checked.

4.4 Post-tensioning systems

The post-tensioning system was based on the Freyssinet range and early discussions with Freyssinet enabled both the physical properties and design parameters to be incorporated into the design process at an early stage.

External tendons in the fin are all Freyssinet System 2, 37R15, and a total of eighteen are used. The Type 2 system comprises discreet greased and sheathed strands in an HDPE duct, which is cementitiously grouted prior to stressing. The protection system is provided by the greased sheath and the duct. The grout is a spacing mechanism to enable individual strands to be replaced, should the need arise, in the future. In general, due to the limited access, stressing was carried out using a mono jack from one end although jacking anchorages were provided at both ends.

Grouted tendons in the slab are all Freyssinet System 1, 37C15, and a total of six are used. The Type 1 system comprises conventional bare strands in a corrugated plastic duct, cast into the concrete, which is cementitiously grouted after stressing. The protection system is provided by the grout and the duct. Stressing was carried out using a conventional multistrand jack from the intermediate support anchorages although jacking anchorages were provided at both ends.

The vertical stressing at the west abutment comprises four slightly inclined ungrouted tendons. Each tendon contains 19C15-galvanised strands within galvanised steel formwork tubes. The corrosion protection at the gap between the deck and the substructure is a flexible plastic gaiter.

5 CONSTRUCTION AND ROTATION

5.1 Articulation and bearings

In its final position the superstructure is supported and torsionally restrained at three locations.

The intermediate pier (Fig. 4) is integral with the superstructure and supported on and torsionally restrained by the two outer bearings at a low level (on the underside of the downstand pier). Longitudinal and lateral restraint is provided by a ‘pivot’ bearing, which takes no vertical load. The two vertical support bearings were fitted with extended base plates to permit the rotation of the superstructure into its final alignment.

The east support bearings are offset (Fig. 5), both to provide the necessary structural clearances to the operational rail tracks and the torsional restraint to the superstructure. The greater proportion of the vertical load is taken by the south bearing but the north bearing was positioned to ensure that it was always subjected to a positive reaction. The south bearing is longitudinally guided and a deck expansion joint fitted. The track alignment at the east end was governed by the proximity and alignment of the operational railway and only minimal lateral or vertical tolerance could be built in to the construction process.

The west support bearings (Fig. 6) are symmetrically located and, as indicated above, are combined with a vertical stressing arrangement to prevent uplift. Due to the vertical and lateral restraints at the intermediate support and east abutment all tolerances due to construction, stressing and creep had to be incorporated at the west abutment where some flexibility in vertical and horizontal alignment was available.
5.2 Construction position and sequence

The limitations on clearances, as indicated above, to the operational railway and the required final alignment of the bridge resulted in the centre of rotation of the bridge being offset from the actual structure centreline (Fig. 7). The geometric definition of the superstructure falsework (in the construction position) was based on the final design line alignment after applying a transposition matrix for an anti-clockwise rotation of $21^\circ$ to give xyz coordinates at any lateral offset from the design line.

Consideration was given to pre-cambering the cast structure both for vertical deflection, plan deflection and torsion. The analysis suggested that when resting on its bearings under permanent loads the vertical alignment would be fairly neutral, both in the short and long terms, and precamber would not be needed. Due to the post-tensioning being applied to a curved-in-plan structure the plan shape changes during stressing. The plan deflection is significant, 50 mm at the west end, the east being fixed, but this could be accommodated by measures incorporated into the west abutment construction and proposed track alignment. No plan precamber was proposed. The vertical deflections due to torsions in service were significant but due to the shape of the soffit did not compromise headroom. The variation in cant was within acceptable limits and consequently no torsional precamber was proposed. The lack of need for any casting precamber greatly simplified the definition of the cast geometry.

The construction sequence was modelled in some detail with a view to:

- Ensuring that the structure was not overstressed, in tension or compression, or under strength at any time during the construction process.
Ensuring that the loading on the falsework, and other temporary works, did not exceed the design magnitudes.

Ensuring that the deflected shape at the time of rotation could be accommodated by substructure and bearing levels.

In view of the fact that the shape of the structure is both load and time dependant, the analysis was continuously re-run during the construction process using real times and actual jack loads in order to ensure the integrity of the structure at all times and to advise of any minor adjustments to enable construction to proceed as smoothly as possible.

5.3 Temporary works

Prior to and during rotation, the superstructure was supported on the intermediate pier bearings and a temporary nose at the east end, bearing on a sliding track over the operational railway. The backspan was in cantilever. The temporary nose was a structural steel fabrication (Fig. 8) fixed to the east bearing downstand and the fin webs with a push pull system of cast-in anchors. The bearing downstand required a substantial steel member to be cast-in in order to adequately transfer the loads from the fin webs into the steel nosing. The sliding end of the nose was offset from the deck end in order to provide adequate clearance to the permanent works and offset from the bridge centreline in order to minimise torsional rotations whilst on the single bearing.

The nosing was supported on a sliding track founded on a temporary reinforced concrete wall inside the construction limits and on a temporary steel beam over the operational railway. The beam was both placed and removed during the same track possession used for the rotation.

5.4 Rotation

The rotation into the final position was carried out during a track possession and achieved by pushing the nose on a PTFE sliding track using a system of jacks on a rack (Fig. 9).

6 CONCLUSIONS

This paper describes the design and construction of an unusual railway bridge structure in Manchester. The Finback Bridge carries the proposed Metrolink Phase 3 over existing heavy rail tracks. A novel design comprising a varying height finback form of construction has been developed. The central torsion box has varying wall thickness to control torsional shear stresses. Three types of post-tensioning are used in the design. The design provides an efficient and cost effective structural solution. Easy access to the main post-tensioning tendons, for inspection and maintenance, is provided from within the void in the fin.

An innovative method of construction has been adopted to minimise the disruption of the adjacent operational railway. The bridge superstructure was constructed in a temporary position roughly parallel to the railway. The substructure was rotated in plan through 21° to its service position during a weekend possession of railway. This proved to be a successful and cost effective construction method that minimised disruption to the operational railway below.

REFERENCES


British Standards Institution. 1990. Steel, Concrete and Composite Bridges. BS 5400 Part 4 – Code of Practice for Design of Concrete Bridges. BSI, Milton Keynes.