A CRITICAL ANALYSIS OF W. T. CLARK’S SZÉCHENYI LÁNCHÍD

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Abstract: This conference paper aims to undertake a critical analysis of the Széchenyi Lánchíd as built by William Tierney Clark in 1849. It will analyze the aesthetics of the bridge, as well as the loading, strength and history of the bridge. The construction of the bridge will also be addressed, and the possible future of the bridge also.

Keywords: Széchenyi, Suspension, Chain, Danube, Budapest

1 Introduction

1.1 Background

William Tierney Clark was an English bridge designer in the mid 1800’s, designing the Hammersmith and Marlow suspension bridges before being contacted by Count István Széchenyi in 1832 to build the first permanent bridge across the River Danube in Budapest. This was part of Count Széchenyi effort as Hungary’s Minister for Transport to link Buda and Pest on either side of the Danube. Count Széchenyi had decided that the current pontoon bridge used in the summer and the occasional use of the ice in the winter to cross the Danube from Buda to Pest was not enough. Indeed, at the time the ice was forming and breaking in autumn and spring, there was no safe way to cross from Buda to Pest. The Count decided to build a permanent link between the two essentially separate cities to create an Hungarian capital, with which to instill the Hungarian people with a sense of national pride.

The funding of the bridge caused a problem however, and as such, the first Hungarian joint stock company was formed, with investment to be collected back via the continued use of tolls for crossing the bridge; not just for the commoners, but for nobles of Hungary, the first tax to apply to the nobility. This was
stipulated in the contract for the building of the bridge, as was the interesting requirement that no bridge be built within a mile of the chain bridge, with both provisos lasting for 87 years, the length of the contract. The original estimates by W.T. Clark in Ref. [2] placed the price at £300,000, however it actually came out at over £600,000.

Many of the first chain bridge’s built in England failed principally “from want of proper knowledge and experience.” This was the view of W.T. Clark in the ‘Report on the proposed bridge at Pesth’. However Clark had built The Hammersmith and Marlow Bridges before this and many engineers supported the idea of a chain suspension bridge in Budapest, the only voice opposed being an American called Wright, who had reservations about oscillations.

However Count Széchenyi felt that this could be handled after seeing Brighton Bridge in a storm. The Count was also a fan of a chain bridge’s comparatively low cost and it’s stability under high loading, seeing Clark’s Hammersmith Bridge during the Oxford-Cambridge Boat Race. W.T. Clark therefore submitted three designs for the bridge, all 3 span suspension chain bridges, with another four being submitted by George Rennie. Rennie’s though were either considered too unstable or dangerous, due to long spans causing oscillations, or short spans causing ice dams. Even Brunel had a design for the bridge, although he didn’t submit it to Hungary.

**Figure 2:** Brunel's Budapest bridge design. Ref. [1]

Two experts employed by the Hungarian’s, John Plews and Samuel Slater, stated that any bridge crossing the river had to:

1. Have as few piers as possible in the river bed.
2. Have flawless foundations.
3. Not be located at the narrowest stretch.

**Figure 3:** Gresham Palace and the Chain Bridge.

The Hungarians felt that these three statements were best supported by Clark’s three span chain bridge in the style of Nákó House, now known as Gresham Palace shown in Fig. 3, at the end of the proposed bridge site. The Hungarian Government commissioned his plans on the 18th November 1838.

Just as the bridge was being finished by Scottish engineer Adam Clark however, the Austrian army was retreating following the Hungarian revolt of 1848. Pest was shelled from 7 am till noon, however only one cannonball hit the Buda side pier, only damaging the façade. This damage was not repaired until 1914. The Austrian Colonel then attempted to blow up the bridge with 400kg of gunpowder. Using his cigar to light the fuse led to Colonel Allnoch being torn to bits by the explosion, although the bridge sustained only slight structural damage, which was repaired immediately. More damage would have been done, but Adam Clark ordered the anchor chambers for the chains to be flooded and destroyed the pumps, stopping the anchors from being destroyed. Again in 1848, the unfinished bridge had its first major test with the rebel army of 70,000 soldiers and 300 pieces of military equipment crossing over it; and the Austrian General Dembinszky ordering it to be destroyed, however Adam Clark managed to convince the general to just let him render the road surface temporarily unpassable. Somehow, the bridge still stood for its official opening on the 20th of November 1849. However, at the time, it could not receive the name Széchenyi Lánchíd because of Count Széchenyi’s involvement in the Hungarian revolution and subsequent house arrest and mental breakdown. This name was only given in 1899, on the 50th anniversary of the bridge’s opening. This bridge though, united the people of Buda and Pest, and gave them the required pride to start on the road to a nation separate from the Austrian rule.

**Figure 4.** Original woodcut of the bridge after its completion, from Ref. [7].

The main reason for a suspension bridge to be used here, was not because of Count Széchenyi’s admiration for the current British use of these bridges, but because of the ice flow down the river. The piers of the bridge needed to be quite large to resist the forces of this ice flow, and the spans needed to be long so that ice couldn’t block the river, causing a natural dam and inevitable flooding. As such, there appeared to be only one style of bridge that would suit both of these necessities. The suspension bridge, by its very nature
would have both large piers and spans, making it ideal for this particular river crossing between Roosevelt Square in Pest and what is now called Adam Clark Square in Buda. These were presumable the reasons for the three statements made by Plews and Slater that convinced the Hungarians to employ W.T. Clark to build this monument of a bridge.

1.2 Rebuilding.

The bridge was strengthened in 1914 to account for the increased traffic. This was mainly due to the level of oscillations that had been observed under heavy traffic, as noted in Ref. [6]. However this structurally repaired bridge was so severely damaged by the Germans in the Second World War that it had to be rebuilt. The towers only needed to be repaired, to make them as close as possible to how they were before; however the deck and cables were rebuilt to take an ever increasing loading and the width of the carriageway was increased from 5.45m to 6.45m, something that could not have been done before due to the roadway passing through the towers. The original 1849 structure used wrought iron chains and wooden stiffening trusses in the railings; however these had both been replaced with high-carbon open-hearth steel in 1914, and 76% of these chains were able to be reused in the 1949 structure. The hangers were also changed at this point to mild steel, also to increase the load the bridge could take. In many ways, if the German’s hadn’t almost destroyed the bridge, there may be good reason for it to have been completely rebuilt anyway. However this bridge, as the first permanent bridge in Budapest, needed to stay as a cultural feature of the city and this meant that further strengthening needed to occur at some point.

2 Aesthetics

In analyzing the aesthetics of any bridge, it is important to remember that what one person considers a beautiful structure, someone else may believe to be nothing worthy of note. Therefore, in order to study the aesthetics in a regular manner and compare separate bridges in these conference proceedings, Fritz Leonhardt’s ten rules of aesthetics in bridge design can be used as a benchmark to aesthetic quality.

The first thing that must be noted with any work by W.T. Clark is that he always laid great emphasis on the aesthetic quality of his work; rejoicing when he heard that a watchman was to be placed at Marlow Bridge, solely to prevent vandalism. Marlow Bridge and the Széchenyi Lánchíd are the only structures designed by Clark that still stand today, albeit with both having been reconstructed. The Széchenyi Lánchíd has a plaque commemorating this fact under one of the lion statues.

2.1 Fulfilment of Function

The Széchenyi Lánchíd is clear in its function and structure. The large towers support the suspension chains which in turn support the deck. This gives a sense of structural stability that is common to most suspension bridges.

2.2 Order and Proportions within the structure.

The middle span of the bridge is nearly twice as large as the two identical outer spans. This lends the bridge good order and proportions.

2.3 Character

However the towers are somewhat large by today’s standards, but there are reasons for this and this just adds to the overall character of the bridge, along with the two-tiered chains and the history of the bridge. The two-tiered chain system was first used by Clark in the Hammersmith Bridge, with the Széchenyi Lánchíd being his most famous and revered two tiered chain suspension bridge.

2.4 Surface Texture and Integration into the Environment.

The towers are made of local stone, which blends in well with many buildings surrounding the bridge, including Buda Tunnel, which was also built by Adam Clark.

2.5 Refinements in Design

In terms of refinements in design, the towers taper towards the top, a common refinement that prevents them from seeming to be overbalancing. The two-tiered chain system also allows the chain links to be twice as long and not as stocky compared to if one chain were to have been used.

2.6 Colour

The chains are also a dull green colour; however this fits with the style and era of bridge and doesn’t look out of place in its setting. With the antique-style lamp posts in the same colour and the stone used in the towers an unremarkable yellow, the effect is to make the viewer focus on other aspects of the bridge, like the tower and chain sizing, or the decorative sculpting of the top of the towers like an ancient column capital.

Figure 5: Original detail of the piers, including the lamp posts and column capital styling. Ref. [2]
2.7 Aesthetic Conclusions and other visual aspects.

The lighting on the bridge is used to highlight the iron structure, with bulbs on the chain and pavement curbs and more general floodlighting on the tower. The lions that guard the ends of the bridge were mounted in 1852; sculpted by János Marschalkó, they combine well the classical arches and add to the character of the bridge.

![One of Janos Marschalkó's lion sculptures and the lighting on the bridge at night.](image1)

Overall, this bridge can be appreciated both for its amazing history and an iconic image, highlighted and accentuated by clever lighting.

3. Construction

The first thing that should be noted about the construction of this bridge, is that the placement was decided, not on engineering issues, but solely on where land could be bought either side of the bridge. This, however, had little effect upon any possible foundation issues, as the bedrock under the Danube along this stretch of river is much the same along the length of the river here. The first piles completed were testing piles done in 1839, however the first real pile installed was on the 28th July 1840 under the Buda side pier.

![Cofferdam and icebreaker for layout for construction use. Ref. [2]](image2)

Piling continued for the next two years, however an icebreaker called the ‘Dolphin’ was installed on the 11th March 1841, to prevent damage to the cofferdams surrounding the pier installation areas. These cofferdams were themselves rammed into place using the same 1.7 ton weight as the actual piles, and were the thickness of three piles, arranged all the way round the piling area, with the Dolphin facing upriver. The foundation stone for the first abutment was ceremonially laid on the 24th August 1842. The piers and substructure of the bridge was completed in 1847, with the first chain lifted into place on the 28th March 1848.

The lifting of these chains was done sequentially; each span, tier and side of the bridge done separately, meaning that there were 12 chains to be lifted and attached. The twelfth chain was lifted as part of a ceremony at 6 pm on the 19th July 1848, however, this was the only chain that caused any problems, with one link failing and dropping the whole chain to the riverbed, causing the death of one worker, and instant panic. The final chain was picked up from the riverbed and lifted into place in August 1848, without ceremony. This was considered at the time to be quite quick for a chain bridge, due in no small part to Adam Clark’s innovative lifting mechanism, comprised of a crane and six-fold pulley mechanism, powered by a 25 horsepower onshore steam engine. The steel structure was mounted on a floating scaffold, with the reinforced concrete sheets placed on the steel structure, and asphalt poured on top to make the roadway. The bridge was passable in early 1849, and officially opened, after many inconvenient problems, on the 20th November 1849.

The materials used were sourced either from the surrounding countries, or directly from England. Both William Tierney Clark and Adam Clark specified that only the best material should be used. This annoyed the nobles financing the project, who believed that both Clarks were stealing the money for themselves. However after the gunpowder attack and shelling of the city made little impression upon the bridge, the nobles admitted the use of superior materials, and indeed workmanship, was of great benefit, regardless of the cost. The bridge did end up over budget and somewhat late, however it stands, albeit in a slightly changed form, to this day.

The stone used came from a quarry made especially for this bridge, and the stone from it was granite. The facade of the piers uses carved Mauthausen Granite, as are the icebreakers and structural base stones that carry the shoes of the chain saddles. Most of the original iron was from England; however businessmen like Rothschild and Andrassy demanded that some iron came from their foundries in Austria. The need for the best structural elements however demanded that at the very least the whole chain had to come from England, and so it was shipped over and up the Danube to a temporary wharf next to the site of construction.

This wharf was the first that Adam Clark had ever had to build, and as such, he had his father secretly take measurements of Blackbow wharf using his walking stick and send him details of the connections. This showed that Clark would use whatever way possible to finish the engineering of the bridge. This was just one aspect of Adam Clark’s work ethic that convinced the Count and W.T. Clark to hire him on for the entire bridge project, as opposed to just the piling as had originally been agreed. The timber for the wharf, cofferdams and foundations was either oak or larch, from Slavonia in Croatia and Styria in Austria respectively. The bricks used as infill for the piers came from Steinberger and Lechner’s factory in Pest and Ceskô and Chisten’s factory in Buda, to maintain the idea of linking the two separate towns together. All cement used came from a new factory
on Buda’s banks, with the marl shipped in from Beočin in Voivodina in modern day Serbia.

Figure 8: Piers showing masonry and infill section and piles. Ref. [2]

The foundations contained thousands of piles, each between 20 and 24 metres in length and were 38 centimetre squares in cross-section. The ram used weighed 1.7 tons, dropped from 6.7 metres and at one point there were 800 hands working on the foundations. Each pile took on average 400 beats to put in, with a 1.5 metre distance between the rows of piles. The structure of the foundations is supplemented by grid timbering and cross propping.

The foundations for the abutments were 2.74 metres deep, 10.67 metres wide and 42.67 metres long. They were constructed from solid rubble and hydraulic lime. These abutments contained the concrete anchoring chambers for the chains, that then went up and over the piers, resting originally on fixed saddles, however this was changed to a steel roller system in 1914 so that only vertical forces were imparted upon the piers.

Figure 9: Steel roller system to support chains. Ref. [1]

These allowed the pressure acting on the soil to be reduced by half. This did not make much difference though because of the increase in traffic meant that the loading on the bridge had increased. The increased force taken by the chain to the anchoring chambers meant that 5000 cubic metres of concrete had to be placed on either side of each anchoring block. However the increased loading did not include trucks over 13 tons, as these were ordered to be re-routed to Margaret Bridge in 1876. However the 1914 strengthening needed to occur because the level of swaying under the increased traffic was deemed unsuitable. It was also seen that merely increasing the bracing to reduce swaying could not fit on the bridge, so the steel structure was replaced.

Originally this structure consisted of chain links that were alternating between twelve and thirteen sheets wide, were 70 cm wide at the joint and 36 cm high. Each link was a different length so that the hangers were always spaced 1.8 metres apart, the hangers alternating between the top and bottom chains. The chain sheets were superior wrought iron by Howard and Rovenhall in England. The original saddle and anchor castings were by Hunter and English. The deck consisted five of latticed double girders, upon which rest the crossbeams. Together these supported the wooden deck. Much of the iron here originally came from Rothschild and Andrassy’s factories.

The rebuilt structure, taking into account the 1914 and 1947-49 work, has brand new stiffening girders, chains and deck, with the metal used now a mild steel. The wooden deck was replaced with 15 cm thick dual-layer reinforced concrete sheets, which are covered with poured asphalt. The footways are only on 10 cm reinforced concrete sheets. The piers were also widened in 1949 to allow two buses to pass through at the same time.

The road layout also changed so that the central span was no longer slightly higher than the side spans. This hadn’t mattered when the mode of transport had been horse and carriage; however this made a significant difference to the cars now crossing the bridge. During the 1914 work, ice floating down the Danube meant that much of the falsework centering had to be removed for the winter. In 1973, the bridge was repainted and X-ray tests were conducted randomly on the metal parts of the structure. These tests showed that the bridge was performing satisfactorily in terms of fatigue, and there were no problems with rust.

Figure 10: Schematic of the deck, with dimensions. Ref. [3]
4. Loading

Bridges today are designed to their country’s relevant standards, however in 1849, Hungary didn’t have a relevant bridge standard and as such this bridge was built to the best practice of the era. The following loading analysis is based upon the British Standard for steel, concrete and composite bridge design, using the load combinations and definitions therein, as given in Ref. [5].

The values for the loads all need to be factored, with the two factors being \( \gamma_f \), and \( \gamma_3 \). \( \gamma_f \) is a factor taking into account the load combination whilst \( \gamma_3 \) is a factor for inaccuracies in bridge type analysis. Under serviceability limit (SLS) state, the \( \gamma_f \) factor is usually equal to 1.00, except for certain loads, for example, superimposed dead loading is loads to a factor of 1.20 in all five load combinations. For ultimate limit state (ULS), BS 5400 contains a table that gives the load, the load combination and it’s relevant value of the \( \gamma_f \) factor. The \( \gamma_3 \) factor is equal to 1.00 under SLS, whilst under ULS it changes depending upon the main structural material and the type of analysis used. For all steel bridges, the factor used is 1.10, as does elastic analysis on concrete bridges, whilst plastic analysis of concrete bridges yields a factor of 1.15. These factors apply to all loads in all load combinations. The combinations to check under both ULS and SLS are:

1. All permanent loads plus primary live loads.
2. Combination 1 plus wind (and temporary loads if erection considered).
3. Combination 1 plus temperature (and temporary loads if erection considered).
4. All permanent loads plus secondary live loads and their associated primary live loads.
5. All permanent loads plus loads due to friction at supports.

These combinations must be made in such a way that they portray the worst possible stresses within the structure. Any relieving action that cannot be removed (ie.dead load from the bridge deck) can have its factors lessened to minimize its impact.

At ULS loading, the bridge will need to be structurally tested to make sure that there is no major structural failure. At SLS loading, the bridge merely needs to conform to the rules that are applied in BS 5400. All factoring in the following calculations is done for combination 1, or if not in combination 1, for the relevant combination.

4.1 Dead Loading

The dead loading is the load of the steel substructure and the concrete deck combined together. This includes the longitudinal and the transverse steel, but not the road surfacing and street furniture as these are included in the superimposed dead loading. The volume in the deck of longitudinal steel is 53 cubic metres, whilst the transverse steel is 13.2 cubic metres. The volume of concrete in the deck is 534.85 cubic metres.

\[
\text{Volume of steel} = \frac{53+3.67}{380} = 0.54/380 = 13.26kN/m
\]

\[
\text{Factored steel dead load} = 4.36x1.05x1.1 = 5.04MN
\]

\[
\text{Factored concrete UDL} = \frac{12.59x1.15x1.1}{380} = 15.93MN
\]

\[
\text{Factored dead load UDL} = 41.91+13.26 = 55.2kN/m
\]

4.2 Super-imposed Dead Loading

The super-imposed dead loading contains the road surfacing and the street furniture, for example the lamp posts and parapets. These are difficult to give an accurate value of loading to and as such I will give a conservative value of 0.5kN/m². This will need to have the same factors applied to it as all other aspects of loading. At ULS, the factors applied for superimposed loading in combination 1 are 1.75 for \( \gamma_f \) and 1.10 for \( \gamma_3 \). The \( \gamma_f \) value is significantly higher than for just dead loading to account for the unpredictability of presence of this form of permanent load.

\[
\text{Factored Super-imposed dead loading} = \frac{0.5x11.92x1.75x1.1}{380} = 11.5kN/m
\]

4.3 Primary Live Loading

The primary live loading in this case is traffic loading. This is done in BS 5400 using HA and HB loading; normal heavy traffic and an abnormal truck load.
This bridge can be expected to fail the HB loading as it was not designed to take these loads and in fact has a weight limit imposed upon it. The HA loading consists of a UDL and a knife edge load (KEL). The UDL is dependent upon the length of the loaded length of the notional lanes. These notional lanes are taken from BS 5400, with the Széchenyi Lánchíd being 6.45m, therefore having 2 notional lanes, one in either direction, with no central reservation, so HB loading has to occur also. If there had been a central reservation, with a notional lane width of less than the width of the truck used in HB loading, there would be no need to test for HB loading.

The loaded length of the bridge is the central span of 202m. This gives a UDL of 12.2kN/m along the length of each notional lane. Since there are two notional lanes, the total unfactored UDL is 24.4kN/m along the central span of the bridge; with an additional unfactored 120kN KEL placed at the most adverse point. These loads need to be factored, with the relevant factors. The $\gamma_f3$ factor is still equal to 1.10, whilst the $\gamma_fL$ value at ULS is 1.50.

\[
\text{Factored primary live HA UDL loading} = 24.4 \times 1.10 \times 1.50 = 40.3kN/m \quad (9)
\]

\[
\text{Factored primary live HA KEL loading} = 120 \times 1.10 \times 1.50 = 198kN \quad (10)
\]

The HB loading encountered is an abnormal truck of four axles, with each axle having four wheels placed 1 metre apart along the axle. The two front and back axles are placed 1.8 metres apart, with the central span of the truck being either 6, 11, 16, 21 or 26 metres, whichever has the most adverse effect. Each wheel carries a nominal load of 112.5kN, derived from each axle taking 10kN per unit of HB loading. There are 45 units of HB loading in total per truck, which is spread evenly throughout the wheels.

### 4.4 Wind Loading

The wind loading for this bridge uses certain assumptions. The mean hourly wind speed, $v$, is an assumed 30 m/s. This is multiplied by a number of factors to achieve the maximum wind gust velocity, $v_c$. The $K_t$ factor is a wind coefficient for converting $v$ into $v_c$. $S_1$ is a funneling factor, assumed to be 1.00 unless funneling is considered likely. $S_2$ is a gust factor, given in BS 5400. $K_t$ is assumed to be 1.29, with $S_2$ taken to be 1.00.

\[
v_c = 30 \times 1.29 \times 1.00 \times 1.00 = 41.1m/s \quad (11)
\]

The wind loadings come in combinations as well. These combinations are:

1. $P_t$ only.
2. $P_t$ +/- $P_v$
3. $P_t$ only.
4. $0.5P_t$ + $P_t$ +/- $0.5P_v$

All of these combinations need to be checked to see which one gives the worst effects. In the following calculations, $q$, the dynamic pressure head, is taken to be $0.613v_c^2$. $A_1$ is the solid horizontal projected area, and $A_2$ is the plan area. $C_D$ is the drag coefficient and $C_L$ is the lift coefficient. $C_p$ is a factor dependant on the ratio of breadth to depth of wind affected area. The graph provided in BS 5400 gives a value for $C_D$ of 1.3, whilst the $C_L$ value is 0.4 and is also dependant upon the breadth upon depth ratio. However in calculating the $C_D$ value for $P_{LL}$ is 1.45.

\[
q = 0.613v_c^2 
\]

\[
q = 1035.5 
\]

\[
A_1 = 202 \times (2.5 + 1.62) 
\]

\[
= 832.24m^2 
\]

\[
A_3 = (8.87 + 1.29 + 2.17 + 2.17) \times 202 
\]

\[
= 2929m^2 
\]

\[
A = 0.613 \times 41.1^2 
\]

\[
= 1035.5 
\]

\[
P_t = qA_1C_D 
\]

\[
= 1035.5 \times 832.24 \times 1.3 
\]

\[
= 1.12MN 
\]

\[
P_v = qA_3C_L 
\]

\[
= 1035.5 \times 2929 \times 0.4 
\]

\[
= 1.21MN 
\]

\[
P_L = P_{LS} \text{ or } P_{LS} + P_{LL} 
\]

\[
P_{LS} = 0.25qA_1C_D 
\]

\[
= 0.25 \times 1035.5 \times 832.24 \times 1.3 
\]

\[
= 1.12MN 
\]

\[
P_{LL} = 0.5qA_1C_D 
\]

\[
= 0.5 \times 1035.5 \times 832.24 \times 1.45 
\]

\[
= 1.25MN 
\]

\[
P_L = 1.12 + 1.25 
\]

\[
= 2.37MN 
\]

\[
P = qA_1C_D 
\]

\[
= 0.613v_c^2 
\]

\[
v_c = 41.1m/s 
\]

\[
P_t = 1.12 \times 1.1 \times 1.1 
\]

\[
= 1.36MN 
\]

\[
P_v = 1.21 \times 1.1 \times 1.1 
\]

\[
= 2.82MN 
\]

\[
P_L = 2.37 \times 1.1 \times 1.1 
\]

\[
= 2.87MN 
\]

\[
P_{LL} = 0.5 \times 1035.5 + 2.87 + 0.5 \times 1.46 
\]

\[
= 4.28MN 
\]

This shows that case 4 is the worst wind loading case, and so should be used in the overall load combinations.
load combination 2. This load needs to be converted to a UDL to be spread across the central span.

\[
\text{Factored Case 4 Load as UDL} = 4.28 \\times 202 \\
= 21.2 \text{kN/m}
\]  
(25)

4.5 Temperature Effects

The Széchenyi Lánchíd has no expansion joints built into the deck, and as such, temperature differences will cause appreciable stresses within the structure. These stresses will occur due to both the overall temperature differences and due to a variation in temperature between the top and bottom deck. The maximum and minimum temperatures regularly recorded in Budapest itself are 28°C and -4°C respectively.

The coefficient of thermal expansion for both steel and concrete is \(12 \times 10^{-6}/°C\). The Young’s Modulus of concrete is 30 kN/mm\(^2\) and for steel is 200 kN/mm\(^2\).

\[
\varepsilon = \alpha \Delta T \\
= 12 \times 10^{-6} \times (28 - (-4)) \\
= 384 \mu \varepsilon
\]
(26)

This strain can then be turned into a stress by multiplying it by the Young’s Modulus. In this case the stress in the concrete is different to the stress in the steel. Firstly, the stress in the steel is:

\[
\sigma = E \varepsilon \\
= 200 \times 384 \\
= 76.8 \text{N/mm}^2
\]
(27)

Whilst the stress in the concrete is:

\[
\sigma = E \varepsilon \\
= 30 \times 384 \\
= 11.5 \text{N/mm}^2
\]
(28)

These values of stress will be combined with the values of stress given by the other loads in load combination 3 to give the final stress values. These however are the stress differences between the maximum and minimum temperatures. Assuming the structure was installed at the average temperature between the two maximum and minimum temperatures, the stresses involved will be half the above stresses, and will need to be applied as either compressive or tensile stresses, whichever gives the worst effect.

There is also of course the effect of a difference in temperature between the top and bottom of the deck structure, which can also cause significant stresses. This is more complicated to calculate, as the difference in temperature changes across the change in material in the deck, from concrete to steel.

4.6 Ice Loading

The Danube freezes often; in the history of this bridge, there is at least one time at which ice floating down the Danube caused the strengthening on the bridge to be postponed in 1914. The loading analysis below is taken from the Canadian code CSA S6 from 1978 as given in Ref. [4]. The given formula is:

\[
F_b = C_1 C_2 p b t
\]
(29)

Where \(C_1\) is a coefficient related to the angle of the pier nose from the vertical; \(C_2\) is related to the ratio of \(b/t\); \(p\) is the effective ice pressure; \(b\) is the pier width at water level; \(t\) is the ice thickness.

The relevant value for \(p\) given in the Canadian code is 690 kPa, for ice in small sections. The value of \(b\) is 11.5 m, and \(t\) is assumed to be 0.75 m, giving a \(C_2\) value of 1.0. The \(C_1\) value is 0.5 because of the pier inclination of this bridge are between 30° and 45° from the vertical. This gives a value for \(F_b\) of:

\[
F_b = 0.5 \times 1 \times 690 \times 11.5 \times 0.75 \\
F_b = 2975.6 \text{kPa}
\]
(30)

This force is applied in the direction of water flow.

5. Strength

The idea of a suspension bridge is that the deck can be slim, with the weight being taken on the hangers and suspension chain or cable.

\[
T = \sigma A \\
= 460 \times (350 \times (700/2)) \\
= 56.4 \text{MN}
\]
(31)

This tension is related via Pythagoras’ Theorem to the vertical and horizontal reactions in the pier. These can be expressed in terms of the loading, \(w\), length of span, \(l\), and distance from deck to chain saddle, \(f\).

\[
H = w l^2 / 8 f \\
V = w l / 2
\]
(32)

\[
T^2 = H^2 + V^2
\]
(33)

Which in this case yields

\[
T = 19.6 w^{0.5}
\]
(34)

Therefore

\[
T = (56.4/19.6)^2 \\
T = 8.28 \text{MN/m}
\]
(35)

This is the value of load that can be taken by each chain, meaning that over all 4 chains, the total loading on the deck can take is 33.1 MN/m. This is far above any of the relevant load case combinations stated above, and as such, it can be seen that the chains will not
break under dead, HA and wind loading, combination 2, often the worst combination for suspension bridges.

6. Vibrations

Although this bridge is a road bridge and not a footbridge, it does have access for pedestrians. This access means that the bridge should be checked for vibrations using the bridge’s natural frequency. Although the bridge has been strengthened to minimize vibrations under loading from cars, the pedestrian aspect of the bridge means that certain other requirements must be met at SLS to allow for the psychological effects the vibrations can have on pedestrians. At less than 5Hz, the maximum vertical acceleration needs to be checked to ensure that it is less than half the square root of the fundamental natural frequency. For natural frequencies of more than 5Hz and less than 75Hz, the bridge is deemed suitable.

6.1 The Effect of the Stiffening Trusses

In this bridge’s case, the parapets between roadway and footway act as stiffening trusses to increase the second moment of area of the bridge deck as can be seen in Fig. 10. The distance from the bottom of the parapets to the Neutral Axis is the first step in finding the overall second moment of area,

\[
\text{Distance to NA from bottom of Parapets} = (A_d + A_p) / (A_d + A_p + A_s)
\]  

(36)

Table 1: Definition of the parameters used in Eq. (36)

<table>
<thead>
<tr>
<th>Definition of Parameters</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of concrete ((A_c))</td>
<td>1618500mm²</td>
</tr>
<tr>
<td>Area of longitudinal steel ((A_s))</td>
<td>139500mm²</td>
</tr>
<tr>
<td>Area of parapets ((A_p))</td>
<td>296901mm²</td>
</tr>
<tr>
<td>Distance to concrete centroid ((d_c))</td>
<td>1281mm</td>
</tr>
<tr>
<td>Distance to steel centroid ((d_s))</td>
<td>926mm</td>
</tr>
<tr>
<td>Distance to parapet centroid ((d_p))</td>
<td>1532mm</td>
</tr>
</tbody>
</table>

The distances described in the above table are all taken from the bottom of the parapet. When the numbers are applied into Eq. (36), the distance from the bottom of the parapets to the Neutral Axis is 1293.2mm. This is 87.2mm above the longitudinal steel and concrete interface. If there were no parapets, the Neutral Axis would be 46.8mm above the interface of steel and concrete. Using the parallel axis theorem to find the second moment of area,

\[
I = \sum (I_{local} + y^2 A)
\]  

(37)

Table 2: Definition of parameters used in Eq. (37)

<table>
<thead>
<tr>
<th>Definition of Values</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Second moment of area of the system</td>
<td>(I)</td>
</tr>
<tr>
<td>Second moment of area of each section</td>
<td>(I_{local})</td>
</tr>
<tr>
<td>Distance from NA to centroid of section</td>
<td>(y)</td>
</tr>
<tr>
<td>Area of relevant section</td>
<td>(A)</td>
</tr>
</tbody>
</table>

The three variables within the equation can be worked out for each relevant section.

Table 3: Values for concrete with parapets

<table>
<thead>
<tr>
<th>I-values</th>
<th>(I_{local}=bd^4/12)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(I=3.29\times10^{10}mm^4)</td>
<td>(y=87.2-75=12.2mm)</td>
</tr>
<tr>
<td>(I_{local}=3.05\times10^{10}mm^4)</td>
<td>(A=1618500mm^2)</td>
</tr>
</tbody>
</table>

Table 4: Values for concrete without parapets

<table>
<thead>
<tr>
<th>I-values</th>
<th>(I_{local})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(I=4.34\times10^{10}mm^4)</td>
<td>(y=75-46.8=28.2mm)</td>
</tr>
<tr>
<td>(I_{local}=3.05\times10^{10}mm^4)</td>
<td>(A=1618500mm^2)</td>
</tr>
</tbody>
</table>

Table 5: Values for longitudinal steel with parapets

<table>
<thead>
<tr>
<th>I-values</th>
<th>(I_{local})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(I=2.64\times10^{10}mm^4)</td>
<td>(y=280+87.2=367.2mm)</td>
</tr>
<tr>
<td>(I_{local}=7.565\times10^{10}mm^4)</td>
<td>(A=139500mm^2)</td>
</tr>
</tbody>
</table>

Table 6: Values for longitudinal steel without parapets

<table>
<thead>
<tr>
<th>I-values</th>
<th>(I_{local})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(I=2.25\times10^{10}mm^4)</td>
<td>(y=280+46.8=326.8mm)</td>
</tr>
<tr>
<td>(I_{local}=7.565\times10^{10}mm^4)</td>
<td>(A=139500mm^2)</td>
</tr>
</tbody>
</table>

Table 7: Values for parapets

<table>
<thead>
<tr>
<th>I-values</th>
<th>(I_{local})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(I=2.77\times10^{11}mm^4)</td>
<td>(y=1532-1293.2=238.8mm)</td>
</tr>
<tr>
<td>(I_{local}=2.323\times10^{11}mm^4)</td>
<td>(A=791738mm^2)</td>
</tr>
</tbody>
</table>

Table 8: Values for overall second moment of area.

<table>
<thead>
<tr>
<th>I-values</th>
<th>(I_{local})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(I=3.07\times10^{11}mm^4)</td>
<td>(I=2.68\times10^{10}mm^4)</td>
</tr>
</tbody>
</table>

As can be seen in Table 8, the parapets add orders of magnitude to the second moment of area of the section, despite being less than half of the area of the concrete used in the deck. To use these values in the natural frequency equation, Eq. (38), we must have the second moment of area in m⁴ as opposed to the standard units of mm⁴. These second moments of area in m⁴ are 0.307m⁴ and 0.0268m⁴ respectively. The formula given is:

\[
f_0 = \frac{\beta l l}{\sqrt{(EI/ml)}}
\]  

(38)

Where \((\beta l l)\) is equal to 22.37 for a bridge with two fixed connections as the piers here are. \(E\) is the Young’s modulus in N/mm². \(I\) is the second moment of area, \(m\) is the mass per unit length of the unloaded bridge in kg/m and \(l\) is the length of the longest clear span in metres. In this case the natural frequency for the bridge with parapets is:

\[
f_0 = \frac{22.37\sqrt{(2\times10^2\times0.307)^2/(4547\times202)}}{5.8Hz}
\]  

(39)

This value of 5.8Hz is just acceptable as it is above the 5Hz limit. If the bridge did not have the stiffening trusses as parapets, this value would be
The parapets make a huge difference to the natural frequency of the bridge and the bridge would need some form of stiffening if these were to be removed, either purposefully or in an accidental collision.

It should also be noted that this is calculated as if this were solely a footbridge. The value of mass having the traffic load on it would change this answer, however this would introduce too many factors affecting the vibration of the bridge to be feasible.

7. Inspection and Maintenance

All structures need to have regular checks, to ensure that they maintain structural stability. Bridges in particular have undergone some spectacular failures in their past; failures that sometimes have been discovered during the inspection periods, but sometimes have only been exposed after the event. For example, the Forth Road Bridge underwent routine inspection and engineers heard the sound of individual wires snapping within the suspension cable. This was due to water managing to get through the vapour barrier surrounding the suspension wires. The Forth Road Bridge is now undergoing dehumidification of the suspension cable to stop further wire failures; however some irreparable damage had already been done.

Of the same relevance for the Széchenyi Lánchíd was the random x-ray inspections done when the bridge was repainted in 1973. These showed no significant failure of any structural parts, with all parts found to be fine in terms of fatigue. However a structural failure within any part of the chain would have far more catastrophic consequences for the Széchenyi Lánchíd. However with consistent monitoring and inspection, there is no reason why this bridge won’t stand, albeit with probably more adjustments and replacements or repairs, for another 150 years. Indeed, during and after rebuilding, the bridge underwent analysis, both handwritten and computer, to determine its suitability under the Bridge Department of the Ministry of Transport’s loading. This included modeling buses (18kN/m) and passenger cars (0.5kN/m) along separate 24m sections of the bridge.

The bridge itself needs little structural maintenance over any long period, but when any repairs are done, they tend to be on a large scale, like the 1914 restrengthening which required falsework to be placed underneath the bridge to maintain structural stability.

Figure 12. Computer analysis of the bridge originally by Professor F. Papp from Ref [3].

8. Conclusions

It can be relatively safely assumed that the future of this bridge is safe, as it is a bridge that is seen as a major link with the city’s past, it’s beginning as a single city indeed. The bridge has been restrengthened, rebuilt and had laws passed for its benefit; and as such there can be no doubt that all efforts will be made by most Hungarians to save this bridge, both for its aesthetics and its history. The bridge is a monument as much as a simple roadway and is a symbol of the Hungarian pride in their nation.

Acknowledgements

I would like to thank the ICE library for their help with finding W.T. Clark’s account of his bridge, Ref. [2].

References