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A Critical Analysis of Kylesku Bridge

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Abstract: The aim of this paper is to perform a critical study of Kylesku Bridge. The areas of interest for this include the background of the bridge, aesthetics, construction, loading, strength, geotechnics, serviceability and durability. Calculations have been performed where necessary to aid the understanding of key aspects, such as for some of the loading conditions and the strength of the deck during the worst case scenario.

Keywords: *Kylesku Bridge, Loch a' Chàirn Bhàin, Incremental Launching, Prestressed concrete, Dyform Strands*

1. Introduction

The history of the site, located in Sutherland in the Scottish Highlands, dates back to the early 1800s, which saw the first efforts made to cross Loch a' Chàirn Bhàin in the form of a rowing boat for passengers. However, as commercial traffic consisted mostly of cattle being herded towards central Scotland, having just a small rowing boat forced the farmers to force their cattle to swim across. So since then larger ferries have been introduced, culminating in the Maid of Glencoul in 1976, the first of the ferries at this site capable of transporting fully loaded commercial vehicles. The ferry saved motorists the need to take a 100 mile detour via Lairg, and so as demand for the route increased, a more practical solution was of great importance. Therefore, the Kylesku Bridge was constructed in 1984 by Morrison Construction Group. It is one of the few beautiful concrete bridges in the UK, and has been described as one of the most beautiful bridges in the world, winning several awards.

Figure 1: Photo of Kylesku Bridge

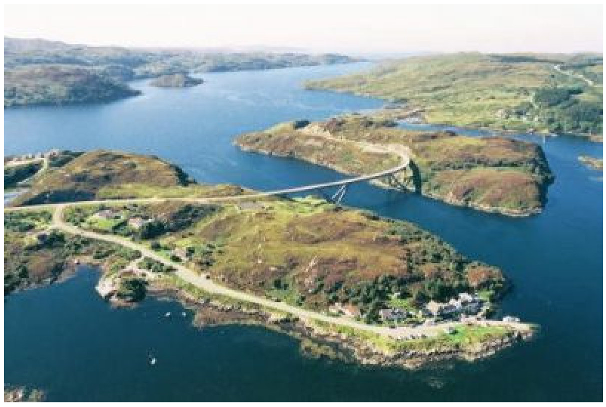

Figure 2: Photo of Loch a' Chàirn Bhàin

The total length of Kylesku Bridge is 277m. This is broken down into 5 spans, the longest of which is 79m long. It is a portal frame, meaning that it has roughly the same clearance along its length, which is 24m over the water. It has two traffic lanes and a footway on one side only, amounting to a total width of about 9.3m at the widest point, 8.2m at the thinnest. There are 8 20m high piers, all of which inclined and paired together in V shapes. There are two of these V-piers on each side of the bridge, 16m apart at ground level, but also inclining towards the centre. The deck is a prestressed concrete box girder, and it is rigidly jointed to the piers.

2. Aesthetics

2.1. Aesthetics Overview

There are 10 rules that were set out by Fritz Leonhardt for the aesthetics of a bridge, and they are as follows:

1. Fulfilment of function
2. Proportions of the bridge
3. Order within the structure
4. Refinement of design
5. Integration into the environment
6. Surface texture
7. Colour of components

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8. Character
9. Complexity in variety
10. Incorporation of nature

If a bridge were to adequately fulfil the majority of these aspects, it would very likely be classified as an aesthetically pleasing design.

2.2. Aesthetics of Kylesku Bridge

The simplicity of the bridge lends itself well for the fulfilment of its function, as it is immediately clear how the bridge works structurally. The thick deck also helps with this, despite looking slightly overdesigned, as it installs faith in the bridge for the users. However, due to the thin fascia along the edge of the deck, the deck still looks in proportion as a lot of the deck is in shadow, drawing the eye away from most of its depth. Because of this, the slender columns also look in proportion with the deck. The columns are also tapered at the top, which gives the impression that the columns are actually a lot longer than they actually are. However, the joints between the columns and the deck have been done very poorly, most likely as an afterthought once the rest of the bridge was designed, resulting them sticking out from the side of the deck which unfortunately completely breaks the order of the bridge.

It was decided that the bridge would not be painted at all, but instead to leave it as was with the standard concrete grey colour. Whilst this may sound as if it might make the bridge dull to look at, it instead ensures that the bridge blends in well with the gneiss bedrock which is visible along the water's edge. This greatly helps the bridge to become integrated with the surrounding environment, and therefore offsetting the initial dullness of the colour. However, due to the harsh weather of the Scottish Highlands, there has been some slight weathering of the concrete, and while this does make it look slightly more natural in its surroundings, this is now offset by the poor surface texture resulting from it.

Part of the beauty of Kylesku Bridge is its simplicity to behold, but it does also have a slight amount of complexity to visually stimulate the beholder. This complexity is clear when observing the inclined columns, which from most viewpoints cross over one another. The two designated viewing areas on each side of the bridge have been located so that this feature is displayed.

Figure 3: Photo showing the supports blending into the gneiss bedrock, the crossing of the supports, the joints from the supports to the deck, the thickness of the deck and the weathering of the concrete

2.3. Summary of Aesthetics

By fulfilling 6 of the 10 rules (see chapter 2.1), Kylesku Bridge can be said to be a success from in this aspect, but it is also let down in 2 areas. However, when it was first built, weathering would not have been an issue, making the surface texture of the bridge also a success. Therefore, it was deserving of the award it received when it was new, but poor maintenance has distorted this.

3. Foundations and Geotechnics

The area of Loch a' Chàirn Bhàin and Kylesku Bridge sits on a layer of gneiss bedrock, which is at quite a shallow depth below the topsoil. Gneiss is a metamorphic rock that has been formed from sedimentary or igneous rocks by regional high temperatures and pressures. It is generally a grey or pink rock (in this case grey) with lighter and darker streaks and layers. The grains range from medium to coarse, but the texture is dominated by the discontinuous layers of minerals. The lighter bands consist of minerals like feldspar and quartz, while the darker streaks have minerals like muscovite, biotite and hornblende present. There would also be several other minerals that depend of the specific regional metamorphism.

The advantage of gneiss over other metamorphic rocks is that less than 50% of the minerals are aligned into thin foliated layers, as opposed to rocks like schist which is much more strongly aligned. Because of this, gneiss doesn't fracture as much along those planes of mineral streaks, making it a more durable bedrock to sink foundations into.

The foundations used are most likely piles sunk deep into the bedrock in order to make the most use of the natural strength of gneiss. There is also not enough space for shallower pad foundations to be properly utilised due to the proximity of the water. It is the most likely scenario that there are four piles, two on each side of the

Loch, underneath where the inclined support meet at ground level.

Figure 4: Gneiss rock

4. Construction

The bridge was designed to be able to be constructed in four key stages which are as follows:

1. Constructing inclined support columns and the abutments on both side of the loch
2. Incrementally launch end spans from both sides, over the first inclined support columns
3. Continue the incremental, cantilevering past the second columns on both sides
4. Construct the central part of the deck, between the two cantilevered ends

4.1. Stage 1

The first stage of construction was to make the eight inclined columns that comprise the support legs for the deck. These columns have a width and breadth of about 3m each, but tapering slightly at the top, and are 20m long. They are constructed in groups of four, with one group on each side of the loch. They are arranged so that the bases of two of the columns in the group meet in the middle on each side of the deck, forming two V shapes. The two V supports are also inclined towards the deck, and so also meet in the middle beneath the deck. Temporary restraints are also put in place at this stage to help with the construction of the inclined supports, and to add some extra support for construction loading. The supports are made from $1393m^3$ of reinforced concrete. At the same time that the supports were being constructed, the abutments at each end of the bridge were also constructed, each consisting of $278.5m^3$ of reinforced concrete.

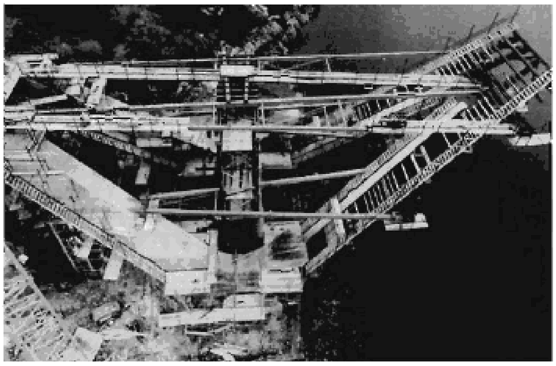

Figure 5: Photo of the construction of stage 1 (Nissan et al., 1985)

4.2. Stage 2

The next stage was to start the construction of the bridge deck. It was built using the incremental launching method, and the evidence of this is apparent if the underside of the deck is observed as there are clear lines showing the different segments that have pushed out. All of the launching segments were cast in-situ, which negates the issue of transporting large concrete segments to the remote site, and as the bridge is fairly close to ground level, minimal pumping was required to cast it there instead. Because of the remoteness of the site, incremental launching also brought other advantages, such as not needing to get a crane into the area and not needing to bring any falsework. It also means that there was plenty of space behind the abutments, which is necessary for the casting and launching equipment. However, as the bridge does not have a constant radius along its entire length, this process had to be done from both abutments, as there is a different constant radius on both sides. Both sides were initially launched out slightly further than the first set of inclined supports, so that there was a slight cantilever beyond them.

Because of the relatively short spans along the length of the bridge, it is unlikely that the launched segments would have missed their bearings. Despite this, constant surveying was undertaken just in case, and any adjustments were then made to the formwork to keep them inside the tolerances.

Large amounts of prestressing were needed for the construction process to deal with the amounts of sagging and hogging moments that each segment was put under, which was much higher than they would have been for other methods and during the rest of the bridges life.

Figure 6: Photo of the underside of the bridge deck, showing evidence of incremental launching

Figure 7: Photo of the construction of stage 2 (Nissan et al., 1985)

4.3. Stage 3

The next stage of construction was much the same as the second stage, as it was this stage that saw the continuation of the incremental launching to reach the second sets of inclined supports from both sides. The only real different was that the launching was continued for 18m past the inclined supports instead of only slightly cantilevering past as before.

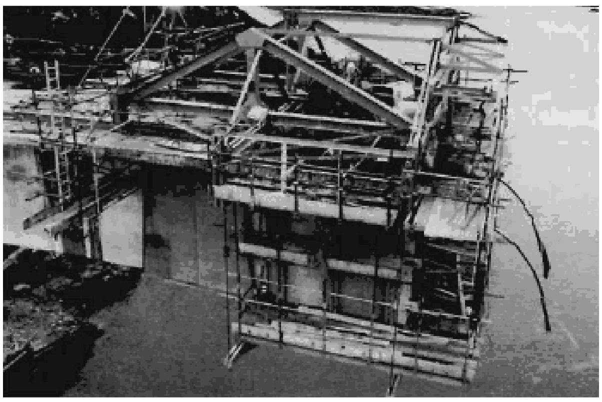

Figure 8: Photo of the construction of stage 3 (Nissan et al., 1985)

4.4. Stage 4

The final stage of construction was to link together the two sides the incrementally launched sections of the bridge. The section for this was a 43m long piece that was precast in a factory and transported on site. This was necessary as the existing formwork for casting the segments for the incremental launching would not have been large enough for create a beam of this size. This beam was then lifted into place from the back of a boat, and an in-situ concrete slab was then cast on top. By doing it in this way, no falsework was needed to be put up, which would have had to been transported in especially for this section, as none was needed for the incremental launching. The temporary restraints put up during stage 1 can were then removed before the continuity cables between the new central section and the existing sections on either side were stressed.

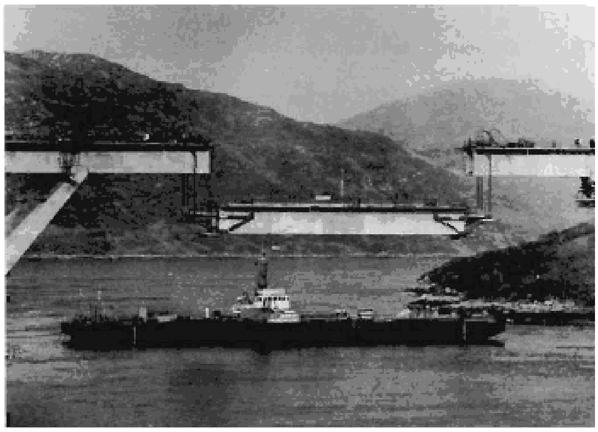

Figure 9: Photo of the construction of stage 4 (Nissan et al., 1985)

5. Loading

Kylesku Bridge can be simplified for loading as a multi-span portal frame. As such, there are different worse case scenarios for the maximum sagging and maximum hogging moments. In both cases, the dead loads and superimposed dead loads can be assumed to be constants over the length of the bridge, but the position of live loads can vary. Therefore, the maximum sagging moment is likely to occur when there is high loading on the longest span, which is the middle span, but none on the adjacent spans. Maximum hogging will be when there is high loading on two adjacent spans, such as the middle span and one to either side, and no loading the others.

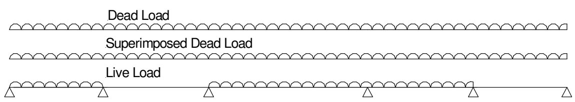

Figure 10: Loading diagram for maximum hogging moment

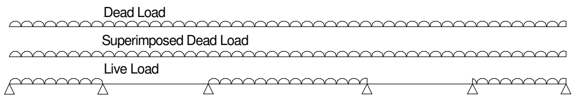

Figure 11: Loading diagram for maximum sagging moment

5.1. Dead Loads

A bridge's dead load is the load generated by the weight of the structural members. Reinforced concrete has a density of 2400kg/m³, and so it can be assumed that this value can also be taken for prestressed concrete. Based on this, the dead weight of the deck was found to be 160.98kN/m length and the supports were each 204.98kN/m height. This difference in weight is due to the deck being a hollow box section, whereas the supports are solid sections.

5.2. Superimposed Dead Loads

Where the dead loading deals with all loading on the bridge that is structural and therefore permanent, superimposed dead loading deals with other parts of the bridge that are not structural and therefore could be removed or added to over the bridge's design life. The majority of this loading comprises of the fill material and surfacing applied to the bridge deck. Assuming 300mm of saturated fill material and 100mm of bitumen surfacing, this comes out at 5.87kN/m² and 2.35kN/m² respectively. Added on to this is an assumed value of 1kN/m² for any loading brought on by road furniture or services. Therefore, the total superimposed dead load comes to 9.22kN/m².

5.3. Live Loads

The live loading on a bridge needs to be further subcategorised into normal loads (HA) and abnormal and exceptional loads (HB). HA loading is a uniform load acting over a single notional line, combined with a knife edge load (KEL) which is positioned at any point in the notional lane to produce the most adverse affect. For bridge up to 285m long, the load can be assumed to be 10.3kN/m length. This is then divided by the width of the notional lane to get the nominal uniformly distributed load (UDL). The thinnest breadth of Kylesku Bridge is 8.2m, and so this is the breadth to use assigning the number of notional lanes, which is 3 for bridge between 7.6m and 11.4m wide. Therefore, the width of each notional lane is 2.73m, making the UDL for a single notional lane 3.77kN/m². This load is then fully applied to any 2 notional lanes in order to get the worst case scenario, which in this case will be either of the outer lanes as well as the middle lane. A third of the full HA load must also be applied to any remaining lanes, of which is just the opposite outer lane for this bridge. The KEL is taken to be 120kN for each notional lane.

HB loading is not taken as a UDL like HA loading is, because of the fact that it is supposed to be a rarer occurrence. A full HB load is taken to be 112.5kN per wheel, assuming that each axle has 4 wheels (making each axle have a load of 450kN). Because the notional lanes on Kylesku Bridge are only 2.73m wide, this means that a HB load must straddle 2 of them, which must therefore be the same outside notional lane as the full HA load was taken in, as well as the middle lane again. It is also taken as standard that the there is a 25m gap in front of and behind a HB load.

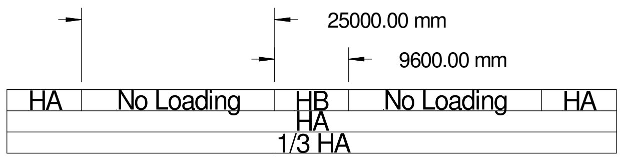

Figure 12: Loading diagram for HA and HB loads on the bridge deck

5.4. Wind Loads

The Scottish Highlands and subjected to very high wind speeds throughout the year, meaning that wind loading on the bridge is a key issue, having one of the highest in the whole British Isles. This is mainly caused by the proximity to the sea that this site in particular has. As explored earlier, the bridge also has a very deep deck, which will increase the contact area that the wind has with the bridge, which will therefore increase the effect that it has.

The first step in calculating the wind loading is to calculate the maximum wind gust (v_C). This is based on 4 factors; the mean hourly wind speed (v), the wind coefficient (K_1), the funnelling factor (S_1) and the gust factor (S_2). They are related as such:

$$v_C = vK_1S_1S_2. \tag{1}$$

The mean hourly wind speed for the local area is 35m/s. The wind coefficient, which is based on the bridge's height above the ground level and the length of the bridge, is 1.48. Because the area around the bridge is relatively flat, the funnelling factor is assumed to be 1.00, and the gust factor, which is also based on how high the bridge is above the ground, is 1.13. This means that the maximum wind gust is 52.68m/s.

The next step is to calculate the horizontal wind load that the bridge is subjected to. The equation for this is:

$$P_t = qA_1C_D. \tag{2}$$

where q is the dynamic pressure head, and is calculated from the maximum wind gust:

$$q = \frac{\rho_{air} v_C{}^2}{2}. \tag{3}$$

where ρ_{air} 1.226N/m³. This comes out with q equalling 1701.19N/m². A_1 is the solid horizontal projected area, which in this case is 1177m², and C_D is the drag coefficient, which based on the breadth to depth ratio, and was found to be 1.5. Therefore, P_t came out to be 3MN. This wind load is assumed to be acting on the centroid of the horizontal plane from which A_1 was calculated.

The longitudinal wind loads also need to be taken into account, and can be split into two different effects; those acting on the live loads (vehicles) (P_{LL}) and those acting on the structure (P_{LS}). The equations for both of these loadings are as follows:

$$P_{LL} = 0.25qA_1C_D. \quad (4)$$

$$P_{LS} = 0.5qA_1C_D. \quad (5)$$

In both of these equations, q and A_1 are the same as for the horizontal loading, but the drag coefficient changes to 1.45. However, neither of these equations is particularly accurate, but they are best available for a simple analysis. Using these equations P_{LL} was calculated to be 0.73MN and P_{LS} was calculated to be 1.46MN.

The final piece of wind loading that has to be considered is the vertical loading (P_V), in this case uplift as the bridge deck cross section is very roughly aerofoil in shape, creating a difference in pressure from the top side and bottom side. The equation for this is:

$$P_V = qA_3C_L. \quad (6)$$

where q is the same as for the previous loadings, A_1 is the plan area of the bridge, which is 2423.75m², and C_L is the lift coefficient, which is normally based of the breadth to depth ratio unless the superelevation is between 1° and 5°, which it is in this case, making it 0.75 as an assumed standard. This makes P_V come out as 3.09MN.

Once all the wind loads have been calculated, the worst combination of them must be found from the following list:

1. P_t alone
2. P_t in combination with P_V
3. P_{LS} alone
4. $0.5P_t$ in combination with $P_{LS} + P_V$

Of these options, number 2 was found to give the highest result at 6.09MN.

5.5. Temperature Effects

Any temperature differences in the bridge can cause the bridge to expand or contract, even if it's only by a small amount. However, even if the expansion or contraction is small, it must still be taken into account as considerable stresses can still be built up. The extreme air temperature around Kylesku Bridge ranges from a minimum of -14°C and a maximum of 29°C. The actual minimum temperature is increased to -9°C because of the bridge being made from concrete and the insulating effect of the surfacing. The maximum temperature is the same at 29°C, as the concrete has the effect of raising it, but the surfacing insulates it to bring it back down. The maximum possible temperature difference is therefore 37°C.

$$\varepsilon_T = \alpha\Delta T, \quad (7)$$

where α is the coefficient of expansion for concrete, which is $12 \times 10^{-6}/°C$. This means that the bridge could build up strains of 444με from expanding or contracting. The expansion or contraction of the deck (ΔL) can then be calculated using the following:

$$\Delta L = \varepsilon_T L. \quad (8)$$

From this equation, the change is length comes out to be 123mm.

Normally, this change in length would be allowed for because of the expansion joints. However, it could be the case that these joints become blocked or damaged, and so unable to allow the bridge to move. This would then mean that high stresses will build up in the bridge equal to:

$$\sigma_C = \varepsilon_T E, \quad (9)$$

where E is the Young's Modulas of concrete, which is 30GPa. This makes the possible stresses equal to 13.32N/mm².

5.6. Loads Due to Curvature

As traffic drives round a curve on any road, centrifugal forces will be built up. Whilst on normal roads this isn't an issue, on a curved road bridge, such as Kylesku Bridge, this generates extra loading on the bridge members, and so must be calculated and designed for. The worst case loading for this is when all the lanes of the road are occupied by moving traffic, and the resulting load is calculated with the following equation:

$$C = {wv^2}/{12.7r} \quad (10)$$

where C is the centrifugal load generated in kN/m length, w is the distributed live load in kN/m length, v is the maximum speed in km/hour and r is the radius of curvature in m (Kumar and Kumar). Using the live load of 5kN/m² and the average breadth of 8.75m, this gets a distributed live load of 43.75kN/m length. The speed used will be the speed limit for single carriageway roads in non-urban areas of 60mph, which converts of 96.56 km/hour. The bridge has two different constant radii along its length, as shown below in figure 13.

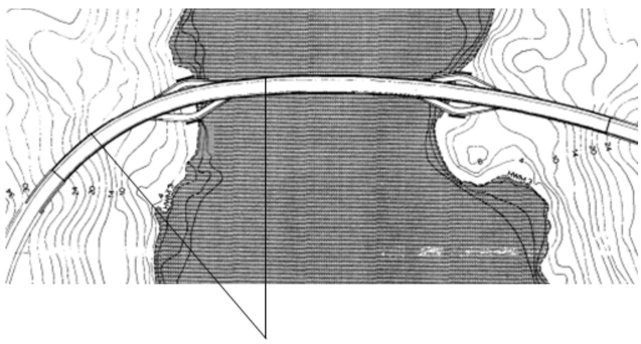

Figure 13: Plan view of Kylesku Bridge showing the smaller of the two radii (Nissan et al., 1985)

By using a to scale plan view of Kylesku Bridge, the two radii were calculated to be 189.3m and 646.9m. Therefore, the different loads on the two curved sections of the bridge are 169.68kN/m length and 49.65kN/m length respectively. However, if the bridge is simply designed to withstand the extra 169.68kN/m length it will be more than adequate the rest of the bridge as well.

5.7. Snow Loads

There is often high snow fall in the Scottish Highlands during winter, therefore snow loads also need to be taken into account. However, it can be assumed that these are simply a nominal load of 0.5kN/m².

6. Strength

The most critical period for the strength of the deck was during the launching process, as during this the deck would have been subjected to hogging and sagging moments much higher than it would normally be subjected to. It is therefore important to determine if the prestressing tendons are effective enough to prevent the deck from exceeding 1N/mm² of tensile stress during launching.

The prestressing tendons are made up from 19 15.2mm Dyform strands, with a tensile strength of 1820N/mm². Each strand has an area of 165mm, so the peak load for each is 300kN (Bridon); therefore each tendon has a strength of 5700kN. These tendons are arranged so that there are 12 in the top flange of the box section and 6 in the bottom. The area of the box section is 6.83m² and the second moment of area is 14×10^{12}. The neutral axis is at 0.45d.

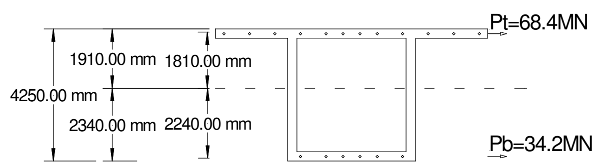

Figure 14: Simplified box section with prestressing tendons

Using the following equations, the stress state of the section can be calculated without the added moments from launching using the following equations:

$$\frac{P}{A} = \frac{102.6\times10^6}{6.83\times10^6} = 15.02\text{N/mm}^2, \qquad (11)$$

$$\frac{P_T e_T y_T}{I} = \frac{68.4\times10^6\times1810\times1910}{14.6\times10^{12}} = 16.20\text{N/mm}^2, \quad (12)$$

$$\frac{P_T e_T y_B}{I} = -19.84\text{N/mm}^2, \qquad (13)$$

$$\frac{P_B e_B y_T}{I} = -10.02\text{N/mm}^2, \qquad (14)$$

$$\frac{P_B e_B y_B}{I} = 12.28/\text{mm}^2. \qquad (15)$$

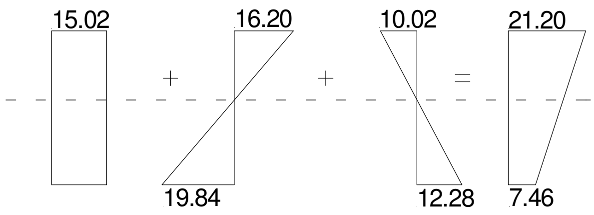

Figure 15: Stress state for box section before hogging and sagging moments applied

The worse case hogging moments will occur during the launching of stage 3, where the deck is being constructed between the inclined supports on each side, where the span is 52.5m, just before the nose reaches the next inclined supports. It will not be over the main span of 79m because this will only be launched 18m from sides (see chapter 4.3). Using just the dead weight of the deck (as there would not be any superimposed dead load from surfacing, fill or roadside furniture, or any live loads from vehicles on the deck at this point), and assuming that the nose is 10m long and has a negligible weight, the maximum moment here is:

$$M_{Max\ hogging} = \frac{wl^2}{2} = \frac{160.98\times42.5^2}{2}, \qquad (16)$$
$$= 145.39MNm$$

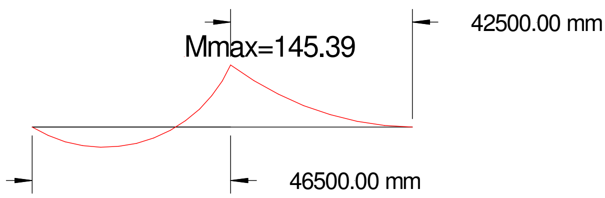

Figure 16: Bending moment diagram for maximum hogging moment

The worse case sagging moments will also be on this span during launching, but will instead be when the deck has reached the second inclined supports.

$$M_{Max\ sagging} = \frac{wl^2}{10} = \frac{160.98\times52.5^2}{2}, \qquad (17)$$
$$= 45.06MNm$$

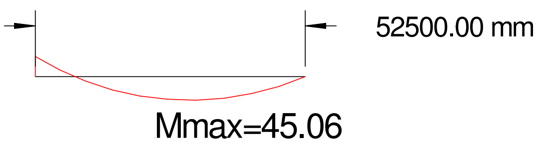

Figure 17: Bending moment diagram for maximum sagging moment

The stresses generated from these moment can now be added to the original stresses in the box section to see if the tensile stress is low enough, and also if the compressive stress is not too high.

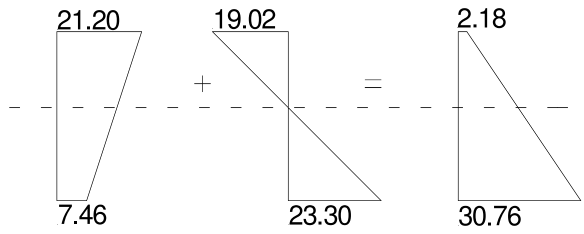

Figure 18: Stress state for box section with worse case hogging moments

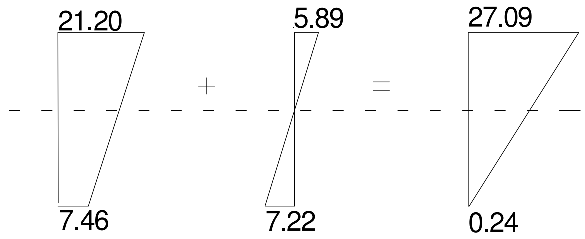

Figure 19: Stress state for box section with worse case sagging moments

As the tensile stress doesn't exceed 1N/mm², and the compressive stress doesn't exceed the f_{ck} of concrete, the prestressing regime is acceptable.

7. Serviceability

The main serviceability issues with concrete structures of all kinds are to do with cracking from creep or shrinkage, settlement and volume changes brought on by varying temperatures. As previously explored, temperature changes could pose a potential problem for Kylesku Bridge in particular, and so checks must be made regularly to ensure that the bearings are not blocked or broken (see chapter 5.5). Settlement, however, is less likely to cause any issues with this bridge because it is built directly on top of the gneiss bedrock (see chapter 3).

7.1. Creep

Creep is a problem for any concrete structure, as it is brought on by the setting of the concrete which continues for the entire life of the structure (although most of it occurs in the first year after construction). It is therefore critical that the bridge is monitored throughout its life so that if creep does become a massive problem, some maintenance can be performed to lessen the load.

7.2. Natural Frequency

Concrete beam bridges are not as susceptible to vibrations as other bridges are, such as suspension or cable stayed bridges, making this less of an issue. The fact that it is primarily a road bridge and very few pedestrians will be crossing it, lessens the issue of vibration comfort even further. However, it is still worth a check to see if the bridge is within the 5-75Hz acceptable range using the Rayleigh-Ritz equation:

$$w_n = (\beta_n l)^2 \sqrt{EI/_{ml^4}} \qquad (18)$$

where $(\beta_n l)^2$ equals 15.42 for a clamped to pin beam or 22.37 for a clamp to clamp beam, EI is the deck stiffness in vertical bending, m is the mass per unit length in kg/m and l is the span length. For the 79m main span, this comes out as 10.42Hz for when its clamp to pin, and 15.36Hz in case damage reverts it to being a clamp to clamp. In both cases this is within the acceptable range and so is unlikely to cause any foreseeable issues.

8. Durability

It is inevitable that during the lifespan of any bridge, issues can arise that adversely the effect the quality, leading to a reduction in longevity. The majority of these issues are not ones that can be avoided, but some can be designed against to begin with, if the designers show the initiative to do so.

8.1. Chemical Attack

Snowfall is often quite high in the Scottish Highlands, and so de-icing salts are often necessary to make the roadways usable during winter. The problem with this is that this can lead to chloride ingress on the reinforced concrete structure of bridges. For this to occur, there has to be at least a moderate level of humidity, because the chloride ions have to be dissolved in water in order to ingress into concrete, which is done by a diffusion process driven by differing concentrations. As the Scottish Highlands is a fairly moist area, this condition is likely to be met on a regular basis. However, the bridge is not totally immersed or cyclic wet and dry, and so the class is simply XD1. Other factors that can affect the rate of chloride ingress are if the concrete has a high cement content, a low water to cement ratio, or if blended cements are used. However, the exact mix proportions for Kylesku Bridge are unavailable, meaning it is not possible to comment on these additional factors.

Chloride ingress does not directly attack the concrete of the structure, but instead it inhibits the passive oxide protection of any steel reinforcement (Polder et al, 2005). The attacks are localised and lead to pitting of individual areas of steel reinforcement, usually in areas of existing weaknesses such as rust. However, if there is a nearby site with an even higher level of chloride, this can act as an anode to protect the region of study, as well as others around it. Because of this, it is very difficult to determine when and if chloride induce corrosion will actually take place, and to what degree, so the threshold limit used in design is instead regarded as the risk of corrosion taking place. This is expressed as a ratio of chloride to cement, typically being between 0.5% and 2.5% of the overall mass.

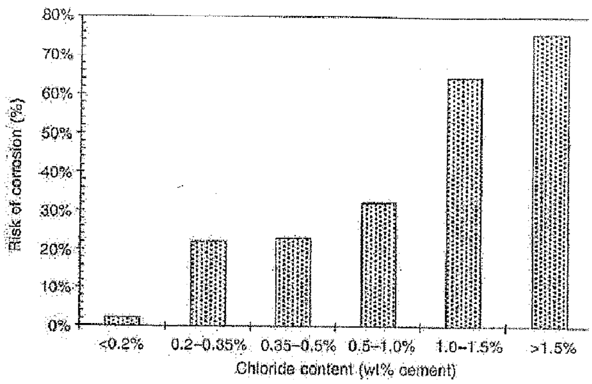

Figure 20: Risk of corrosion as a function of chloride content (Vassie, 1984)

Another possible form of chemical attack is sulphate attack, which could be brought on in the inclined supports as they are exposed to salt and water spray. However, the designers have compensated for this by using sulphate resisting cement in the supports, thereby minimising the effect of this form of chemical attack.

8.2. Accidental Damage and Vandalism

The level of accidental damage is likely to be kept to a very small amount as the bridge crosses over a little used waterway, and the few boats that do pass underneath are small and slow moving. This effectively negates any issues from vehicles colliding with the supports. The only other accidental damage that could occur would be from vehicles colliding with the parapets, but this would not cause major structural damage as the parapets are not structurally tied in with the rest of the bridge. All that would be needed to be done would be the replace the area of parapet where the vehicle collided.

Acts of vandalism are also small due to the remote location of the bridge, meaning very few people live in the direct vicinity of Kylesku Bridge. It is also a very unlikely target for any planned assault on the country, again because of its remote location and small usage in comparison to other bridges.

8.3. Loss of Prestress

As the deck is loaded goes through various loading conditions, the prestressing tendons running through it often lose some of their effectiveness. This loading affects include the creep of the concrete, stress relaxation of the tendons, as well as many other minor effects. The only way to design for this is to ensure that the prestressing tendons can be easily reached for maintenance and to make sure that if one of the tendons breaks or is removed that there are still enough to provide the bridge with enough reinforcement to stay standing.

8.4. Climate Change

There are two key issues with the durability of the bridge to do with climate change. The first is the rising sea levels, which will flood a lot of the area around Kylesku Bridge as the Scottish Lochs are essentially part of the sea as they are salt water. This may result in the supports being partially submerged in the salt water, leading to a greater chance of chemical attack (see chapter 8.1).

The other issue that could be raised because of climate change would be the increased wind loading on the bridge, as it is expected for there to be an increased frequency of powerful storm winds.

9. Future Changes

The majority of bridges in the world will need some modification or treatment several years after they have been constructed in order to evolve to deal with issues that were not foreseen in the design, or issues that were foreseen but couldn't be dealt with in advance.

9.1. Resurfacing

This is the least critical change that is going to need to be performed, but it is almost certainly going to be needed to be done. It is also the easiest change to make, as it is a simple matter to just remove the top 50mm of surfacing and relay 100mm on top of it. This future change has already been accounted for in the generous amount of surfacing assigned when calculating the superimposed dead load (see chapter 5.2).

9.2. Reaction to Climate Change

There are two major issues raised because of climate change, higher winds and possible flooding (see chapter 8.4). It may therefore be necessary to modify the bridge in order for it to be able to cope with these changes. One modification that could be done to minimise the impact that the high wind speeds would have the bridge is make the deck more aerodynamic. This could be achieved quite simply by attaching some carbon fibre to the sides of the deck to make the deck into an aerofoil shape. This would make the wind flow a lot easier around the deck, meaning that the load on the deck would be smaller. Doing this would also add on minimal extra weight to the deck because carbon fibre is a lightweight building material.

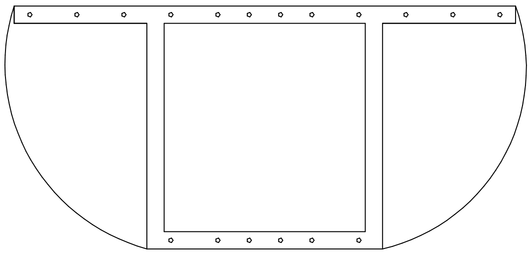

Figure 21: Box section modified for high winds

Clearly this isn't a perfect aerofoil shape, and it would be very hard to produce one as a modification to an existing bridge, but it would reduce the wind loads from either direction to less than they would be without it, and with proper research in a wind tunnel the optimum design could be found.

If climate changes leads to the area around Kylesku Bridge flooding, but not enough to completely decommission the bridge, the bridge will have to adapted to cope with its supports being partially submerged in salt water. This is the worst possible situation for chemical attack (see chapter 8.1) as there would be areas of the concrete that would be cyclic wet and dry. The supports are already made from sulphate resisting cement, but some further surface treatment may be needed to further protect the supports, even if it's just as simple as painting them. If a surface treatment is not enough, the supports may need to be clad in something else, such as carbon fibre.

9.3. Increased usage

If the use of Kylesku Bridge dramatically increases within its life enough so that splitting the existing two lanes into the three notional lanes available isn't enough to hold it, the bridge may have to be widened. This will not be a simple matter like it is when resurfacing, but there are two possible methods to do it. The first would be to extend the top flanges of the original concrete box section on both sides. If this method is done it is important that the extension is done on both sides so that the forces on the bridge remain in balance as much as possible. Depending on the extra forces on these flange extensions, the bottom flange and webs may also need to be extended outward to support them. The second option would be to effectively build a second bridge alongside the existing one that they are touching, but not actually structurally dependent on one another. This would be the method of choice if the demand for the bridge increases massively, as the new bridge can be built as large as needed to cope with it.

The other issue that may arise from increased usage is that the loads on the bridge would increase (especially if the flanges are extended). This could lead to the bridge needing to be strengthened. One way of doing this would be to add some more steel reinforcement, which could be done relatively easily because the deck is a hollow box section. Another option, which is becoming more popular in recent years, is to clad the critical areas of the bridge, such as the supports, in carbon fibre.

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