Detailed design of the main bridge – viaduct spans in Padma Multipurpose Bridge project, Bangladesh

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ABSTRACT: The Padma Multipurpose Bridge Design Project comprises a new fixed crossing of the Padma River in Bangladesh, which will consist of a new main bridge approximately 6.15km long with spans crossing the Padma River. Viaduct spans on both sides of the Padma River support both the highway and railway traffic and lead this traffic onto the upper and lower levels of the main bridge. The length of the approach road viaducts ranges from 720m to 875m long. The length of the railway viaducts ranges from 2.36 km to 2.96 km. The superstructure of the approach road viaducts consists of precast, pre-tensioned concrete Super-T girders which will become the first Super-T girder structure to be constructed in Bangladesh.

1 INTRODUCTION

The viaduct spans are separated into the approach road and the railway viaducts. The main bridge is a two level structure which required a complex arrangement of the viaducts to separate the railway from the highway and alternative options were considered during the Scheme Design Phase of the project.

There are a total of four viaducts supporting the highway, two on each side of the river. The approach road viaducts range from 720m to 875m long and comprise 38m spans. The superstructure consists of precast, pre-tensioned concrete Super-T girders which will become the first Super-T girder structure to be constructed in Bangladesh. The Super-T girder is an economical beam commonly used on highway bridges in Australia and is becoming more widespread on projects throughout Asia. The introduction of the Super-T girder to Bangladesh presents an opportunity for future use on other projects throughout the country. This paper describes the design features of the Super-T girder.

There are two viaducts supporting the railway, one on each side of the river. The railway viaducts range from 2.36km to 2.96km and they also comprise 38m spans similar to the approach road viaducts. The superstructure consists of precast, post-tensioned concrete I-girders. The railway loading adopted is one of the heaviest used for railway bridges in the world. This paper describes some of the design features of the railway viaduct to withstand these heavy loads.

The detailed design of the viaduct structures posed some major challenges in bridge engineering specifically involving earthquakes under soil conditions highly susceptible to significant depths of liquefaction. A multi modal response spectra analysis was used to analyse and design the viaducts for a seismic event with a return period of 475 years. This paper describes the dynamic analysis procedure and the design features of the structure to withstand these seismic events.

A transition pier is located at the interface of the viaduct spans to the river spans and supports the end spans of the main bridge, the approach road viaduct structure and the railway viaduct structure. The transition pier also provides a location for the diversion of the gas pipe, power cables and telecommunication utilities off the main bridge whilst also enclosing an access stairwell for inspection, maintenance and emergency evacuations.
2 APPROACH ROAD VIADUCTS

2.1 General description

The Padma Multipurpose Bridge provides a fixed crossing for a four lane dual carriageway highway and a single line broad gauge railway over the Padma River in Bangladesh. The river bank on the northern approach of the bridge is located at Mawa. The river bank on the southern approach of the bridge is located at Janjira. The approach road viaducts are designed to accommodate the road and railway onto the main bridge. The road connects to the existing Dhaka-Mawa highway on the Mawa side of the river. The road connects to the existing NH8 highway on the Janjira side of the river.

The structure of the river spans of the main bridge is a two level steel truss with a composite deck slab. The railway is supported on the lower level and the highway is supported on a concrete deck slab at the upper level. The purpose of the approach road viaducts is to transition the highway from the main bridge to the approach embankments. The alignment design of the approach road viaducts has a complex arrangement. The highway is located above the railway which requires separation of the two carriageways otherwise the approach viaducts would extend for kilometres beyond the main bridge.

The northbound and southbound carriageways split outwards into two structures away from the railway. As the viaduct structures become clear of the railway, the alignment transitions into a steeper vertical grade to reach the approach embankments in the shortest distance possible. The southbound carriageway then curves back inwards to pass beneath the railway viaduct structure and to join up with the northbound carriageway.

The length of the approach road viaducts ranges from 720m to 875m long and comprise nominal 38m spans (Figure 1). The number of spans ranges from 19 spans to 23 spans. The superstructure consists of simply supported 1800mm deep pre-tensioned precast concrete Super-T girders and a cast-in-situ composite deck slab. An asphalt layer is installed on the deck slab to form the wearing surface of the bridge. Traffic barriers are located on both edges of the viaducts.

2.2 Scheme Design

During the scheme design phase, various options and alternatives were considered for the approach viaducts. A two-level superstructure of the main bridge was the preferred option for the scheme design. The railway is located directly beneath the highway which presented a challenge in determining the structural arrangement of the approach viaducts (Figure 2). Continuing the two level steel truss to ground level would be too expen-
sive, so different superstructure forms were considered to support the railway and highway to the approach embankments.

The most cost effective solution for the viaduct spans was a typical beam and slab superstructure and this was selected as the most preferred option for the scheme design (Figure 3). Many beam types were considered although with the consideration of a precast yard, the pre-tensioned precast concrete Super-T girder proved to be the most economical solution.

Two alternatives were considered for the alignment of the highway to separate the highway from the railway. One option was for the alignment of both carriageways to curve outwards in the same direction to overpass the railway. The other option was to split the alignment of the carriageways in two and curve the alignment outwards in opposite directions. This alternative of splitting the carriageways required the shortest distance of viaduct structure before reaching the ground level and was thus selected as the preferred option (Figure 4).

Figure 3. Ground view of the approach viaducts leading onto the main bridge at the transition pier.

Figure 4. Aerial view of the approach viaducts at Mawa.

2.3 Detailed Design

The detailed design phase of the viaduct spans required a global linear static analysis, a global dynamic analysis and local grillage analysis to complete the design of the viaduct spans. The seismic analysis procedures were the most important and challenging process as this was the governing criteria for the majority of the elements of the bridge.

2.3.1 Design Criteria

The design of the approach road viaducts were carried out in accordance with British Standards BS 5400 Steel, Concrete and Composite Bridges. However the British Standards do not cover the design of bridges for seismic events so AASHTO LRFD Bridge Design Specifications was used to design the viaduct spans for earthquake effects.

The traffic loading includes the HA and HB traffic loading in accordance with the British Standards. In Bangladesh, a large number of vehicles are heavily loaded or overloaded. The HB 25 vehicle and the HB 45 vehicles have both been considered in the design. The axle loads of these vehicles are 25T and 45T and reflect the current and potential loading conditions in Bangladesh for the next 100 years.

The most significant loading condition for the viaduct spans is the seismic loading. The viaduct spans are designed for a seismic event with a return period of 475 years. The following parameters were used in the design to AASHTO LRFD:

- Acceleration coefficient: 0.143g
- Seismic Zone: 2
- Peak ground acceleration: 0.1414 m/s²
- Soil Profile: Type II
- Site coefficient: 1.2

The following response modification factors were used in the design of these elements.

- Pier columns: 2.5
- Connections: 1.0
- Piles: 2.0 (bending moments)
- Piles: 1.0 (axial and shear forces)
Under a seismic event, the top layers of the soil will liquefy to a depth of approximately 20m below the existing ground level. In the analysis and design for seismic, the lateral restraint and skin resistance of the soil has been ignored over the liquefiable depth.

2.3.2 Superstructure

The superstructure consists of simply supported 1800mm deep pre-tensioned precast concrete Super-T girders with a cast-in-situ composite deck slab. The girders are supported on laminated elastomeric bearings and have a nominal span length of 38 metres. The superstructure is divided into three modules for each viaduct. Expansion joints are located at the abutments, the connection to the main bridge, and at intermediate locations to separate the modules. The superstructure is restrained horizontally at the piers which causes the structure to behave similar to a portal frame structure.

2.3.3 Substructure

The substructure consists of slender rectangular reinforced concrete pier columns. At the top of the piers, the width of the pier columns taper outwards to enhance the aesthetic appearance of the bridge (Figure 2). The pier columns are 1300mm to 1500mm thick and the width varies from 4000mm up to 6000mm at the top of the column. The pier columns are supported on a reinforced concrete pile cap. The pile cap is located underneath the existing ground level to enhance the aesthetic appearance of the bridge. The foundations are bored piles with a diameter of 1200mm to 1500mm and extend to a depth of up to 50m below the existing ground level.

3 DYNAMIC ANALYSIS OF THE MAIN BRIDGE VIADUCT SPANS

3.1 General

The AASHTO (2007) LRFD bridge design specification was selected as the most appropriate standard to use for the analysis and design for a seismic event for the viaduct spans. The purpose of a seismic analysis is to determine the force and displacement demands on the bridge under a specified seismic event applicable to the bridge environment. Earthquake loading is an ultimate limit state event and it is not economical for a structure to be designed to behave within the elastic range. The objective of the AASHTO (2007) standard is to design and detail the structure to potentially suffer damage during a seismic event but should have a low probability of collapse.

3.2 The multi-modal spectral analysis procedure

In a multi-modal spectral analysis, structural analysis software is used to perform a dynamic response analysis subject to the earthquake loading input data given in the form of acceleration response spectra. The analysis considers the vibration of the structure and analyses mass load cases to determine the natural frequency, period and mode shapes for a user defined number of modes. The spectral analysis then considers this data to compute the deflections, forces, moments and reactions in the structural elements.

In accordance with AASHTO (2007), the following considerations shall be used in a multi-modal spectral analysis:

- A linear dynamic analysis using a three dimensional model shall be used to represent the structure.
- The number of modes used in the analysis should be at least three times the number of spans in the model.
- The member forces and displacements may be estimated by combining the respective response quantities (moment, force, displacement, or relative displacement by the Complete Quadratic Combination (CQC) method.

Chen & Duan (2003) and Caltrans (1999) describe some of the major considerations to be taken into account when developing a structural computer model.

- The number of degrees of freedom and the number of modes considered in the analysis shall be sufficient to capture at least 90% mass participation in the longitudinal and transverse directions.
- A minimum of three elements per column and four elements per span shall be used in the model.
- The effective stiffness of the components should be used in order to obtain realistic evaluation of the structures period and displacement demands and include the effects of concrete cracking, rein-
force and axial loads for concrete components, residual stresses, out-of-straightness and the
restraints of the surrounding soil of the piles.
- For ductile concrete column members, effective moments of inertia \( I_{\text{eff}} \) should be based on
  cracked section properties and can be determined from the initial slope of the moment curvature
  curve between the origin and the point representing first yield of reinforcement. This is defined by
  the following equation:

  \[
  E_s \times I_{\text{eff}} = \frac{M_y}{\varphi_y}
  \]

- The effective torsional moment of inertia \( J_{\text{eff}} \) of the concrete column shall be used and taken as
  20 percent of the uncracked section properties.
- For prestressed concrete superstructures, \( J_{\text{eff}} \) is assumed the same as \( I_{\text{gross}} \) because prestress-
ing steel limits the cracking of concrete superstructures.
- Soil spring elements should be used to model the soil-foundation-structure interaction.

The interpretation of the results and understanding of the dynamic response of the structure is important in
developing the most economical design. The results also require interpretation to determine the correct sec-
tional properties to be used in the model. Further iteration of the analysis is required by altering the section
properties of the cracked pier to represent the effective moment of inertia. This alters the structural response
and further iterations may be required.

3.3 Global analysis of the main bridge viaduct spans

The superstructure of the main bridge viaduct spans was isolated into three modules between the expansion
joints. As each module is isolated, the structural computer models can deal with each separate module, reduc-
ing the computational effort required. A typical structural computer model for a module is shown in Figure 5.

The superstructure was modeled as a line beam in the model using the sectional properties of the full cross
section. The alignment of the superstructure includes horizontal and vertical curves. The curved superstruc-
ture has been modeled as a series of straight members, as chords on the curve, between spans to represent the
approximate geometry. The superstructure elements are connected to the vertical pier members. The vertical
pier members were initially modeled with the full gross sectional properties of the reinforced concrete pier.
Cracked sectional properties were then inserted at the base members of cracked pier locations during the it-
ervations of the analysis.

Pilecaps and piles are also modeled with corresponding gross sectional properties. Member releases have
also been used in the model to represent the rotational characteristics of the simply supported superstructure
and the translational characteristics of the structure at expansion joint locations.

![Figure 5. Computer structural model used representing the module of the viaduct spans nearest to the main bridge.](image)

Force-displacement spring restraints were applied to nodes on the piles to represent the soil-foundation-
structure interaction. The stiffnesses of the spring restraints were calculated from the moduli of lateral sub-
grade reactions which were estimated from the N-values obtained from the standard penetrometer tests (SPT)
carried out on boreholes drilled at the bridge location.

All the applicable loads and load combinations were applied to the structural model which included all
dead loads, superimposed dead loads, live loads and other applicable transient loads. The global analysis for a
seismic event however only includes the dead loads, superimposed dead loads, one third of the highway live
load and the railway loading.
Under a seismic event however, it was found that the top layers of the soil would liquefy to a depth of approximately 20 metres. Therefore a further, revised model was used for the seismic load cases with the node spring restraints on the piles removed over the liquefaction depth. This also significantly changed the structural response of the structure.

The acceleration response spectrum (Figure 6) was included in the model as spectral load data. This is then modeled in combination with the applicable mass load case in the longitudinal and lateral directions for a specified number of modes. This resulting dynamic analysis determines the dynamic response of the structure. The response modification factors were also input into the model and used to determine the results of the analysis.

![Figure 6. Normalised acceleration response spectrum used for the viaduct spans.](image)

The results of the non-seismic and seismic load cases are then interpreted to understand the behavior of the structure. The seismic load cases are also combined with the other load cases such as creep and shrinkage effects. The seismic analysis was by far the governing load case for the substructure elements.

It was found that the bending moments in the base of the piers were greater than the cracking moment of the reinforced concrete section. Therefore the period of the structure and the results of the analysis were not quite accurate. A moment curvature diagram was developed for the reinforced concrete sections of the pier to determine the effective moment of inertia to be used in the model at the base of the cracked piers.

A second iteration of the analysis was then carried out which gave more accurate structural periods and results. The results were then used to size the sections and define the reinforcement. Alterations to the cross section or reinforcement changed the effective moment of inertia and further iterations of analysis were required until the correct balance was achieved.

### 3.4 Second order analysis

In a seismic event, the earth oscillates horizontally (and vertically), which causes the structure to translate. As the superstructure translates and the pier columns undergo curvature, the vertical superstructure forces on the column are offset from the neutral axis of the column. In addition, the axial forces within the column itself also act at an eccentricity to the column neutral axis. These eccentric loadings cause additional bending moments in the column causing further translation and bending until equilibrium is achieved.

In slender columns, the second order effects are much more significant and must be taken into consideration into the design. In a simplified calculation, a moment magnification factor is derived from the effective lengths of the members and used to amplify the bending moment results to be used in the design. This method is used as a simplified alternative to carrying out a second order analysis and is known to generate conservative results.

However in slender columns, the moment magnification method can result in an excessive increase to the bending moments. Carrying out a second order analysis requires more design effort, although the results are much more accurate and tend to be less conservative. The second order analysis, as carried out on the viaduct spans, resulted in an increased bending moment in the range of 20-50 percent depending on the slenderness. Due to the high depths of liquefaction, in a seismic event in combination with slender piers, the translation of the structure became quite significant which penalized the structure with second order effects.

The second order analysis, carried out on the viaduct spans, resulted in an increased bending moment in the range of 20-50 percent depending on the slenderness. Due to the high depths of liquefaction during a seismic event, the translation of the slender piers is quite significant, and the structure is subject to significant second order effects.

It is important to note that the second order analysis should be carried out on the seismic load cases with a response factor of 1.0 which gives the actual deflections during an earthquake. It is also important to calculate the second order effects on the total deflections of the structure for the combined load case, rather than adding the second order effects for each individual load case.
4 PRE-TENSIONED PRECAST CONCRETE SUPER-T GIRDERS

4.1 Background

The Super-T girder is a bridge beam that was developed in Australia during the 1990’s. Prior to the 1990’s, the precast concrete I-girder was one of the most commonly selected beam for medium span bridges. The I-girder is considered as an effective beam, although collaboration between the precast and construction industries identified that a beam could be developed with greater versatility that would result in significant improvements to design efficiency, manufacturing, constructability, transportation and safety.

Connal (2010) discusses the development and history of the Super-T beam which originated from collaboration between the precast and construction industries and designers, and was aimed at satisfying several objectives, namely:

- Easy manufacture and multi-use of a single outer form for the full range of beam sections;
- The ability to achieve a daily casting cycle to allow high production rates;
- Use on bridges with spans of 18 to 36m, including for bridges with modest curvature;
- Minimisation of on-site formwork and the creation of a continuous, safe deck working surface when the beams are assembled edge to edge immediately on installation;
- Simplified prestressing using straight pre-tensioned strand, with some strands debonded at the beam ends to control transfer stresses; and
- A stable beam shape that does not require any added support to retain lateral stability during transfer or erection.

Connal (2010) also states that many of the bridges constructed in the last ten years in Australia have been with superstructures using Super-T girders. The Super-T girder, for bridges in the 18m to 36m span range, has been found to be the most economical form of bridge beam for a wide range of applications. Australian bridge engineers working on international projects have ‘exported’ this development to other countries including Vietnam, Philippines and Malaysia.

The Padma Multipurpose Bridge Project will be the first project in Bangladesh to construct a bridge with Super-T girders. The introduction of Super-T girders on this project will introduce a new technology of precast bridge beams to the construction industry. The industry is currently limited to manufacturing post-tensioned beams and this project will highlight the potential benefits of manufacturing pre-tensioned beams as an alternative for future projects in Bangladesh.

4.2 Cross section

The Super-T beam can be manufactured in sizes ranging from 750mm – 1800mm deep and can be used on bridges with a span length of 18 – 38m, depending on the live load. The Super-T girder is a ‘T’ shaped girder and consists of a 75mm thick open top flange, a 100-120mm minimum thick vertical web and a 260 - 325mm thick bottom flange (Figure 7a).

The top flange acts as formwork and the open section of the top flange has a recess to place sacrificial formwork. The girders are erected side by side and with the formwork in place, a flat safe working area is created to construct the deck slab (Figure 7b).

The bottom flange contains the pre-tensioned strands which are arranged in rows of 50mm spacing. Two strands are located in the top flange to assist in minimizing tensile stresses during transfer. The strands are straight for the full length of the beam. Internal diaphragms are located at intermediate locations (Figure 8 and 9). The purpose of the intermediate diaphragm is to prevent any web distortions of the beam, particularly during transport and erection. Solid end blocks are also located at the ends of the girders. The purpose of the end
blocks is to resist the bursting stresses during the stressing of the girder. The end blocks are also required to increase the shear capacity of the section where the shear forces are highest near the supports.

Figure 8. Typical Super-T girder elevation.

Figure 9. Typical Super-T girder plan view

4.3 Materials

The materials used for Super-T girders are designed to give the girder a design life of 100 years. The following materials are typically used for Super-T girders.

- Concrete: 28 day characteristic cube strength of 60 MPa.
- Reinforcement: Grade 460 - 500 MPa
- Prestressing Strands: 7 wire, stress relieved, low relaxation strand 12.7 – 15.2mm diameter.

Lower transfer strengths are nominated to ensure just adequate concrete strength at transfer and to ensure adequate production rates of manufacturing. However it is also quite common for the precast manufacturers to use higher strengths of concrete so that the transfer strength is reached earlier and production rates are increased. The maximum jacking force of the strands is limited to 80 percent of the ultimate tensile strength. Failure of prestressing strands under tension contains a high risk to safety and it is normal practice to limit the jacking force to 75 percent of ultimate tensile strength.

4.4 Design at transfer

Transfer is the stage where the pre-tensioned strands are released to transfer an axial compression force to the precast beam. To allow gradual introduction of prestress and to limit the stresses in the top of the beams at their ends, some of the strands are debonded from the concrete at the beam ends with plastic tape or tubes (Figure 8). A high prestressing force is not required at the beam ends as the applied bending moments are not as high at these locations.

4.5 Design for resistance to bending, shear and torsion

The Super-T girder is designed to control cracking at the serviceability limit state and to prevent failure at the ultimate limit state. Cracking is controlled at the serviceability limit state by limiting the compressive and tensile stresses in the concrete beam. At the ultimate limit state, the capacity of the beam is limited by the tensile stress in the steel prestressing strands in the bottom flange.

Depending on the live load criteria used in the design, it is usually found that the design is governed by bending. The objective during the design is to minimize the stresses at transfer and the serviceability limit state whilst preventing failure at the ultimate limit state. The design usually requires a balance between the three design requirements to achieve the most efficient design.

The prestress and vertical reinforcement in the webs of the girder is used to provide the shear and torsion capacity of the beam. The reinforcement extends from the bottom flange to the deck slab. After the deck slab is cast and becomes composite with the Super-T, the beam becomes a box section with good torsional stiffness.
4.6 Manufacturing

The manufacturing procedure of the Super-T girders is usually carried out in a precast facility or a precast yard. Long casting beds can be set up to allow the manufacturing of several girders at a time. The steel reinforcement is usually prefabricated and then fixed in place with the prestressing strands anchored to a bulkhead. The strands are then tensioned by jacks at one end and concrete is cast into the moulds to the required shape of the girder. The bottom of the mould is usually movable to allow manufacturing of different beam depths whilst maintaining the same shape in the upper section. Inner moulds are also used to achieve the shape of the void and intermediate diaphragms. The webs of the Super-T and intermediate diaphragms are tapered to allow easy removal of beam from the outer mould and the inner mould. Once the concrete has reached the transfer strength, the strands are cut and the beams are ready to be transported to site.

4.7 Transportation

The Super-T beams have good stiffness properties for bending in the non-principal axis which gives the beam rigidity during lifting and transportation. The beams are easily transported using a prime mover at one end of the beam and an independently steered jinker at the other end of the beam (Figure 10).

Figure 10. Transportation of Super-T girders using a prime mover and steered jinker.

4.8 Constructability

Another benefit of the Super-T girders is the versatility in erection techniques. The girders can either be erected by mobile cranes, or they can be erected span by span with a launching gantry (Figure 11) where the girders are delivered to the most recently constructed span. Alternatively, the girders can be installed with a vertical lift erection gantry, where the girders are delivered via the ground level (Figure 12).

The most dominant feature of the Super-T girder is the safe working platform that the beams create once erected. The outer flanges are erected edge to edge to the adjacent girder and sacrificial formwork is placed in the open section. It is also possible to erect handrails on the edge girders, which will eliminate any potential falls.

Figure 11. Girders erected with a launching gantry.     Figure 12. Girders erected with an erection gantry.

5 RAILWAY VIADUCTS

5.1 General Arrangement

There are a two viaducts supporting the railway, one on each side of the river. The length of the railway viaducts ranges from 2.36km to 2.96km and consists of 38m spans similar to the approach road viaducts. Railway stations are located on both the Mawa and Janjira sides of the river where the railway viaduct structures
reach the embankment level. The maximum vertical grade of the railway is 0.5% which causes the railways to extend a long way beyond the main bridge before reaching the embankment level.

5.2 Design Criteria
The railway live loading is one of the heaviest railway loading criterion in the world. A provision has been made for a single track broad gauge railway with a design operating speed of 160km/h for passenger trains and 125km/h for freight trains. The railway viaduct is designed to form part of the Indian Railways Dedicated Freight Corridor (DFC), with the railway live load consisting of a series of 32.5T axle loads.

5.3 Superstructure Design
The superstructure consists of simply supported 2200mm deep precast, post-tensioned concrete I-girders with a composite deck slab. A track slab is incorporated into the deck slab. End diaphragms and intermediate diaphragms are located between the beams. A walkway is incorporated into the barrier and is located on both sides of the railway.

The precast beams were selected due to cost efficiency and ease of construction. In order to withstand the heavy live load, the beams are spaced closely and tied together with the intermediate and end diaphragms. The purpose of diaphragms is to distribute the railway live loading evenly between the beams such that all beams are effective in resisting the load. The deep superstructure and closely spaced beams give the overall structure a high stiffness in the principal plane of bending which enables the structure to remain within the railway deflection limits.

5.4 Substructure Design
The substructure consists of tapered rectangular reinforced concrete pier columns with a rectangular headstock. The pier column is supported on reinforced concrete pile caps which are located below the ground level. The foundations are bored piles which extend to a depth of up to 50m below the existing ground level. Some of the piers are supported by the same pile cap as the approach road viaduct and the analysis and design has been carried out in parallel with the approach road viaducts. Similar to the approach road viaducts, the design of the substructure of the railway viaducts was governed by the seismic load criteria.

6 TRANSITION PIER
The transition pier is a ‘Y’ shaped pier located at the interface of the viaduct spans to the river spans and supports the end spans of the main bridge, the approach road viaduct structure and the railway viaduct structure (Figure 3). The transition pier also provides a location for the diversion of the gas pipe, power cables and telecommunication utilities off the main bridge whilst also enclosing an access stairwell for inspection, maintenance and emergency evacuations.

The pier columns are designed with a void in the centre which encloses the access stairwell. Openings in the columns are located at the ground level and the lower level of the steel truss to provide access for maintenance staff and for emergency evacuations. The columns also enclose the power cables and the telecommunications cables and transfer them underground disclosed from the public view. The gas pipe, which is located within the truss, bends downwards at the transition pier and is attached to the riverside face of the pier. The pier has two levels of supports. The bottom level is a solid concrete plinth between the columns and supports the end spans of the steel truss and the railway viaduct girders. The upper level is a reinforced concrete cross beam spanning between the pier columns and supports the girders of the approach road viaduct.

REFERENCES
British Standards, BS5400 Steel, Concrete and Composite Bridges.