A CRITICAL ANALYSIS OF THE NEW ÅRSTA BRIDGE, STOCKHOLM

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Abstract: This paper provides a detailed study of the new Årsta Bridge in Stockholm, completed in 2005. It examines the primary design objectives set out by the client and discusses how these requirements were met through a holistic design process. Structure and loadings are addressed, with some basic quantitative analysis, with further sections focussing on the construction and serviceability. Concluding comments are made regarding the future use of the bridge, as well as highlighting some pertinent maintenance issues.

Keywords: Årsta, box girder, Foster, railway

1 Introduction

The new Årsta Bridge spans across a body of fresh water known as the Årstaviken, an attractive inlet that separates the districts of Tanto and Årsta, 2km to the south of the centre Stockholm. The Årstaviken treasured by the people of Stockholm, providing not only an unblemished organic landscape, but also a place of peace and tranquillity.

In its entirety, the city consists of 14 separate islands, rendering bridges a vital component in facilitating the mobility of the city’s population between districts. Rail travel in Stockholm is well established and has a growing subscription, which has brought about the need for regular expansion of infrastructure.

During the mid-1990s, the Swedish Rail Administrator, Banverket, identified the commuter route coming from the south into the centre of the city as a particular pinching point in the network, and addressed options for expanding the route, which hitherto had

shared just two tracks with the nationwide network serving Stockholm Central Station. The final decision was that a new two-way track was to be built to the south of the Central Station, with a new bridge being required to span the proposed new line across the Årstaviken.

At this point, two crucial factors became paramount: a bridge already existed across the Årstaviken (the East Årsta Bridge), carrying the existing train lines and was considered a precious monument to the city; while the Årstaviken, as I have already implied, is also a much cherished feature on the city’s landscape, whose gentle environment was to be stanchly maintained.

In an attempt to satisfy both of these requirements, the authorities launched an international design competition, outlining the highly prescriptive
requirements of the design. The eventual victors were Foster and Partners, who, in tandem with their engineering collaborator, Ove Arup International, produced a modest design with sensitive contextual consideration.

The bridge consists of two railways lines running conversely down the centre of the deck, with a pedestrian/cyclist walkway on one side, and a service track for rescue vehicles on the other. It is situated just 45m from the older East Årsta Bridge, which is a concrete arch bridge with a single truss arch span over the Årstaviken navigation channel, and was built during the 1920’s.

The new Årsta Bridge is constructed of a series of smooth concrete box girders, which vary in depth to produce an undulating soffit profile. The piers are elliptical in form and equally spaced to support the 78m spans of the bridge [3]. In all, the bridge has a total length of 833m.

A 40m wide navigation channel is present to the south of the Årsta Island (labelled Årsta Holmar in Fig. 1), with the adjacent piers being protected from direct boat impact by means of fendering, founded on vertical steel piles. The channel to the north of the Årsta Island is not an official navigation channel but also boasts fendering due to the presence of pleasure boats and other small vessels that frequent its waterway.

2 Aesthetics

As I have already alluded to, the primary directive of the bridge’s brief was to ensure a congruent integration of the new bridge with both the existing East Årsta Bridge, and the soft surroundings of the Årstaviken.

Foster’s final submission consisted of a simple, yet subtly elegant reinforced concrete box girder deck, supported by a series of elliptical piers, with the entire exposed visage of the bridge being a concrete mix impregnated with iron oxide [3] to award it with a rich red colouring, more sympathetic on the bridge’s environs that the more conventional grey so synonymous with the material. This, combined with use of highly textured pine formwork, is vital in creating an aesthetic which can be considered agreeable with the organic context of the surroundings.

The basic geometry of the bridge was dictated by two principle factors: the parameters imposed by the presence of the navigation channel on the south side of Årsta Island, and the aesthetic language of the existing East Årsta Bridge. The navigation channel required a vertical clearance of 26m and a width of just 40m, with an additional allowance for the piled fendering.

The concrete box girder deck varies in depth, and is most slender at mid-span. This was crucial in the sense that it not only minimises the visual impact of the bridge, but also, for those viewing the bridge from the west, the East Årsta Bridge is not unacceptably obscured by the new bridge. It would also be fair to say that if the maximum deck depth was maintained throughout the entire span of the bridge, it is likely to look unacceptably deep, with its capacity at mid-span being well in excess of any stresses that would build up in that location. The deck also incorporates a recessed channel through which the train lines are mounted, allowing sound absorbing material to be applied to the channel walls to minimise noise egress to the surrounding area.

Whilst the spans of the two bridges are not the same, the ‘rhythm’ of their spacing has been assimilated, and both bridges’ decks are at concurrent levels. As Foster & Partners are at pains to point out, “...the two bridges are in harmony with each other. They use the same rhythm of support spacing, yet they play different melodies and sound different” [4].

The use of curvaceous elements for both the bridge deck and the piers gives an impression of a structure that is more slender than is actually the case. The curvature allows light to be exploited, and the edges of members to be softened (Fig. 3). The piers are consumed into the deck, with the connection being made internally. This helps retain the monolithic appearance of the bridge, as well as keeping the soffit of the deck as clear as possible. The elliptical nature of the piers provides a soft appearance, but does mean that the bridge appears rather opaque when viewed along its length.

The parapets are lightweight and slender, with sturdy steel posts spanned by grill panels, creating a light and diaphanous appearance. This helps to minimise the perceived depth of the deck and further enhances the simplicity of the bridge’s appearance.

The structure retains its perceived structural integrity by virtue of the distribution of mass within its form. Even to an uneducated beholder, it is obvious that the point of interaction between the bridge deck and supporting piers is going to be the area where the greatest stresses are built up – justifying the thickening of the deck at that interface.

Whilst the bridge by no means appears incomplete, much of the critical structure is held latently, hidden within the structure to maintain a clean fascia. The bridge consequently takes on a monolithic appearance, with no obvious contrast made between pier and deck. Such uniformity and visual simplicity visually renders the bridge as a single mass.

3 Structure

The spine of the Årsta Bridge’s structural solution is its highly refined concrete box girder deck, which undulates both transversely and longitudinally along its length. This awards the bridge a genuinely slender figure, with an increase in the depth of the deck at the points where the bending moments are at their greatest - above the supporting piers (Fig. 4).

The two rail tracks were designed to be mounted directly onto the bridge deck, reducing the dead weight of
the structure by removing the need for heavy ballast [3]. The tracks are supported by steel cartridges which, combined with the mineral wool added to the parapets, help reduce the amount of airborne noise generated by passing trains.

Figure 4: An illustration of the change in deck depth with bending moments

Despite being a commonly utilised structural form in bridge design, the box section used for the Årsta Bridge was a heavily engineered structural element, which required extensive finite element (FE) modelling to produce optimised performance.

Two FE models were established; the first analysed the deck as a pure beam element, allowing the pre-stressing of the bridge to be optimised; and the second, modelled individual elements against applied loadings (for both SLS and ULS) [6].

Figure 5: Graphical representation of bridge structure [7]

The models were also used to produce 3D representations of the reinforcement required in the bridge to satisfy the stresses calculated from the inputted data. Such advanced design techniques are likely to have greatly reduced the amount of time spent both designing the overall bridge structure, as well as the painstaking task of reinforcing it all. The highly stressed nature of the undulating deck is represented by the fact that, on average, every cubic metre of concrete contains 220kg of unstressed reinforcement [3].

These computerised models were of particular use in understanding the complex internal force flows in the deck section caused by the pre-stressed cables. There were around 50 such cables distributed throughout the cross-section of the girders [3], and these were critical in tying together all the different segments of the bridge (usually 7/8 per 78m span). Material behaviour, such as that involved in creep, pre-stressing and shrinkage are all considered by the FE model, with temporary loading cases given time sequences to yield the best possible accuracy in the structural analysis.

As a consequence of the advanced design involved in the bridge structure, more flexible calculation methods were accommodated to ensure that the efficacy of the bridge was in no way compromised. An example of such analysis was evident is the assessment of the deck structure where longitudinal shear flows and force distributions were determined through the qualitative theory of scales [6]. Decisions such as this, where the design deviates away from the prescribed design literature, were taken with advice from a highly specialised reference group, which included a number of figures from regional technical institutions [3].

The elliptical piers spaced at every 78m were also heavily engineered structural members due to the requirements placed upon them during both construction and operation. These piers measured a maximum of 7x2.5m in section and are heavily reinforced to support the enormous weight of the bridge deck, as well as the imposed live loadings from operation and during the construction sequence.

4 Loading

All bridges are subjected to combinations of dead and imposed loadings. Other loading effects are also considered, including wind, creep and thermal expansion, as well as relevant secondary effects induced by traction and braking. In a more detailed analysis of the bridge’s loading, we would include consideration of the pedestrian track running alongside the track, as well as a service road for emergency access.

<table>
<thead>
<tr>
<th>Load</th>
<th>Load Combination</th>
<th>1</th>
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<tr>
<td>Dead</td>
<td>ULS</td>
<td>1.05</td>
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<td>SLS</td>
<td>1.20</td>
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<tr>
<td>Primary and secondary rail</td>
<td>ULS</td>
<td>1.40</td>
<td>1.20</td>
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<td></td>
<td>SLS</td>
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<tr>
<td>Temperature</td>
<td>ULS</td>
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<td>1.30</td>
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Table 1: Partial load factors (γfl)

Rail bridges are common in occurrence, and they consequently have their own set of guidelines laid out in most recognised design standards. Although designed to the Swedish design standards for rail structures, BV Bro [8], the load cases that the bridge was designed to are the universal delineations of UIC 71 and SW/2 [9]. These are equivalent to the loadings outlined in BS 5400-2:2006 [10], and it therefore more convenient to undertake our analysis using the procedures from this standard. Table 1 displays the value of the partial load factor, γfl, for given types of load in different loading combinations.

4.1 Dead and Superimposed Dead Loads

The entire bridge superstructure consists of 23,000m³ of reinforced concrete [3], which can be multiplied by its density (taken approximately as 2400kg/m³) to give an overall dead weight of 542MN. This total load equates to around 650kN/m or 33kN/m² when broken into equivalent distributed loadings. Such dead loads are then factored using γD = 1.05 at ULS and γD = 1.00 at SLS. Superimposed dead loads (e.g. surfacing...
to pedestrian walkways and service tracks) are factored using $\gamma_f = 1.75$ at ULS and $\gamma_f = 1.20$ at SLS. In more detailed analysis, the weight of the bridge’s parapet would also be included in numerical analysis.

4.2 Live Loading

The bridge was designed in accordance to the live loadings prescribed by Banverket, the Swedish rail authority [10]. The loadings set out are harmonised with the UIC Code 776-1: ‘Loads to be considered in railway bridge design’ [9], which is guidance produced by the International Rail Authority. In essence, the guide outlines the universal loading action of trains, of which the UK standards are very closely related. It is therefore acceptable to take the designated ‘Type RU’ from BS5400-2 as it is considered an equivalent loading regime to that laid out in the UIC Code [9]. Another additional loading regime is outlined by the UIC Code, labelled SW/2. This loading is similar to that of the ‘Type SW/0’ loading from the British Standard, but with a slightly higher UDL applied to it. We must, of course, remember that any loading parameters that we do apply must be applied twice as the Årsta Bridge consists of two tracks of train flow.

The RU loading is represented by a series of evenly spaced point loads, preceded and followed by a UDL. The SW/2 loading is the same as the SW/0 loading from BS5400-2, but utilises a slightly larger UDL of 150kN/m [9].

A dynamic factor, $D$, is applied to the imposed loadings, due to the dynamic effects that such live loading induces into the bridge. The derivation for the dynamic factor given in Eq. (1) was taken from the Swedish design standard [8], with the value of $L_0$ taken as 78m - the spans between piers. Given these parameters, the dynamic factor can therefore be calculated.

$$D = 1.0 + \frac{4}{(8 + L_0)}$$  \hspace{1cm} (1)

$$D = 1.047$$

Any distribution of the point loads outlined in cl. 8.2.4 [10] as part of the RU loading will be overlooked because unlike traditional tracks, the lines on the Årsta Bridge have been mounted straight into the concrete deck, as opposed to the more conventional setup of using sleepers and ballast. This action was taken to help try and minimise the dead weight of the bridge’s superstructure.

Application of the dead, superimposed dead and primary live loads provides us with the base load case. Further addition of wind and construction loads will give us load combination 2, and temperature effects will be considered for combination 3.

With regards to the pedestrian/cycle path that runs parallel to the train line, cl. 6.5.1.1 [10] outlines the basis behind calculating the live loading.

4.3 Traction and Braking Forces

In loading combination 3 [10], it is necessary to take into consideration the effects of traction and braking forces which are induced by the train, into the bridge’s structure. Both of these effects induce longitudinal stress into the bridge and, as cl. 8.2.10 [10] lays out, the loads are assumed to act at rail level, parallel to the direction of the tracks. In a two-tracked arrangement, it is assumed that the effects of such factors are simultaneously applied, with one braking force and one traction force, both acting in the same direction. From Table 18 [10] we can derive values of traction and braking to be equal to 750kN and 310kN respectively (therefore 1060kN combined). These judgements are made on the assumptions that the loaded length for traction is over 25m, and equal to 10m for breaking. Applied in combination 3, a partial load factor of $\gamma_f = 1.20$ would therefore bring the value of loading up to that suitable for ULS analysis. For SLS, values remain unchanged (see Table 1).

5 Strength

Using the loadings that have already been addressed, some worst case loading scenarios can be considered for bending, compression and torsion of the deck.

5.1 Bending

There are two critical bending conditions that must be considered; there is that of hogging, incident where the piers support the deck; and there is sagging, which is maximum at the mid-span between piers. Fig. 8 shows the variation in the cross-section of the deck between the pier support position and the mid-span.
The maximum hogging moment is produced in the bridge deck when two sets of RU loading are found on adjacent spans (Fig. 9) and given by Eqs. (1,2).

$$M_{\text{hog}}^{\text{dead}} = 2 \times \left( \frac{WL}{12} \right)$$  \hspace{1cm} (1)

$$M_{\text{hog}}^{\text{live}} = 2 \times \left[ \frac{Wa}{12L} (3L - 2a) \right] + \left( \frac{Pa_1b_1^2}{L^2} \right) + \left( \frac{Pa_2b_2^2}{L^2} \right)$$  \hspace{1cm} (2)

$$M_{\text{hog}}^{\text{total}} = 805.8 \text{ MNm}$$

$$\sigma_{\text{sag}} = \frac{My}{I} = \frac{805.8 \times 10^9 \times 2663}{75.85 \times 10^{12}}$$  \hspace{1cm} (3)

$$\sigma_{\text{sag}} = 27.2 \text{ N/mm}^2$$

The moments induced in sagging are maximised at the point where two sets of RU loading pass the mid-point of a span concurrently (Fig. 10). See Eqs. (4,5).

$$M_{\text{sag}}^{\text{dead}} = \frac{WL^2}{8}$$  \hspace{1cm} (4)

$$M_{\text{sag}}^{\text{live}} = 692.0 \text{ MNm}$$

$$M_{\text{sag}}^{\text{total}} = 140.1 \text{ MNm}$$

$$M_{\text{sag}}^{\text{live}} = 634.4 \text{ MNm}$$

$$\sigma_{\text{sag}} = \frac{My}{I} = \frac{(634.4 \times 10^9 \times 1713)}{16.19 \times 10^{12}}$$  \hspace{1cm} (6)

$$\sigma_{\text{sag}} = 67.1 \text{ N/mm}^2$$

Additional resistance to bending is offered by the post-tensioned cables that run through the deck’s cross-section. In a manner synonymous with a stress ribbon bridge, the cables carry a certain level of stress which resists the downwards deflection caused by the imposed loadings. These cables are present throughout the box-section deck, but are particularly prevalent along the curved bottom face – the most critical region during sagging action, which exhibits a higher stress than that induced due to hogging, Eqs. (3,6). This is the case – despite the higher moment induced in the hogging case – because the second moment of inertia is considerably lower for the smaller mid-span cross section.

5.2 Compression

When braking forces are applied, compression will be induced longitudinally in the deck. As the author has already outlined, cl. 8.2.10 [10] summarises the determining of such loads. It is particularly important at this point to remember that the section of the bridge deck varies periodically across its spans. It is therefore important to assess longitudinal compression at the point at which the deck section is smallest, with a minimum value for moment of inertia. When both the braking and traction forces are simultaneously applied, a total longitudinal force of 1060kN (750kN + 310kN) is induced. A strength class of K60 ($\sigma \approx 60\text{MPa}$) has been obtained throughout the bridge’s superstructure [3], and this is reflected in the calculation in Eq. (7). The minimum I-value has been used, which corresponds to the deck section at mid-span.

$$P_E = \frac{\pi^2 EI}{L^2}$$  \hspace{1cm} (7)

$$P_E = \frac{\pi^2 (60 \times 10^3) \times (16.19 \times 10^{12})}{(78 \times 10^3)^2}$$

$$P_E = 1576 \text{ MN} >> 1060 \text{kN}$$

For loading combination 3, we must also consider the influence of temperature effects on compression in the deck. This is calculated using Eq. (7) and we can see that the value of $P_E$ obtained is lower than that of the maximum applied forces derived from Table 18 [10], where a traction force of 750kN, and a braking force of 310kN combine to produce an overall longitudinal force of 1060kN.
5.3 Torsion

An induced torsional moment will be a common occurrence in the Årsta Bridge as it will be rare for two trains to cross the bridge concurrently, particularly as the direction of travel on the two lines runs in opposite directions. Fig. 11 graphically represents the two main components of torsion induced in the bridge deck.

The tracks are centred over the vertical separators in the box section, which are situated about 2.2m from the centre line of the bridge deck. Considering a single 250kN point load acting over 1.6m spacing (as per RU loading [10]), this induces a torsional moment of 343.8kNm/m into the bridge deck. This torsional moment will be carried through a combination of the abutments and the bridge’s piers. Due to the elliptical configuration of the piers, they will provide excellent resistance to torsion as their dimension in the transverse direction is provides excellent bracing against rotation. The shear flow induced into the bridge’s deck is represented by Eq. (8) below.

\[ v = \frac{T}{2A_{ef}} \]

\[ v = \frac{343.8}{2 \times 45} \]

\[ v = 3.82 \text{ N/mm}^2 \]

As the author addressed earlier, analysis of wind influence is difficult to gauge with in long-span bridges, and effects are often analysed through physical modelling in a wind tunnel, or using FE analysis. For bridges with a total span of less than 200m, analysis can be undertaken using cl. 5.3 [10], which allows a bridge’s resistance to maximum wind gust to be calculated.

6 Serviceability

During its construction, the Årsta Bridge has a number of pieces of instrumentation installed, with the intention of continuously monitoring the bridge’s behaviour. Data will be collected for the first ten years of service [7] with the dual function of providing not only reassurance that the bridge is performing satisfactorily, but also to yield valuable research information in the field of dynamic loading. It is hoped that information collected from this bridge could be used to scrutinise the existing guidance supplied by Swedish design standards.

The resistance of the bridge deck to torsion is enhanced thanks to the use of a closed cell concrete box girder as the deck. Such a rigid unit is a highly effective form against the torque induced by asymmetric loading due to trains and wind. It makes use of structural elements in vertical and horizontal planes, providing orthogonal resistance against the twisting action.

It is also pertinent to highlight the clear intention of the designers to position the tracks as centrally as possible on the deck section. This naturally recues the torque, as the level arm separating the centre of the section and the line of action is minimised.

Most of the monitors installed on the bridge were either fibre-optic cables or strain gauges, Fig. 13. These devices collect information during service, and transfer the data via a secure internet connection to be analysed [7]. This ensures that problems may be quickly identified, and also mitigates any need for awkward and arduous data collection from each device.

The primary reasoning behind the use of these monitoring techniques is to ensure that the static stresses and strains remain within permissible limits, but also that the dynamic behaviour of the bridge is in keeping with its predicted action, as well as checking that cracking in critical sections does not exceed the parameters set out in the design [6].

Concrete was considered in the design and was included in the FE material model. The timing of specific
sequences were also programmed to maximise the accuracy of the analysis [6].

7 Construction

Two different construction techniques were adopted in the building of the Årsta Bridge. This can be attributed to the curvature present in the path of the bridge at its northern end.

To the south, the line of the bridge is straight, and was therefore produced using casting girder, which provided the support for the timber formwork required to produce the box girder deck. To the north however, the line of the bridge was slightly curved, and therefore construction using a casting girder would have been especially difficult. Consequently, a conventional scaffold support system was established (Fig. 15), as this allowed the curved form of the bridge to be more easily created.

The casting girder was actually two 3m high box girders, 180m long and supported by roller supports mounted on the elliptical piers [3]. These were used to incrementally launch the bridge from the south side of the Årstaviken, with the plywood formwork dividing at its centre to allow it to be progressed along the length of the deck, in line with construction (Fig. 16). Temporary lateral supports were also installed during the launching sequence to protect the girders against wind loading.

The temporary loading conditions of the two truss girders was considered during FEM analysis, with factors such as creep, shrinkage and duration of loading taken into account [6]. Consideration was also given to the sequence in which the pre-stressing cables were to be tightened, as this obviously impacts on the strength of the deck.

The size of the trussed girders became substantial due to the difficulty of placing intermediate supports between the spans to reduce the hogging moment induced in the girders at pier supports. However, because of the presence of the Årstaviken, it was presumably deemed more economical to simply fabricate larger girders than try and install sizable temporary supports on an uneven bed surface, in the middle of a busy bay.

Figure 16: Divisible formwork mould being projected by large trussed girders [3]

During the actual construction of the bridge deck, the reinforcement cages were lifted into position using a gantry crane which was supported by the casting girders. Eight differently sized cages were manufactured to produce the undulating profile of the deck [3], which were then tied together on site using loose lacing bars. The red concrete was then pumped into position, sometimes with travel distances of up to 200m.

The concrete box-girders were produced using in-situ concrete pours. Given the length of the bridge, some consideration had to be given to the suitability of the concrete’s consistency in ensuring that it retained its strength and possessed a suitable open time, given the distance it had to be pumped. Refinements that were made to ensure that such requisite properties were met included washing of the ballast, the use of plasticiser and retarder in the mix, and also the dispersing of the red pigment in water 24 hours prior to addition [3].

Four of the bridge’s piers were located in the Årstaviken, and therefore a construction method had to be adopted whereby the pier and its foundations could be produced safely.

To the north of Årsta Islands, ground conditions were fairly uniform in level, and bedrock was located at a depth of 15-20m. Excavations were therefore undertaken to this depth, and concrete foundations cast underwater, sitting directly onto bedrock. However, to the south of the Island, the depth to rock varies significantly due to the presence of a fault line in the navigable channel [3]. This meant that to mitigate the need for excavations at very great depth, and an elevated pile system was used (see Section 8). To allow this to take place, a waterproof base was cast underwater, and a sheet pile cofferdam installed to create an independent pool. The water in this pool was then pumped out to allow reinforcement and casting of the elevated foundation slab and pier to take place.

8 Foundations & Geotechnics

As the author has already outlined, the depth of strong bedrock varies greatly across the length of the bridge due to the presence of a deep fault line that exists on the south side of the Årsta Island. This necessitated the need for alternative pier foundation techniques to be
derived, depending on the depth of the bedrock at that point.

In total, foundations for ten piers were required; six of which were located on land, and the other four in the Årstaviken. Two of these four piers in the Årstaviken were situated north of the Årsta Island, and the other two to the south. The intention was for the footings to be founded on solid bedrock, but due to the variable ground conditions, not all of these footings could be homogeneous in design.

The main problem that the engineers had to overcome was the deep fault line that was present in the bay to the southern side of Årsta Island. This meant that the distance to the bedrock was particularly large, rendering a typical pier footing (i.e. a foundation slab which sits directly onto bedrock) incongruous. The issue was therefore resolved by creating an elevated concrete pile cap for the piers using 800mm diameter concrete filled steel piles [3], which extended down into the bedrock (Fig. 17). This eliminated the need for difficult underwater construction techniques.

Where foundations were founded in the shallow waters of the Årstaviken bay, a more common method of pier footing was employed: sheet piling was driven down to the bedrock and any soft material above it removed and taken away by dredges. The sheet pile cofferdam was then established and pumped of any water, before the bedrock was broken away to produce the pockets into which the foundation slabs were to be cast (Fig. 17). The slab was tied into the existing bedrock using steel reinforcement to help resist any uplift, before the casting of the slab took place.

Eq. (9) below calculates the bearing strength of an individual pile. Although the piles are generally 25-35m in length, they have been elevated above the bed surface, and skin friction has therefore only been considered for 15m depth. Ground conditions are unknown and variable so an undrained shear strength of 200kPa.

\[
Q_{out} = q_{b}A_{b} + q_{s}A_{s}
\]

\[
Q_{utb} = (200 \times 10^{3} \times \pi (0.4)^{2})
+ (0.5 \times 200 \times 10^{3} \times 15 \times 2\pi (0.4))
\]

\[
Q_{utb} = 3870.4 kN
\]

The dead loading is taken as 650kN/m, acting over a 78m span. The loading, P, carried by an individual pier is given by Eq. (10) as 50.7MN. The cap consists of 30 piles [3], which gives a total capacity of 116.1MN, Eq. (11).

\[
P = 650 \times 78 = 50.7MN
\]

\[
Q_{total} = 3870.4 \times 30
\]

\[
Q_{total} = 116.1MN > 50.7MN
\]

9 Temperature

Temperature effects are important to consider in bridge design as these can affect the structure through two different modes. Firstly, the overall temperature change (effective temperature) should be considered as this induces longitudinal stress in the deck. And then the variation in temperature between the top and bottom surfaces of the bridge structure should also be considered, as this induces stress across the section of the deck. The Årsta Bridge acts in a fashion synonymous with a continuous beam, and the columns are considerably weaker in the direction of longitudinal movement because of their elliptical form. Consequently, the stresses caused by changes in temperature will mostly be carried by the deck section.

For the sake of the calculations, the bridge is required to accommodate a 1:120 year temperature extreme, and support conditions are considered to be fixed. The coefficient of expansion of steel and concrete is taken as 12x10-6/°C. For the sake of the calculation, the author has assumed a temperature fluctuation of 50°C (+30°C and -20°C) for Stockholm.

\[
e = (12 \times 10^{-6}) \times \Delta T
\]

\[
e = (12 \times 10^{-6}) \times 50°C
\]

\[
e = 600 \mu ε
\]

\[
\delta = (600 \times 10^{-6}) \times l
\]

\[
\delta = (600 \times 10^{-6}) \times 833 \times 10
\]

\[
\delta = 500 mm
\]

\[
\sigma = 60 \times 10^{3} \times 600 \times 10^{-6}
\]

\[
\sigma = 36 N / mm^2
\]

This simplified calculation suggests that even in the event of the bearings malfunctioning, and therefore no longer allow movement, the compressive stress induced longitudinally in the bridge deck is still found to be within the limits for the K60 grade of reinforced concrete, Eq. (14). The bearings themselves are pot bearings [6], which consist of a rubber pad which is placed into steel pot, with the rubber layer losing its stiffness under high pressure, precipitating horizontal movement.
Fig. 18 shows profiles of differential temperature for a concrete box girder. The calculation below outlines the moment induced in the bridge's deck as a consequence of a 25°C temperature difference across the depth of the section. Eq. (19) yields the moment induced by the temperature differential, which is evidently much smaller than the forces induced by dead and primary live loadings.

\[ \varepsilon = (12 \times 10^{-6}) \times 25 \]
\[ \varepsilon = 300 \mu \varepsilon \]
\[ \sigma = E \varepsilon \]
\[ \sigma = 60 \times 10^3 \times 300 \times 10^{-6} \]
\[ \sigma = 18N/mm^2 \]
\[ \sigma_{average} = \frac{2}{3} \times 18 = 12N/mm^2 \]
\[ N = \sigma A = 12 \times 70 \]
\[ N = 840kN \]
\[ M_{diff} = \frac{\sigma A I}{y_b} \]
\[ M_{diff} = 8326kNm \]

10 Wind

Wind loading is often a very important factor in bridge design as it can apply very high lateral pressures to a superstructure. In Britain, wind loading on bridge structures is analysed using cl. 5.3 [10], using data derived from 120-year peaks, at a height of up to 10m above ground level and 300m above sea level. Its analyses methods do not, however, extend to bridges that have a total span in excess of 200m. In such a situation, the guidance notes recommend reference to specialist advice. Generally, in projects involving long-span bridges, such as that at the Årsta Bridge, modelling of either the physical or electronic form are employed. A physical model – equipped with measuring devices – allows the user to acquire actual data on the bridge’s reactive behaviour. This method is expensive however, as the model and facilities come at considerable cost.

Computer analysis of the problem is also frequently adopted, with local wind data being fed into an existing model of the bridge to produce structural reactions. Depending on the detail of the model, it is also possible for other effect, such as the natural frequency of the structure, to be taken into account.

Assessing the Årsta Bridge in a purely qualitative fashion, there are some important refinements that appear to minimise the effect of wind forces on the superstructure. Firstly, the curved soffit of the deck produces less obtrusive resistance against air flow. It is therefore inevitable that the amount of vibration induced in the deck through vortex-induced vibration is sure to be reduced, as the air’s path around the deck section is more controlled and smooth, reducing the likelihood of vortices forming. It is also likely to reduce the effect of wind buffeting from turbulent wind, as the reduced area of incidence will decrease the fluctuating force that the wind can induce.

It would also be fair to say that the elliptical nature of the piers is also likely to have reduce the impact of wind on the bridge’s superstructure as, despite having an exposed surface area similar to the that of a standard square-section pier, the dimension of the pier in the transverse direction is considerably more slender, and also exhibits smooth edges, reducing vortex-induced effects. However, without detailed design tools, discussions such as this are hard to draw conclusive remarks from.

In reality, the new bridge is also likely to be sheltered somewhat by the presence of the existing East Årsta Bridge (Fig. 19). Wind coming from an easterly direction is likely to be broken by the older bridge, reducing its strength and intensity by the time it becomes incident. It is, however, unreasonable to take such an effect into consideration in design, as the projected lifetime of the East Årsta Bridge is likely to be much shorter than the new bridge.

11 Natural Frequency

The value of natural frequency for a bridge’s track slab is given by Eq. (20) below. The value obtained from this relationship helps inform us as to how the slab will behave in dynamic situations. A high natural frequency will see the bridge move due to dynamic actions induced by train movement, but not in a critical manner. At low frequencies, vibrations have the potential to cause the bridge to fail, providing that their amplitude is high enough.

The square of the eigenvalue, \( \lambda_1 \), is taken as 17 [7] which is derived from data collected from the operational bridge and processed through complex iterative analysis.

\[ f_o = \frac{\lambda_1^2}{2\pi n^2} \sqrt{\frac{EI}{m}} \]
\[ f_o = \frac{17}{2\pi (4.5)^2} \sqrt{\frac{(60 \times 10^9)(0.0028)}{2400 \times 0.322}} \]
\[ f_o = 59.6Hz \]
\[ 75Hz > f_o > 5Hz \]

The natural frequency is found to lie between the recommended frequency parameters to ensure that psychological effects are not induced (>75Hz) or that the member will catch gusting wind (<5Hz). However,
without further information regarding the bridge as a whole, it is difficult to determine an all encompassing natural frequency. For the design of the bridge, the structural engineering partners, COWI, determined the natural frequency of the bridge using their detailed FE model, and made use of this to make dynamic checks on the bridge.

12 Durability and Vandalism

It was chosen that the bridge would be coloured red by means of a pigment placed in the concrete mix in preference to simply painting the bridge. The fact that the colour is intrinsic means that maintenance demands on the bridge are lower, as the colour is naturally present and therefore does not require periodic replenishment. It was also found through extensive iteration of the concrete mix that the pigment approach offered better mechanical strength and a longer lifetime than the more conventional painted approach [3]. Although predominantly a train line, access is also offered to the public and emergency vehicles meaning that some salt based products are likely to be used to protect against frost. The use of such products may produce some eventual staining to the surface of the concrete.

![Figure 20: Vandalism to bridge piers [13]](image)

Sweden experiences from widespread graffiti abuse, and the bridge has therefore suffered from minor defacing. Such vandalism can be removed using readily available commercial equipment, but benefits from the fact that its colour is built-in, meaning that no repainting needs to occur once the graffiti has been removed.

13 Future Changes

Due to the highly bespoke nature of the bridge, and its integral function in the city’s transport network, it seems inconceivable that the bridge will ever be altered, or its function adapted. Its construction marks the incipient steps of a major extension to the rail network in the south of Stockholm and it is therefore highly unlikely to ever be used for anything else.

The deck has been designed in a very inflexible manner meaning that, due to the embedded track through which the rail line will pass, it is unlikely to ever carry anything other than trains. It is more likely that the East Årsta Bridge will experience some alteration, as it is considerable older and exhibits a more flexible layout.

14 Conclusion

The main premise behind the Årsta Bridge is essentially a basic bridge product – the concrete box girder. However, having refined both the structural performance and the aesthetics through measured design practices, an incredibly unique and attractive solution has been produced by the design team.

Whilst many other bridges have been built with the sole intention ensuring that they are labelled as iconic structures, conversely, the Årsta Bridge renders itself passive amidst its lush, unblemished environment. An ostentatiously presented bridge would have represented not only a contradiction of the peaceful surroundings of the Årstaviken, but also of Stockholm in general.

References