

## THE ORDSALL CHORD NETWORK ARCH BRIDGE – ADDRESSING COMPLEX DEMANDS THROUGH COLLABORATION

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### SUMMARY

The first network arch in the UK has been designed to support a new railway line that connects the Piccadilly and Victoria Stations in the heart of Manchester. This is an asymmetric twin network arch structure, which will satisfy the aspiration for a landmark bridge that will mark the regeneration of the particular site and at the same time show respect to the history of the area. The project constraints posed challenges to both the design and construction team that could only be resolved via collaborative working. The aforementioned constraints, the challenges and the respective solutions are presented in this paper.

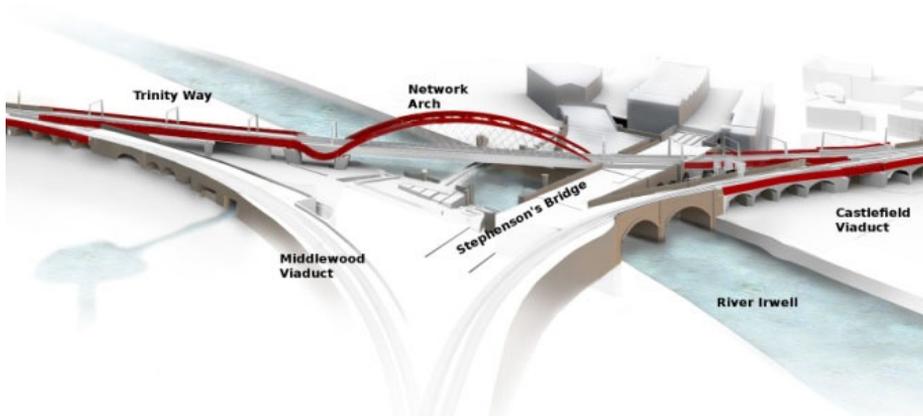
**Keywords:** *Network arch, railways, weathering steel, high strength steel, sustainability, BIM, advanced analysis, collaborative behaviours.*

### 1. INTRODUCTION

The Northern Hub Programme is a major infrastructure project in the North of England improving the capacity of the railway network. When completed, it will allow more trains to run in the area, thus increasing the available train passenger seats significantly. Amongst a number of interventions on the existing infrastructure, the project includes the construction of a new railway viaduct in the heart of Manchester, the Ordsall Chord. A visualisation of the Chord is shown in Fig. 1.

The Ordsall Chord is approximately 300 m long and connects two existing masonry arch viaducts, namely the Castlefield and Middlewood viaducts. It crosses a site with significant historic value to both cities of Manchester and Salford. Specifically, the Viaduct crosses Water Street, Stephenson's Viaduct, which is a Grade II listed Heritage structure, the River Irwell and Trinity Way, which is part of the Manchester and Salford Inner Relief Road. This paper concentrates on the River Irwell Crossing, a steel network twin arch, the first of its kind in the UK, spanning 89 m across the River Irwell, which is a navigable river. A brief description of the requirements and constraints will be presented that lead into a number of design and construction challenges, together with an explanation of the solutions given for both the superstructure and the supporting structures.

The structure comprises a system of two inclined, braced network arches supporting two tracks via a steel and concrete composite deck with transverse members supported from the longitudinal steel tie beams.



*Fig. 1. The Ordshall chord.*

A visualisation of the proposed structure and a photograph of the existing site conditions are shown in Fig. 2 and 3.



*Fig. 2. Visualisation of the network arch together with its surrounding structures.*



*Fig. 3. Existing conditions.*

### 1.1. The requirements

As illustrated in Fig. 3, the current conditions comprise an existing structure, Prince's Bridge, which is currently being used as a pedestrian crossing of the river. The existing structure is on the route of the new viaduct and will be demolished in advance of any works in the area. The existing public Right of Way will be substituted by a new footbridge, which will be built before the works for the new network arch bridge take place.



*Fig. 4. The Ordsall chord ribbon.*

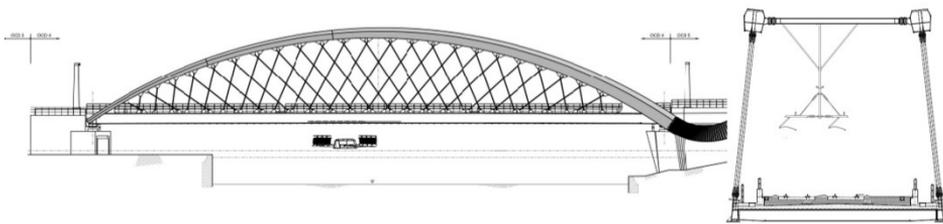
It is the aspiration of both the cities of Manchester and Salford to regenerate the area. As the new viaduct is at an elevation, the need for a landmark structure, which will mark the regeneration, was apparent. However, the new viaduct, as a whole entity, had to be

designed to respect the significant heritage of the site, which once used to be the epicentre of the development of railways in the United Kingdom.

Following public consultations, the aesthetic requirement for a slim and elegant but emblematic structural line has led to the choice of the architectural signature of the Ordsall Chord, a ribbon in weathering steel, connecting and unifying visually the adjoining structures, as shown in Fig. 4.

The requirement for visual continuity between the network arch and adjacent parts of the viaduct influenced the shape of the structure. It should be noted that in plan, the viaduct is not on a straight line but is slightly curved at this location. A series of parametric studies using BIM technologies were performed, in order to establish a harmonic relationship between the curved and the straight lines of the ribbon in both plan and elevation. This had a significant influence on the height, shape and inclination of the arch. Too tall an arch would benefit its stiffness but destroy the harmony with the adjacent straight line. Too shallow, it would benefit the aforementioned harmony but make the arch too slender for use as a railway bridge, whose stiffness is profoundly important for the safe operation of a railway itself.

The Promoter sought a structure, which was robust and straightforward to maintain, but these aspirations had to be balanced with the need for an architecturally very high quality structure at a visually sensitive location. A parametric analysis was performed comparing the behaviour of the structure with Open and Box Sections. Open sections offer easy access for inspection compared to boxes. However, in order to achieve the targeted appearance the amount of additional required stiffening deemed this solution impractical. Following the aforementioned parametric study, it was decided that the main arch and bracings would comprise box sections, with the remainder of the steel structure being fabricated from open steel sections. The choice of steel type was also important, as it affects both the aesthetics and the maintenance requirements. Weathering steel was chosen for the main arches and bracings and painted steel for the longitudinal ties and transverse girders. The choice of weathering steel, generally, minimises the whole life maintenance costs. However, a separate study was performed to ascertain, how any graffiti could be successfully removed from the weathering steel without damage to the stabilising patina.



*Fig. 5. Network arch elevation and typical section.*

Fig. 5 illustrates an elevation and typical section of the structure. The rise to span ratio was set at 15.25, which is at the lower range for a network arch. In order to enhance the appearance of the arches a 6 degree inward inclination was prescribed at planning stage.

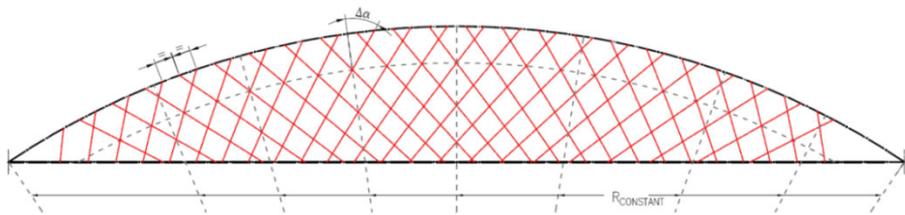
The solution was a good compromise between aesthetic and maintenance requirements and the collaboration between the Planning Authorities, the Architect, the Promoter, the Contractor, the Bridge Designer and the Steelwork Fabricator was instrumental in achieving the final structural configuration.

This arrangement, in combination with the required construction sequence prescribed by the need to keep a clear 7m corridor at any time for navigation purposes, and also the need to economise both from a budget and programme perspectives, posed significant challenges to the Bridge Designer and the Contractor. The approach to designing this structure, as well as the challenges and the solutions given are presented below.

## 2. SUPERSTRUCTURE

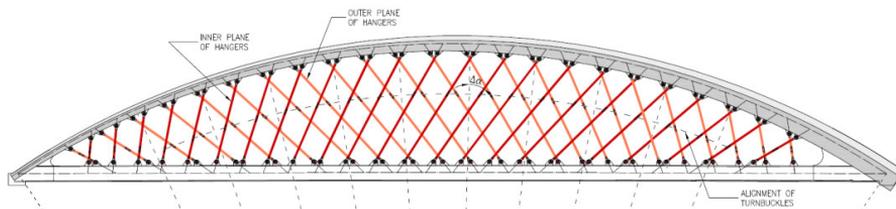
### 2.1. Hanger network definition

The layout of the hanger network is generally based on the theoretical concept of directing the resultant network forces radially with respect to the arch axis. This is achieved by aligning the intersections of the hangers radially towards a common focal point and is illustrated in Fig. 6.



*Fig. 6. Basic principles for the hanger network development.*

The actual layout of the hanger network was developed through a series of collaborative studies investigating its efficiency in terms of maximum and minimum hanger forces and the resulting bending moment profiles in the arches and the tie beams. The studies were based on the intact “wished-in-place” geometry and looked at layouts, where the anchorage nodes are equally spaced along the arch axis or along the tie beam axis.



*Fig. 7. Actual hanger network layout.*

In order to facilitate the replacement of hangers and their stressing during construction, each network was split into two parallel planes. The theoretically derived layout had to

be adjusted to provide sufficient space for the development of the anchorages and, in the case of the short hangers, to allow the installation of the stressing equipment.

The alignment of the hangers was further adjusted to provide a more harmonious appearance of the hanger anchorages along the arch soffit. The resulting layout is shown in Fig. 7.

The network is formed of 46 proprietary tension assemblies, comprising solid steel bars and cast fork anchorages. The basic principle illustrated earlier was maintained but the crossing angle  $\Delta\alpha$  was adjusted.

## 2.2. Arch geometry

The main arches have a continuously varying cross section that forms a “crease” line, which is visually continuous from the tip of the arch to the end of the approach viaduct. The shape of the hexagonal cross section for the arch was carefully designed so that varying curvature, or “warping”, of the plates is avoided.

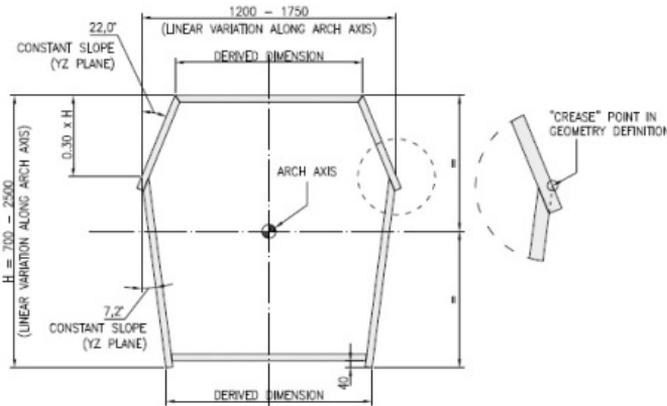
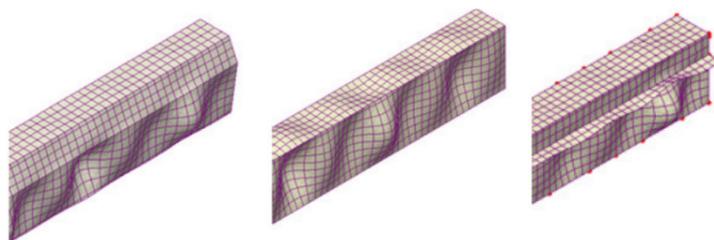


Fig. 8. Arch cross section definition.

This was achieved by maintaining constant inclination of the upper and lower plates with respect to the true vertical plane perpendicular to the axis of the bridge. The cross section definition is presented in Fig. 8.

The “ribbon” theme for the Chord resulted in a very deep arch cross section towards the North end of the bridge. The material distribution along the arch compensates for the increase of section by reducing the plate thickness. This reduction was balanced against the requirement for introduction of longitudinal stiffeners since the latter in combination with the hanger anchorages would lead to congestion within the box.

The design approach utilised the presence of the visual “crease” line by evaluating its stiffening characteristics. A study was carried out comparing the local buckling characteristics of the folded section against an equivalent box with and without effective longitudinal stiffeners at the level of the crease point. The buckling modes for the three cases are presented in Fig. 9.



*Fig. 9. Comparison between actual geometry, parallel web box and stiffened box.*

It was found that the performance of the creased box is almost identical to the stiffened box and more importantly, the local buckling of the web plates was contained either side of the creased line. The effectiveness of the “crease” is measured using the ratio of the eigenvalues for the first web buckling mode. The comparison is presented in Tab. 1.

*Table 1. Comparison between normalised eigenvalues.*

| Case                     | Normalised eigenvalue |
|--------------------------|-----------------------|
| Creased web              | 0.89                  |
| Vertical unstiffened web | 0.55                  |
| Vertical stiffened web   | 1.00                  |

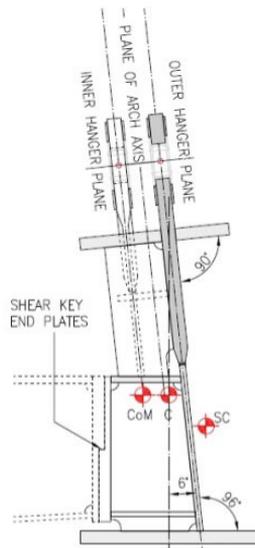
### 2.3. Deck geometry

The aspiration for ease of maintenance was most influential in the development of the deck geometry. Although a closed box would have been a more convenient structural form, an open section geometry was adopted so that touching distance inspection could be undertaken for all deck steelwork not encased in concrete, without the need for confined space inspections. In order to avoid unnecessary bending effects, the arch and tie axes intersect over the centreline of the abutment bearings. In order to maintain direct load path from the hangers into the tie beam, the web was aligned with the plane of the hanger network. The bottom flange of the section was maintained truly horizontal to facilitate site installation. The resulting cross section is presented in Fig. 10.

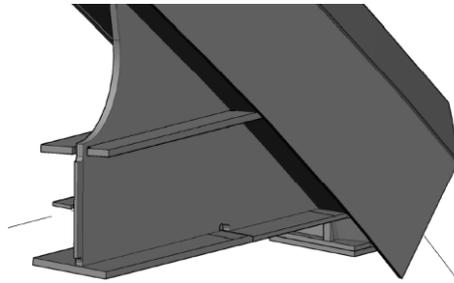
The deck was also designed to incorporate the Promoter’s requirement for a connection that provides a secondary load path for the vertical loads and this is formed by bolted shear key end plate connections for the transverse girders. These were positioned within the footprint of the tie beam bottom flange so that the visual aspiration for “clean” soffit is achieved.

The prominence of the exposed arch surfaces means that their interruption is undesirable since the visual continuity will be compromised. With that in mind the junction between the arch and the tie beam was a defining interface with respect to the cross section geometry. This interface is shown for the North End node in Fig. 11.

As it can be seen from Fig. 10, the centre of mass, the resisting section centroid and the shear centre of the section do not match. Merging these points would have led to an arrangement that would have been detrimental to the appearance of the structure.



*Fig. 10. Typical tie beam cross section.*



*Fig. 11. Tie beam - Arch junction at the North end of the bridge.*

The misalignment of the shear centre and the centre of mass meant that twisting deformations and warping would be induced in the tie section. This was of particular importance for the analysis during erection, since once the composite deck is complete, the deck stiffness is significantly improved. These effects would then become negligible for the superimposed dead and live loads.

The alignment of the web with the outer plane of hangers served two purposes. It provided torsional moments from the stressing of the hangers opposing the ones generated by the permanent loads and avoids the use of external stiffeners, which improves the appearance of the longitudinal tie significantly.

#### 2.4. Erection methodology

It was recognised from the very early stages of the project that the design of this structure was heavily dependent on the construction methodology to be adopted. The

aforementioned methodology was heavily influenced by the requirement to have navigable channel in the river for the duration of the construction. Furthermore, in order to reduce the impact of the temporary works on the construction programme, it was decided to use the same temporary works to facilitate the demolition of Prince’s Bridge. This effectively led to a “piecemeal” erection approach and necessitated in depth discussions between the steelwork Fabricator, the Bridge Designer, and the Contractor, in order to determine the exact methodology, from which the design of the structure evolved.

The tie beams will be erected first in sections followed by the transverse girders and the arches. The alignment of the bridge with respect to the river meant that the deck will have to be installed on skewed temporary towers in the river, as shown in Fig. 12.

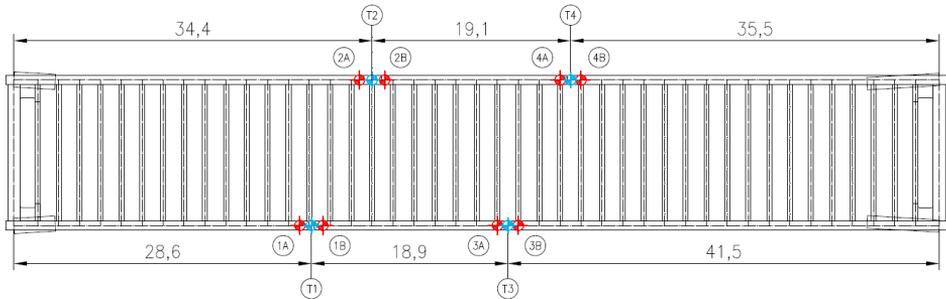


Fig. 12. Skew layout of the temporary supports.

For reasons explained in the deck geometry section, when the tie beam sections are installed, they will deflect and twist. In order to understand the behaviour of the asymmetric tie beam sections better, the deck erection sequence was re-analysed using a shell-element based model for the ladder deck. An illustration of the predicted deformed shape when the deck beams are installed is presented in Fig. 13.

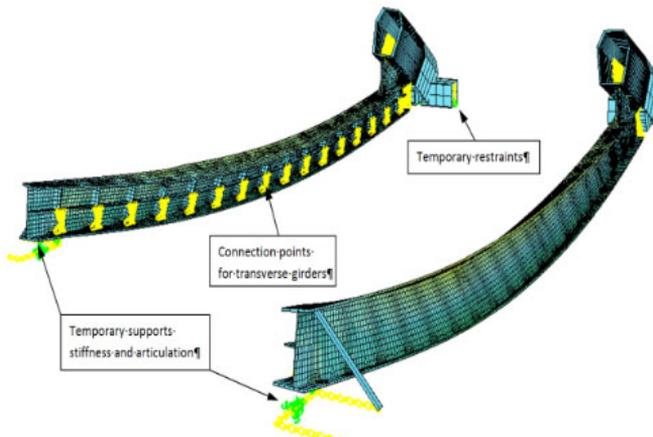
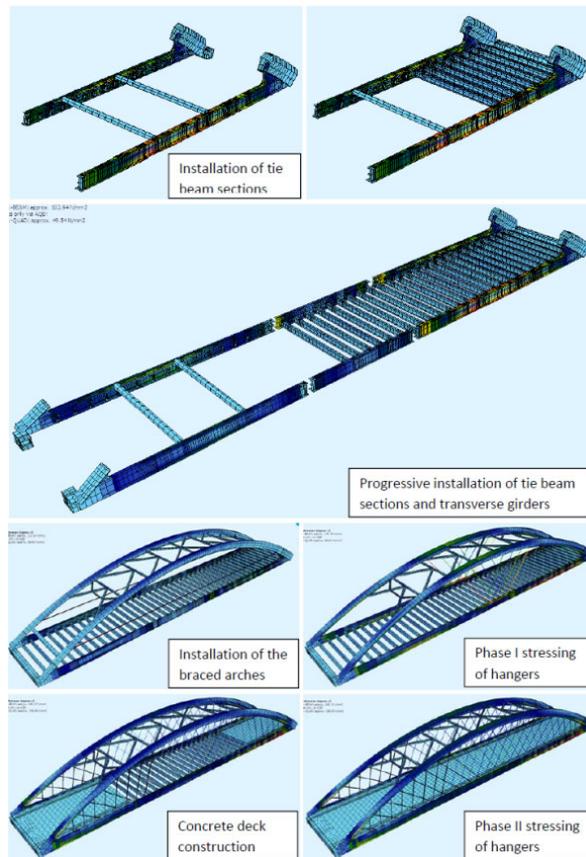


Fig. 13. Installation of the tie beam sections.



**Fig. 14.** Summary of erection sequence.

Such analysis was required to evaluate the stability of the tie beam sections in the temporary case since the cross section does not have an axis of symmetry. It was also required, in order to understand the build-up of rotations about the longitudinal axis of the ties. The open section has relatively low torsional stiffness, which meant that the twist generated during erection may lead to problems during the installation of the hangers. This risk was mitigated by incorporating transverse props and ties that assist in the installation of the transverse girders. Grade 460 high strength steel was adopted for the clevis plates and incorporated hemi-spherical bearings to enhance the out-of-plane rotational capacity, thus providing greater installation tolerance.

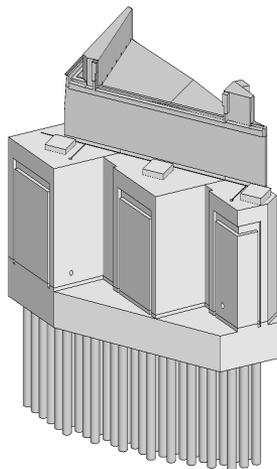
The approach in BS EN 1993-1-1 was adopted where the normalised slenderness was derived from the load amplifiers for strength and linear elastic buckling for a number of discrete erection stages. The “piecemeal” erection methodology induces bending moments in the tie beams that are generally avoided in network arches. In order to minimise the magnitude of these moments, the stressing of the hangers commences as soon as the structural steelwork is erected.

This first phase of hanger stressing relieves the temporary works and transfers the gravity loads in the arches and the tie beams. The hanger installation and stressing pattern is aimed at avoiding “slack” hangers in working conditions. The stressing commences from the centre outwards so that the temporary towers are relieved after the first few stressing operations. Once the first phase of stressing is complete the concrete deck is constructed in sections. Following the initial cure of the deck slab a second phase of hanger force tuning is applied so that the target in-service performance of the hanger network is achieved. The erection sequence is briefly described in Fig. 14.

### 3. SUBSTRUCTURE

#### 3.1. South Abutment

An illustration of the South abutment is shown in Fig. 15. The cube shaped abutment above ground was determined by the aesthetic requirements and the need to replicate part of the existing zig-zag viaduct to the East of the Bridge. As far as the part of the abutment below the ground is concerned, it was influenced by three requirements. In order to minimise the effect on the neighbouring Grade II listed structure (Stephenson’s Viaduct), the required piles had to be as small as possible. This was also required from the river training strength perspective, as the bigger the piles, the heavier the piling rig to install the piles will be required. That would also require a significant amount of temporary works in the river. At the same time, the Promoter wanted a robust structure, the stability of which does not rely on the stability of the river training walls. Taking advantage of the relatively shallow depth of bedrock (5-6m) at this location, it was decided that a “transfer platform” will be created by the installation of closely spaced 600mm diameter bored piles, thus taking into account the confinement of the rock by the closely spaced piles. This foundation will effectively function as a raft foundation but it will be constructed from ground level, without affecting the stability of neither the Stephenson’s viaduct nor the river training walls. Furthermore, the closely spaced piles would act as columns supporting the abutment, in case the river training wall collapsed in the future, thus satisfying the Promoter’s requirement.



*Fig. 15. South abutment.*

### 3.2. North abutment

On the North side of the River Irwell the bedrock is at a much shallower level and approximately 3m below ground finished level. The structure is presented on the right hand side of Fig. 2. Creating a similar transfer platform at the north would not be possible. This is because the track designer required that the track fixity point is set at this location. This results in a significant horizontal load being resisted by this abutment. Therefore, a raft foundation was designed at this location to support the bridge abutment.

A full track structure interaction for this curved track alignment was performed from first principles, in order to justify the forces to be transferred on this foundation and also the implications of temperature and other longitudinal loads on the track design itself.

Another function of the North Abutment is also to form a transition from the network arch box section to the half through I-plate girder of the neighbouring structure to the North, Trinity Way Bridge. This transition was very challenging as the River Irwell Arches and Trinity Way Bridge are not in a straight line in plan. In order to achieve seamless visual continuity that would satisfy the aesthetic requirements for the aforementioned ribbon theme of the Orshall Chord, trial and error curves were devised and the use of BIM was very important in defining successfully this transition in a 3D space.

## 4. CONCLUSIONS

An 89m, twin network arch bridge was designed to span the River Irwell. The Bridge will carry a twin, bidirectional track arrangement as part of the Ordsall Chord intervention. A number of requirements and constraints were imposed, which posed challenges to both the design and construction teams. Effective use of BIM technology and processes and the adoption of a collaborative approach by all parties involved, including the early appointment of the steelwork Fabricator were catalysts to the successful completion of the design phase of the structure. This is the first network arch bridge in the United Kingdom. It is also believed that the particular arrangement of asymmetric arches, open section longitudinal ties and type of loading constitute a world's first variation of a network arch bridge. The structure is currently under construction and due for completion in 2017.

## ACKNOWLEDGEMENTS

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