A CRITICAL ANALYSIS OF THE LEONARDO DA VINCI BRIDGE IN ÅS, NORWAY

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Abstract: This article provides a critical analysis of the Leonardo da Vinci Bridge, constructed in 2001 in the area of Ås, near Oslo. It considers aesthetics, loading, strength, serviceability, construction, temperature, creep, wind, durability, susceptibility to intentional damage and possible future changes. All these aspects are considered in terms of how the design or construction could have been improved.

Keywords: Leonardo da Vinci, pedestrian, arch, glulam, stress-laminated

1 Introduction

The Leonardo da Vinci Bridge is situated in the area of Ås in Norway, about 20km outside Oslo. The bridge was constructed in 2001 and allows pedestrians and cyclists to cross the E18 main road, which links Oslo to Stockholm. The main span of the bridge is 40m with a maximum clearance above the road of 5.8m. The total length of the bridge is 109m.

The bridge is fairly unique both in terms of its history and its construction. The main structure is made up of three exposed glue-laminated timber arches, which support a 2.8m wide stress laminated timber deck.

Figure 1: The Leonardo da Vinci Bridge at night

From Ref. [1] the design team was reasonably large and diverse. It included Reinertsen Engineering (structural engineers), Vebjørn Sand (artist), Selberg Architects, Moelven Group (timber experts) and the Norwegian Public Roads Administration (contractor and owner.)

The finished bridge is very striking and has attracted a large amount of international interest both in the design and the issues it raises.

2 History of the Bridge

This bridge has a long and strange history, and eventually came to exist in a fairly unusual way. According to Ref [2] Leonardo da Vinci produced the original bridge concept in 1502. The design was for a masonry bridge spanning about 240m over the Golden Horn in Istanbul. Although the bridge was never built, academics such as D. F. Stüssi have shown that the rough plans were technically viable, Ref [2]. It was almost 500 years later however, that a Norwegian artist, Vebjørn Sand, saw an exhibition of Leonardo’s work and decided to try and get the bridge built. He then approached the Norwegian Public Roads Administration, who agreed to take on the project. A design team was assembled and the concept design for the modern interpretation of the bridge decided upon. It was decided that the bridge should be timber, due to the tradition of timber bridges in Norway. In a fairly unusual order of events, the team then set about
finding a suitable location for the bridge, eventually settling on the final site in Ås, Ref [1].

The bridge is fairly strange in the fact that the concept, material type, and rough span were already known before the location of the bridge was decided upon. This could have been disastrous, as a bridge should respond to its environment both aesthetically and functionally.

3 Aesthetics

The bridge aesthetics have attracted a huge amount of both positive and negative comments. With some critics claiming it is a beautiful sculpture as much as a bridge, while others claim that the bridge belongs in Disneyland, Ref [1]. Here we will try to objectively assess the bridge aesthetics by principally considering Fritz Leonhardt’s ten areas of bridge aesthetics.

One of the factors that make this bridge very beautiful is its fulfilment of function. The structure is very pure and elegant and it is clear what each element does. The arch itself is a very pure form, which the bridge uses very well without complicating it. The timber interpretation of Leonardo’s original concept is especially elegant as only the most necessary elements are used. The inclined arches serve a clear function and also give the bridge a sense of stability. This is important to ensure that people feel comfortable walking over the bridge.

The proportions of the bridge are also very aesthetically pleasing. This is mainly due to the difference in size between the arches and the deck, which makes the deck appear very slender. The arch dimensions also look correct, with a good span to height ratio and within the arch itself, which becomes much wider towards the supports. The deck appears to be very slender and the detailing of the parapet helps this effect. The parapet is very light and as can be seen from Fig. 3 disappears when the bridge is viewed from a distance.

The artist was very keen to keep the same proportions as were described in Leonardo’s original sketches and so the deck sags gracefully over the main arches. This was achieved by using a stress-laminated timber deck, where the individual timbers could be cut so that the deck took the required shape. This was also chosen, as the designers were keen to use timber as much as possible in the bridge construction.

In terms of order, the bridge works very well, especially from a distance, with smooth lines that the eye can easily follow. Ideally the decks on arch bridges should go smoothly down to the ground, to create a smooth flowing line. This is often not possible especially in urban environments where space is expensive. Fortunately there was lots of space here to allow the ramps to be completely straight, rather than having to use helical ramps for example. This enhances the flowing line and lets your eye run smoothly along the entire length of the bridge. To achieve this aesthetic however the deck ends up being very long. One of the rules given by the highways agency is that pedestrians will not use bridges that are longer than twice the distance to cross at road level. This could be a potential problem here as the deck is longer than twice the crossing distance. However pedestrians could be forced to use the bridge by landscaping the area around it. This could also be improved by reducing the deck length as much as possible. However this could affect the aesthetics of the bridge.

As you get closer to the bridge there are some small issues in terms of the order of the bridge. Firstly the arches do not lie within the same plane as you drive towards it. This breaks up the flow of the arches and your eye may get caught at this point. Also on closer inspection of the bridge, it can be seen that the soffit of the deck is broken up by the parapet connections, as can be seen in Fig. 7. This breaks up the flowing line of the edge of the bridge deck and may be improved by putting these connections within the deck and creating a smooth facia.

Another issue to consider is the refinement of design and generally this bridge is quite carefully refined. The glulam arches are carefully shaped, producing very smooth lines and changing from wide at the base to relatively thin at the centre. However there are some parts, which seem to be overlooked and could possibly have been further refined. Firstly the abutments are above ground and the concrete is very noticeable.
Abutments should be hidden from view as much as possible and are often completely hidden below the ground. However it is obvious that this was very difficult to achieve in the design of this bridge, as the durability of the timber was the critical issue. Therefore the most important thing was to keep the bases of the timber arches away from the ground. There are still improvements that could have been made here. Landscaping could be introduced around the abutments so that they are not so visible, however it would need to be carefully designed and drained so that moisture was not transferred to the timber. Also the base detail could be further refined by using a steel ‘shoe’ at the base of the arch. This could be used to protect the base of the arch and to lift it up from the ground so that the abutments did not need to be so obvious. The angle and size of the abutments show the size of the thrust at the base of the arch and this seems to reduce the elegance of the structure. As these forces at the base of the arch are quite large however this may mean that a steel ‘shoe’ may become too cumbersome and could further detract from the bridge aesthetics.

Also in terms of refinement, the steel columns that support the deck from the glulam arches seem to be fairly unrefined and have the feeling that they were almost after thoughts. However these members are minimised as much as possible; they are very slender and are painted a light grey. As can be seen in Fig. 3, from a distance this detailing means that the supports are very difficult to see and the deck almost seems to float.

As previously mentioned this bridge is fairly unusual in the fact that many design decisions had been made before the final location had been decided. This could have been a significant issue, as one of the main rules for bridge design is that a bridge should be appropriate and integrated into its environment. In actual fact the bridge fits well into its environment (as it would have done in many parts of Norway!) According to Ref [3] the choice of material was made very early in the bridge development. It was decided that the bridge should be timber due to the tradition of timber bridges in Norway. The fact that the bridge is timber means that the bridge immediately fits into its environment and the choice of material has played a huge part in its aesthetic quality.

For pedestrian bridges, texture, colour and the quality of details are extremely important as people walk across the bridge slowly and so have time to interact with the structure. Therefore these issues would have been carefully detailed. However the bridge does give the impression that more focus was given to the aesthetic quality of the bridge from a distance compared to the quality of the bridge for pedestrian users. This may be slightly unfair and is not entirely a bad thing. The Highways Agency states that motorway bridges should be unusual and visually stimulating to keep motorists awake, and this bridge is likely to have this effect.

As mentioned surface texture is extremely important, especially on pedestrian bridges, however it is often ignored. The use of timber defines this bridge and gives it a very pleasing surface texture and natural colour. Unfortunately when pedestrians are on the bridge, they do not get to experience the timber structure ‘up-close.’ Therefore the bridge may be improved from a pedestrian’s point of view by introducing timber elements above the bridge deck. This may have durability issues however.

Colour is often used in bridges to highlight or hide elements of the structure to improve the aesthetics. This is most obvious on this bridge in the steel deck supports, which are painted grey. This differentiates them from the rest of the structure however it also means that they disappear as you view the bridge from a distance. For the rest of the bridge, the timber is kept very natural. This was partly for environmental reasons, however the main reason was to create the striking aesthetic, as the bridge looks very natural and is obviously timber. There are however many associated problems in terms of durability, which will be discussed later.

As the bridge is mainly all one colour, light and dark is used to define different parts of the bridge. The edge of the timber deck is very bright and so naturally stands out, especially from the supports. Therefore your eye is drawn naturally to it and the bridge appears very slender.

The aesthetics of this bridge have caused much debate, however it cannot be argued that this bridge is not very striking and has huge amounts of character. Also the fact that the bridge is built in timber makes people more curious about how it was built. Character is important in making a bridge that is memorable and it is also very effective for motorway bridges. As mentioned it is suggested that motorway bridges should be unusual and dramatic to keep motorists awake and this is certainly something that the bridge achieves.

The deck of this bridge is very cambered, which adds to the character of the bridge. It is possible to do this much more with pedestrian bridges than road and rail bridges. Today however pedestrian bridges are hardly ever built where it is not possible to see over the ‘hump’ of the bridge. This is due to the fact that pedestrians do not know if the bridge exists past the ‘hump’ and also that it is not possible to see what is coming towards you on the other side of the bridge. This could be an issue with this bridge. Often landmarks such as masts are used to mark the other side of the bridge, however this was obviously not possible with the arched structure of the bridge.
4 Loading

The main vertical arch supports the deck at three points, which are reasonably close to the centre. However conservatively these point loads will be assumed to act at the centre and the quarter points of the arch. It is also assumed that the inclined arches do not carry any of the vertical loading; however it is assumed that they do support their own weight. These inclined arches carry the lateral loading on the bridge.

The deck build up is a stress-laminated timber deck, which supports a watertight membrane covered by an asphalt wearing course.

4.1 Dead Load

Assuming a glulam density of 440kg/m³ and that the triangular cross-section of the arch is 1m wide and 1m deep. The cross sectional area of the arch is then 0.5m² and the self-weight of the arch is 2.20kN/m.

For the stress-laminated timber deck assume a density of 500kg/m³ (to take into account the weight of steel pre-stressing bars). Taking the deck section as 2.8m wide and 0.3m deep, the cross-sectional area as 0.84m² the self-weight is 4.2kN/m.

4.2 Super-imposed dead load

The assumed superimposed dead load is based on a 50mm depth of asphalt, with a density of 2300kg/m³. Therefore the superimposed dead load for the 2.8m wide deck is 3.2kN/m. This value is increased to 4kN/m to take into account the weight of the stainless steel parapets and also services such as drainage and lighting.

These loadings are classified as super-imposed dead load as they are loads that may change over time. Due to the uncertainty of this loading, the safety factor for superimposed dead load is usually high. This is because the bridge may be resurfaced and services such as drainage, barriers or lighting may be added or updated at a later date.

4.3 Live Load

4.3.1 Deck loading

For a footbridge less than 30m long, a nominal live loading of 5kN/m² is applied. However for bridges, which are longer than this, such as the Leonardo da Vinci Bridge, the probability of this high loading being applied along the entire bridge length reduces as the length increases. This can be taken into account by a correction factor,

\[ k = \frac{\text{nominal HA udl for bridge length}}{30 \text{kN/m}} \]  

The total deck length is 109m and so the nominal HA udl is calculated in Eq. (2):

\[ w = 15(1/109)^{0.475} \]

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\[ w = 16.3 \text{kN/m} \]

Subbing this back into Eq. (1):

\[ k = 16.3/30 = 0.543 \]

Therefore the unfactored live load acting on the bridge deck is:

\[ w_{live} = 0.543 \times 5 \text{kN/m}^2 = 2.72 \text{kN/m}^2. \]

In addition to this the designers may also have considered the possibility that a vehicle could be driven on to the bridge. In this case an additional loading of four wheels, each of 25 units HB loading are applied to the deck. One wheel represents 2.5kN per unit of HB loading, and so each wheel is a point load of 62.5kN. However this loading can be ignored if adequate barriers are installed at each end of the deck. As the deck is a timber construction, it is likely that the designers would want to minimise the design loading and so barriers of this sort have most probably been installed.

4.3.2 Parapet loading

The parapets should be capable of supporting a nominal horizontal loading of 1.4 kN per meter run. Although the possibility of a vehicle being driven on to the bridge is considered, we do not consider any loading of this sort on the parapet.

4.3.3 Direct loading of primary structure

The number of secondary live loading effects on the structure are significantly reduced by the fact that the bridge only carries pedestrians. However impact loading on the supporting arches must be considered as traffic passes underneath the bridge. The primary structure is less than 4.5m from the road and therefore it must resist a load of 150kN at 0.75m and 100kN between 1-3m height from the base of the arches. Collision of tall trucks with points higher up the arch may also need to be considered, as the clearance underneath the arch reduces as you get closer to the supports. This loading could have been reduced if the arch supports had been placed more than 4.5m from the road, however this would have made the arch and foundations significantly larger and therefore made the bridge more expensive.
4.4 Wind

Wind is unlikely to be a major issue in this bridge as it is a reasonably short span and is relatively close to the ground. However wind loading is important to consider, as this is the main lateral load that will be supported by the inclined arches. In addition to the horizontal wind load on the deck and the supporting arches, wind will also cause uplift and downward loading on the deck.

Firstly considering the horizontal loading, the maximum wind gust, \( v_c \) must be calculated. The mean hourly wind speed, \( v \) is assumed to be 30m/s and considering the bridge is approximately 6m above ground and 109m long, the wind coefficient, \( K_1 = 1.37 \) and the gust factor, \( S_2 = 0.91 \). There is no funnelling effect and so the funnelling factor \( S_1 \) is taken as 1.00. The maximum wind gust can then be calculated:

\[
v_c = v K_1 S_1 S_2 .
\]

However, for footbridges, the value of \( v_c \) can be further reduced to reflect wind break action and so for a bridge 6m above the ground the reduction factor is 0.76. The new value of \( v_c \) is therefore 28.4m/s.

The deck is approximately 2.8m wide and 0.3m deep, but according to BS5400 the minimum effective depth is taken as 1.25m when the bridge is live loaded. Therefore the ratio of \( b/d = 2.2 \), which gives a drag coefficient, \( C_D = 1.5 \). The minimum allowable drag coefficient for pedestrian and cycle track bridges is 2.0 and so the horizontal wind load, \( P_t \) on the deck is therefore:

\[
P_t = q A_t C_D .
\]

This is higher than might be expected, as the deck is relatively thin and the parapets are very open, however this takes into account situations such as crowds on the bridge or banners covering the parapets, even though this is very unlikely in extremely high winds. It must be ensured that the connections between the supporting arches and the deck are able to carry this lateral load.

The wind loading on the supporting arches must also be considered and the horizontal force can be calculated in a similar way as before. The arch members are triangular and inclined in two directions and therefore are definitely not typical sections. However to gain a very rough estimate we can approximate each half of the arch to square members orientated diagonally to the wind, with a projected horizontal area of 40m² and a height/breadth ratio of 10. The drag coefficient, \( C_D \) is then 1.2 and so from Eq. (5):

\[
P_t = q A_t C_D .
\]

\[
P_t = \left(0.613 \times 28.4^2 \right) \times (40 \times 1.2)
\]

\[
P_t = 24kN \text{ applied to each half of the arch}
\]

The bridge geometry is obviously not as simple as has been calculated above, most fundamentally in the fact that the deck and supporting structure are arched rather than simple horizontal and vertical members. Therefore wind tunnel testing should be undertaken in order to determine exactly what loads should be applied to the structure.

Wind loading will also cause uplift and vertical downwards force on the bridge deck. Again the ratio of \( b/d = 2.2 \) and therefore the lift coefficient, \( C_L = 0.4 \). The nominal uplift or downwards vertical force is therefore:

\[
P_v = q A_L C_L .
\]

\[
P_v = (0.613 \times 28.4^2) \times 2.8 \times 0.4
\]

\[
P_v = 0.6kNm^{-1}
\]

An additional load that is not considered here is snow loading. In places such as Norway this can be quite substantial and may actually control the design.

4.5 Load combinations

There are five main load combinations that must be considered in bridge design and all possible load combinations must be considered for both ultimate limit state and serviceability limit state. It is likely that for this bridge the worst loading combination will be; all permanent dead and superimposed dead loads plus primary live loads, plus wind loading. Therefore this is the load case that will be considered in this study. The loads must then be applied to the structure to create the most adverse conditions possible.

![Figure 5: Steel connections between glulam arches and stress-laminated deck.](image-url)
5 Strength

The Eurocode EN 1995-2: 2004 offers some guidance on the design of timber bridges. However for simplicity this study will consider BS 5400 for strength calculations. These calculations can give an initial idea of the forces within the structure and the $\gamma_f$ factor will be taken as 1.10.

5.1 Stress laminated timber deck

The stress laminated timber deck is made up of sawn timber sections placed on edge, which run longitudinally along the deck and are clamped together with high-tension steel bars.

For ultimate limit state the factored loads applied to the deck are:

$$\gamma_f \gamma_{fl} w_{dl} = 1.05 \times 1.10 \times 4.2 = 4.9 \text{kNm}^{-1}$$

$$\gamma_f \gamma_{fl} w_{sdl} = 1.75 \times 1.10 \times 4 = 7.7 \text{kNm}^{-1}$$

$$\gamma_f \gamma_{fl} w_{live} = 1.25 \times 1.10 \times (2.72 \times 2.8) = 10.5 \text{kNm}^{-1}$$

$$\gamma_f \gamma_{fl} w_{wind} = 1.10 \times 0.6 = 0.73 \text{kNm}^{-1}$$

The total factored UDL is therefore:

$$W = 4.9 + 7.7 + 10.5 + 0.73 = 23.8 \text{kNm}^{-1}. \quad (8)$$

The stress laminated timber deck acts as a continuous ribbon over the supports. For a continuous UDL along the length of the deck the maximum sagging moment is:

$$M = WL^2/24. \quad (9)$$

However as the deck section is rectangular and symmetrical, we are only concerned with the maximum moment, which will be due to the maximum hogging, which is calculated from Eq. (10):

$$M = WL^2/12. \quad (10)$$

$$M = (23.8 \times 13^2)/12 = 335 \text{kNm}$$

To fully check the maximum moments in the deck however, loading should be applied to create the most adverse conditions. Therefore to find the maximum sagging moment, every other span is loaded with fully factored dead, fully factored superimposed dead and fully factored live load and between this the deck is only loaded with the unfactored dead load. Wind loading could also be added, however its impact would have to be carefully assessed to ensure that the most adverse loading conditions were found.

Similarly for the maximum hogging moment we still want to use the minimum relieving action possible. However the loading is changed so that maximum loading is applied to the central two spans and then every other span from this point.

5.2 Glulam arches

It is assumed that the central glulam arch supports all vertical loading and is loaded at the centre and quarter points by point loads from the deck. We can assume that the worst case loading is due to factored dead, factored superimposed dead and factored live on one half of the arch and unfactored dead on the other half. Conservatively the weight of the arch is also included in the UDL and applied as point loads. Therefore the total unfactored dead load, $w_{dl} = 2.2 + 4.2 = 6.4 \text{kN/m}$ and the factored dead load, $\gamma_f \gamma_{fl} w_{dl} = 7.4 \text{kN/m}$. The assumed loading arrangement is shown in Figure 6:

![Figure 6: Worst case loading on the arch for maximum bending moment at the quarter point.](image)

Assuming that the arch is parabolic, the arch shape formula is simply, $y = kx^2$. At the midpoint $y = 20$ and $x = 5.8$ and so $k = 0.595$. Therefore the quarter point co-ordinates are $x = 4.1 \text{m}$ and $y = 10 \text{m}$. For analysis the arch is assumed to be three-pinned.

To find the vertical reactions, take moments about $A$:

$$384 \times 10 + (160 \times 20) + (96 \times 30) = 40V_B. \quad (11)$$

$$V_B = 248 \text{kN}$$

and resolving vertically:

$$384 + 160 + 96 = V_A + V_B. \quad (12)$$

$$V_A = 392 \text{kN}$$

To find the horizontal reactions, consider the LHS and take moments about the top pin:

$$384 \times 10 + (H_A \times 5.8) = (392 \times 20). \quad (13)$$

$$H_A = H_B = 690 \text{kN}$$

Assuming that the arch meets the ground at an angle of $30^\circ$ the resultant force, $F$ in the arch is:

$$F = \sqrt{690^2 + 392^2}. \quad (14)$$

$$F = 794 \text{kN}$$
It is assumed that the maximum moment occurs at the quarter point of the arch on the LHS and so taking moments about this point:

\[ M = (V_A \times 10) - (H_A \times 4.1) \]

\[ M = 1091 \text{kNm} \] (15)

This assumes that the arch is loaded at the centre and quarter points, however on the real arch the loads are all much more central, which will reduce the moment in the arch. However the support reactions may be higher than this in the case where fully factored dead, superimposed dead, live and wind load are applied vertically downwards along the entire length of the deck.

6 Servicability

In addition to ultimate limit state, serviceability limit state must also be considered. This is to ensure that the bridge is serviceable. The entire bridge is unclad and there are likely to be very few services required to run along the deck. This may mean that a larger amount of movement is allowable in the bridge. The most important issue is likely to be that the deflections in the stress laminated timber deck are not too large. This is to ensure that the wearing surface and services in the deck are not damage during normal loading.

7 Construction

7.1 Prefabrication

A major element of the bridge construction was the prefabrication of the timber elements. Due to the fact that the bridge is almost entirely timber, prefabrication was the only real choice, however this was probably very beneficial as it allowed quick assembly and therefore reduced the amount of time required on site. The central arch is made up of four prefabricated sections, while the two inclined arches are each made up of three sections. Each arch section is rigidly connected to the next by steel plates and dowels, to create a continuous glulam arch.

This project has also highlighted the huge amount of possibilities in forming large glulam sections. Each section was roughly made up of a number of laminations and then ground down by a computer aided grinding machine to give the required finished surface, Ref [3].

The stress-laminated deck also involved a large amount of prefabrication before onsite construction commenced. In line with the original concept the designers wanted the deck to ‘sag’ over the supporting arches. This lent itself well to stress-laminated timber construction and the team were also keen to use timber as much as possible in the bridge. According to Ref [3] each timber laminate was pre-cut to the correct angle so that as the deck was fixed together it would take on the required shape. Holes were also predrilled so that the high-tension steel bars could be threaded and tightened to create the solid prestressed deck.

7.2 Onsite construction

It is very expensive to close off roads and so usually a design is chosen that minimises the time that a road must be shut. For example, in the UK it costs roughly £2 million per hour that a motorway is closed. According to Ref. [6] the bridge was constructed without disrupting traffic. This may mean that only half the road was closed at a time so that the traffic could continue while the main arches were erected. However it likely, due to the danger of connecting large sections together over traffic, that the traffic had to be stopped for short periods of time during the construction. As mentioned the high level of prefabrication meant that there were very few parts to assemble on site, which allowed for reasonably quick construction. The small number of parts also meant that small cranes could be used to move and place members.

Figure 8: The bridge during construction

The bridge could have constructed using cantilever construction, which may have reduced the number of cranes that were required. However a crane would still have been required to move the central pieces into place and so the road would still have had to be closed at certain times. Also this would have greatly increased the required size of the arch sections, especially near the supports.

8 Foundations and Geotechnics

Foundations are often a major proportion of the cost of bridges, however it is likely in this bridge that the foundations were reasonably simple. The designers had
the benefit of choosing a site for the bridge and so it is likely that they choose a site with reasonably good soil conditions. This is because, as shown previously, arch structures create very large horizontal reactions at the supports. The foundations must be able to take this horizontal thrust, otherwise sliding will occur. It is important that this does not happen as the main cause of failure in arches is due to spreading of the supports. Also for this reason the foundations will have been designed against sliding.

The foundations of this bridge are most likely strip footings, which sit below the exposed abutments (shown in Figure 4), however as discussed this does depend on the soil quality. These footings must be able to support the horizontal and vertical reactions at the base of the arch. Additionally the three glulam arches all connect into the same abutment and so it is likely that the abutment is designed to tie the two inclined arches together. This would ensure that the entire structure is stabilised and out of plane slip failures are taken out of the equation.

9 Natural frequency and vibrations

According to Ref. [3] there were worries that the structure may perform badly under traffic or rail loading, due to the affect of vibrations on the glulam arches. This was one of the main reasons for the decision to only design the bridge for pedestrians and cyclists. This demonstrates the importance of considering natural frequency and vibrations in bridge design.

For a simple initial estimate of the fundamental frequency of the deck, we can consider Eq. (16) below:

$$\omega_n = (\beta_l)^2 \sqrt{EI/\text{ml}^2}.$$  \hspace{1cm} (16)

The deck is continuous over the vertical supports and so for a single span we can initially assume a 'clamped-pinned' beam. Therefore for the fundamental frequency, \((\beta_l)^2 = 15.42\).

The assumed deck section properties are:

- Density = 440 kg/m³ (15% moisture content)
- Area = 0.84m²
- Mass = 370 kg/m
- \(I = bd^3/12 = 6.3 \times 10^{-3} \text{ m}^4\)
- \(E = 13500 \text{ N/mm}^2\) (Assuming GL32)

Therefore the fundamental frequency from Eq. (16):

$$\omega_n = 15.42 \sqrt{13.5 \times 10^9 \times (6.3 \times 10^{-3}) / (370 \times 13^4)}$$

$$\omega_n = 43.8 \text{ Hz}$$

The deck is actually continuous, however and so each span is more likely to act as a 'pinned-pinned' beam. This means that our initial estimate is likely to be inaccurate and the fundamental frequency may be significantly lower. The deck frequency must be in the range between 5Hz and 75Hz and the calculated fundamental frequency is well within this range.

It is important that the fundamental frequency is not greater than 75Hz especially on pedestrian bridges, as the vibrations cause a physiological effect that makes people feel very uncomfortable. Similarly it is important that the natural frequency is not significantly lower than 5Hz, as it begins to approach the frequency of walking. If this happens, as people walk across the bridge you get a lock in effect, where people begin to walk in time with the natural frequency of the bridge. This can lead to large movements and possible failure.

The stress laminated timber deck is supported over the supports as a continuous ribbon with reasonably long spans and there may have been some worries about the vibrations in the deck. However this initial check suggests that the fundamental frequency is well within the required range. Due to the fact that the deck is stress-laminated timber it is likely that testing would be required to ensure there are no vibration issues.

10 Temperature

From Ref. [4] thermal effects in timber members are usually insignificant in most bridge applications. However when there are timber members of a significant length, thermal effects may be important. This is certainly the case in the Leonardo da Vinci Bridge, where the stress laminated timber deck is tied together to create a continuous 109m long 'ribbon' over the supports.

Firstly the expansion and contraction of the entire deck length will be considered, assuming a maximum temperature range of 40°C in the Oslo region.

The coefficient of thermal expansion for timber is 40.0 × 10⁻⁶ /°C and therefore the total change in length of the deck is:

$$\Delta L = \Delta T \times L \times \alpha.$$  \hspace{1cm} (17)

$$\Delta L = 40 \times 109000 \times 40 \times 10^{-6}$$

$$\Delta L = 174 \text{ mm}$$

The value calculated in Eq. (17) is reasonably large, however this expansion could be easily taken by expansion joints at either end of the bridge deck. This seems like the most sensible place to put expansion joints as the span is relatively small and it is undesirable to have to put expansion joints within the span. It may be possible to construct the bridge without expansion joints, however even if expansion joints are used, it is possible that these could become clogged and cease to function. In this case compressive stresses would build up in the deck.

Although the deck would not actually extend, the effective strain that would be felt in the deck is calculated in Eq. (18):

$$\epsilon = \Delta L / L.$$  \hspace{1cm} (18)

$$\epsilon = 174 / 109000$$

$$\epsilon = 1.60 \times 10^{-3}$$

The compressive stress in the deck is then found from Eq. (19) below:
\[ \sigma_c = \varepsilon \times E \, . \]  
\[ \sigma_c = 1.6 \times 10^{-3} \times 13500 \]
\[ \sigma_c = 21.6 \text{N/mm}^2 \]

This stress is acceptable for glulam in compression and it is likely that the compressive strength will in fact be higher for stress-laminated timber.

There is however a significant issue that must be considered for stress laminated timber bridges especially in cold environments. According to Ref. [5] the coefficient of thermal expansion for timber perpendicular to the grain is about 2.5 times the coefficient of thermal expansion for steel. This can cause significant problems as the temperature drops, as the compressive effect of the steel rods across the timber will significantly reduce. This in turn may then lead to a reduced capacity in the stress laminated timber deck.

11 Creep

Timber creeps over time and is something that must be considered in timber design. The creep of the stress laminated timber deck will be considered by finding the deflection at midspan, the span is treated as two cantilevers, which is assumed to be 13m. The assumed loading is dead plus superimposed dead load. To find the deflection at midspan, the span is treated as two cantilevers, which support a (0.6 x span) long beam between them. The UDL on the deck is:

\[ w_{ud} = 4.2 \text{kN/m} \]
\[ w_{ud} = 4 \text{kN/m} \]

The effective point load, \( P \) from the ‘simply supported beam’ at the end of each cantilever is:

\[ P = \left( \frac{7.8}{2} \right) \times (4.2 + 4) = 32.0 \text{kN} \]

\[ \delta = 5 \frac{w_{ud}^4}{384EI} \, . \]  
\[ \delta = 5 \times (4.2 + 4) \times 7800^4 \times \left( \frac{384 \times 4500 \times 6.3 \times 10^9}{384 \times 4500 \times 6.3 \times 10^9} \right) \]
\[ \delta = 13.9 \text{mm} \]

The deflection at the end of the cantilever is:

\[ \delta = \frac{w_{ud}^4}{8EI} + \frac{P l^4}{3EI} \, . \]  
\[ \delta = \frac{(4.2 + 4) \times 2600^4}{(8 \times 4500 \times 6.3 \times 10^7)} + \frac{3200 \times 2600^3}{(3 \times 4500 \times 6.3 \times 10^7)} \]
\[ \delta = 8.3 \text{mm} \]

Therefore the total creep over time may be in the region of 13.9+8.3 = 22mm. However a full timber calculation would need to be carried out to access the creep characteristics of the deck properly. This may also be further complicated by the fact that the deck is stress-laminated timber. Therefore over time the steel tension bars may relax and the deck may deflect more than expected.

12 Durability

Durability is a major issue in this bridge due to the fact that it is almost entirely timber but also because of the reasonably low levels of protection applied. According to Ref. [3] the required design life, in Norway, for all bridges including timber bridges is 100 years. However the Leonardo da Vinci Bridge is not expected to reach this age. The design was passed on the grounds that the bridge could also be classified as a piece of public art. The estimated lifetime of the bridge is 40 years without any major reconstruction, however there is some doubt whether the bridge will even reach this age without significant refurbishment, Ref. [3]. This is obviously a serious weakness of this bridge, where a number of improvements could be made.

Due to very specific aesthetic requirements for the bridge, a large amount of possible surface finishes were turned down. This included metal, plastic and timber cladding systems. Additionally a number of chemical treatments were rejected due to either aesthetic reasons or due to environmental reasons (if is was felt that the finish was too detrimental to the environment.)

Ref. [3] states that a large number of treatments and preventative measures were used on the timber to try to improve the life span of the bridge. Firstly each individual laminate, for use in the arches, was pressure treated with a metal free agent called Scanimp. The laminates were then glued together, shaped and finished to create the final arch shapes. The arches were then pressure treated again with a wax emulsion to create a water repellent surface. Finally the entire structure was finished with oil stain to try to further prevent any water ingress. In addition to the environmental considerations this approach was taken to try and create the required aesthetic of a natural timber bridge.

Finally boron bars were inserted into predrilled holes near the base of each arch. These bars should protect the most exposed areas of the timber, where moisture ingress is most likely to occur. When these bars come into contact with moisture they begin to dissolve and this prevents the timber from rotting. These preventative measures do however require continuous future maintenance to be carried out on the bridge. The
boron bars must be regularly inspected and replaced if they are found to have reacted. Also the oil stain must be reapplied every two years. These are reasonable measures, however it must be ensured that this maintenance is carried out as specified, or the bridge could be seriously damaged.

Another design decision that has been made to increase the durability of the structure is the fact that the abutments have been raised up to limit the amount of ground water ingress into the bases of the glulam arches. Unfortunately even after these preventative measures cracks are already beginning to appear in the glulam sections, as shown in Fig. 10. These cracks are not considered to be a major problem at the moment, however it is likely that they could cause significant problems in the future. In the Norwegian climate freeze-thaw effects may cause serious damage to this structure. This effect occurs when water becomes trapped within the cracks and then as it freezes it expands and therefore further opens the crack. This will happen year after year with a ratcheting effect, possibly resulting in delamination and eventually structural problems. This is also an issue, as moisture will begin to reach areas of the timber, which are much less protected than the outer surface.

Figure 10: Cracks appearing in the glulam arches

Significant improvements could have been made in the protection of the glulam structure of this bridge. This may have most easily been achieved by cladding the glulam, however other methods such as chemical treatment or painting may also have been effective. It is understandable that the client was worried about ruining the bridge aesthetics, as this is such an important part of the bridge. However by working together, the design team should have been able to come up with a sympathetic solution, that would have provided good aesthetics but also allowed the bridge to meet the required design lifetime. This is especially true as it is likely that the bridge may have to be clad at a later time in its life anyway.

13 Vandalism

The use of timber does raise certain issues in terms of vandalism, as it is much easier for pedestrians to vandalise a timber structure. The form of the bridge also means that it is possible for people to climb onto the primary structure, which could be extremely dangerous. However it is always important to take into account the location of the structure. A steel or concrete bridge may be much more suitable in a deprived urban environment or alternatively a cladding solution could be used. However the design for the Leonardo da Vinci Bridge seems to be perfectly suitable for its location.

14 Future changes and improvements

There are not many major changes that can be made to this bridge in future. The main changes will probably be to do with the protection of the bridge in terms of durability and trying to extend its lifetime. This is most likely to be in the form of recladding the structure. This is an area that could have been improved in the initial design of the bridge. Additionally improvements could be made in terms of the refinement of design. As discussed details such as the arch base details soffit finish could have further improved the bridge from a closer distance. Overall however the bridge is extremely interesting and can be regarded as a success.

There are plans to build more bridges based on Leonardo da Vinci’s original concept all around the world and in different materials. This however will require a completely different design approach, depending on the material that is chosen. In terms of timber however this bridge has demonstrated the huge amount of possibilities for designing and building with glulam.

15 References


