**Introduction**

Nordhordland Bridge (Nordhordlandsbrua) is a bridge that crosses Salhusfjorden from the mainland to Flatøy in Hordaland County in Norway. The bridge connects the northern parts of Hordaland (Nordhordland) to the city of Bergen. Nordhordland Bridge is a pontoon and cable-stayed bridge with one tower. The bridge is 1614 metres long, the longest span is 172 metres, and the maximum clearance to the sea is 32 metres. The cable-stay part of the bridge has 19 spans. The free-floating bridge has the longest laterally-unsupported span in the world.

Nordhordland Bridge was opened in 1994. It was the second permanent pontoon bridge in Norway.

The Design of the bridge was undertaken by a Norwegian company called Aas-Jakobsen and in particular by a designer named Per Meass.

The cable stayed portion of the bridge contains the longest span of 172m across a ships channel of 32 x 50 metres, and is made of High Strength Light Weight Aggregate concrete. The H-shaped pylon is anchored onto rock on land and the back spans, of total length 190 metres, stabilize the main span.
Aesthetics

The Nordhordland Bridge displays its structure well, and feels stable to both the trained and untrained eye. From its slender deck over the cable stayed portion, with its cables arranged in the ‘harp’ system and clearly linking to structure of the deck, as can be seen below, to the deeper, long pontoon portion, with the volume of the lightweight concrete pontoons giving an assurance of stability, the bridge looks, and feels, safe.

Over the cable-stayed region of the bridge, the thick cables and massive concrete tower look proportional compared to the width and length of deck that they are supporting. I do feel, however, that the steel support under the transition region, from cable stayed to pontoon bridge does appear thin.

Although the obviousness of the link between the cables and the horizontal members underneath the deck does well to describe the fulfilment of function, I feel that it does somewhat detract from the order of the bridge as a whole. These cable anchors break up the smooth line of the deck, however, the constant deck depth, and smooth curvature from the cable stayed region to the pontoon bridge do enhance the appearance.

There are a couple of subtle refinements in the design of the Nordhorland Bridge. The columns going up to make the tower are slightly tapered to make sure they don’t seem wider at the top.

I feel that the Nordhordland Bridge integrates very well into its environment; it is built in a fairly rural area of Norway and therefore it was important to maintain that lightweight, unimposing feel. Also, the fact that it is a pontoon helps to maintain the unobtrusive nature, as it seems less of a fixed structure and more of a creation in tone with the sea. Also, the areas it is connecting are small islands, and the bridge seems to be floating along with them, a clever aesthetic ploy.

There are a number of different textures and colours that make up the Nordhorland Bridge, firstly the stocky, lightweight concrete, then the smooth pure white steel underneath the bridge and the dark, yet smooth steel of the cables. The bright white of the steel stands out and shows the curvature of the bridge vividly, even being viewed from some distance. The brightness of the supports under the transition section from cable stayed to pontoon bridge I feel seems overstated, and would be more appealing if that section had been better camouflaged. The dark cables do hide themselves well against the surroundings, giving the appearance of their absence.
The Nordhordland Bridge I fell, definitely possesses character. The subtle downward curve of the seaward side of the cable stayed portion helps to make it appear that the pontoon part is anchoring the cable stayed portion and that the cable stayed portion also helps to provide an upward force on the pontoon, aiding buoyancy.

Similarly to what’s been said to do with integration into the environment, this bridge does appear natural, with the shape of the pontoons giving the feel of little islands supporting the bridge. For such a long bridge, it is extremely important to make sure it doesn’t disrupt the nature of the region, and the Nordhordland Bridge does this fairly well.

The Nordhordland Bridge fulfils its function simply and is not too difficult to understand structurally. Its aesthetic appeal is fairly obvious and it fits well into its surroundings. Although I dislike small sections of the bridge, specifically the area underneath the transition section from cable-stayed to pontoon bridge, I feel that overall, and this is a beautiful and well engineered bridge, simple and cost-effective.

**Construction**

Cable-stayed bridges are very popular during construction as the stays can be used during the construction in place of special equipment.

The contract for the construction of the Nordhordland Bridge lasted from 1991 to 1994, and the bridge was finally completed in 1994.

### 1. Materials Used

Box girder of normalised steel, typical yield strength of 355 MPa apart from 540 MPa in the critical exterior span.

High strength LWA-concrete in the main span; LC55 (NS3473)

Stays made of 7mm galvanised steel wires in HDPE pipe filled with grease and with HIAM anchors.

### 2. Bridge Geometry

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Length of Bridge between abutments</td>
<td>1246m</td>
</tr>
<tr>
<td>Length of ramp structure</td>
<td>350m</td>
</tr>
<tr>
<td>Typical span between pontoons</td>
<td>113.25m</td>
</tr>
<tr>
<td>Deck width</td>
<td>10.0m</td>
</tr>
<tr>
<td>Deck clearance above water level (min)</td>
<td>11.0m</td>
</tr>
<tr>
<td>Air draft below steel box</td>
<td>5.5m</td>
</tr>
<tr>
<td>Pontoon: L/B/H</td>
<td>42.0/20.5/7.38m</td>
</tr>
<tr>
<td>Pontoon Draft</td>
<td>4.3m typical, 5.6m</td>
</tr>
</tbody>
</table>
3. **Major Design considerations**

The most dominant force acting on the pontoon part of the bridge is the wave action, providing more than 50% of the total internal girder design force. The forces of the waves are proportional to the submerged volume of the pontoons.

There were two alternative options for the bridge girder; steel box and pre-stressed concrete box. Pre-stressed concrete box required a larger pontoon, more than twice the volume, and although the cost of this would still have been similar to the steel box option, the steel was selected due to a more favourable construction schedule.

The pontoons consist of 9 separate compartments, two of which can be completely filled with water without significant damage to the girder. The design of the pontoons is governed by hydrostatic pressure, and the use of LWA-concrete significantly reduced the cost of the project. The LWA concrete is made of Portland cement, Silica fume, Sand and expanded clay.

4. **Deck Construction**

The pontoons themselves were built first, and the deck constructed over the water. A raft is used as a casting girder in this case, and the travelling formwork method was used.

For the cable-stayed region, the deck was hung in place piece by piece as the individual stays were put in, a method known as suspended cantilever construction. The cable-stayed section of the bridge was constructed entirely out of LWA concrete.
The bridge deck is composed of mostly steel, whilst the pontoons are lightweight concrete. The bridge is a free-floating, end-anchored, hollow box section pontoon bridge. The total length of the deck above the pontoon part of the bridge is 1246 metres, and the span between pontoons is 113.25 metres. The deck itself is 10 metres wide and the minimum clearance above water level is 11 metres. From the image above it appears as if the bridge deck is fixed to the main pier, which would cause large bending moments in the pier.

**Loading**

For loading analysis, I have just analysed the cable-stayed region of the bridge as it is effectively a bridge in itself. There are twelve stays supporting this span, spaced 11m apart from each other, and with 13.25m clearance to the main pylon and 17.75m to the start of the pontoon portion.

*Elevation of cable-stayed bridge:*

1. **Dead Load**

**Partial Load Safety Factors for Concrete**

<table>
<thead>
<tr>
<th>( \gamma fi )</th>
<th>ULS 1.15</th>
<th>SLS 1.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma f3 )</td>
<td>ULS 1.10</td>
<td>SLS 1.00</td>
</tr>
</tbody>
</table>

Total quantity of concrete over 160.0m span = 1140m\(^3\) (from AAJ)
Density of LWAC = 1900kg/m\(^3\)
Therefore, Dead load of cable stayed bridge = 135.4kN/m

2. **Superimposed Dead Load**

Services such as lighting and parapets have been kept to a minimum. After making a few assumptions as to the size of lampposts and parapets, the I have achieved an unfactored value for Superimposed Dead Load of 12kN/m

**Safety Factors for Superimposed Dead Load**

<table>
<thead>
<tr>
<th>( \gamma fi )</th>
<th>ULS 1.75</th>
<th>SLS 1.20</th>
</tr>
</thead>
</table>
3. **Live Loads**

The carriageway width, as can be seen from the cross-section above, is 12m. This can be divided into 4 notional lanes, each of width 3m. The two different types of traffic loading, HA and HB loading, need to be applied over the carriageway area of the bridge.

Seeing as the span is 172m, this gives a value of HA loading at 13.2 kN/m, dividing this by the lane width gives a value of 4.4 kN/m². This value is applied over two lanes and a third of this value applied over the remaining two lanes. The knife-edge loading of 120kN is applied in the same way, fully across two lanes and 1/3 across the other two.

With this loading state applied, a value of 37.1 kN/m is acquired for the critical span of this bridge.

HB loading is loading applied by an exceptionally heavy truck on the bridge; Each wheel of this truck represents a load of 112.5kN nominally. For this bridge the critical case is such a vehicle, with front and rear axles spaced at only 6m apart, at mid-span of the critical 172m span. This vehicle will be straddling two notional lanes on one side of the bridge for the critical case, and this will produce a value of 47.5 kN/m over the critical span, greater than the HA loading alone.

4. **Wind Loading**

I will assume a mean hourly wind speed of 32 m/s as this is a coastal region of southern Norway. The loaded length I am analysing is an 172m span, the wind height above ground level is 32m, and the parapet is open and has a height of 1.0m.

So \( v_c = v \times K_1 \times S_1 \times S_2 = 63.0 \)

a) For unloaded state

From cross-section above:

\[
A_1 = 1.142 \times 172.0 = 196.4 \text{m}^2
\]

\[
b = 15.1 \text{m}
\]

\[
d = 1.142 \text{m}
\]

Therefore: \( b/d = 13.2 \), so \( C_D = 1.0 \)

Thus, \( P_t = 478kN \)

\( P_v = 1580kN \)

And \( P_{LS} = 119kN \)

Note that \( P_v \) can be both a down force and an uplift.

b) With Live Load

\[
A_1 = 4.142 \times 172.0 = 712.4 \text{m}^2
\]

\[
b = 15.1 \text{m}
\]
Therefore: \( b/d = 13.2 \), so \( C_D = 1.0 \)

Thus, \( P_t = 1730 \text{kN} \)
\( P_v = 1580 \text{kN} \)

And \( P_{LL} = 805 \text{kN} \)

Thus, the uplift/down-force on this span is \( 9.18 \text{kN/m} \), and the horizontal wind load, for the critical case for this analysis is \( 10.1 \text{kN/m} \).

5. Temperature Effects

Assuming that the entire bridge cross section increases in temperature by 25 degrees Celsius, and that the co-efficient of thermal expansion = \( 12 \times 10^{-6}/\text{C} \), because the bridge deck is 362m in length overall, the expansion of the bridge deck (\( \delta \)) can be calculated with the following two equations;

\[
\begin{align*}
\varepsilon &= \alpha \times \Delta T \\
&= 12 \times 10^{-6} \times 25 \\
&= 300 \mu \varepsilon
\end{align*}
\]

\[
\begin{align*}
\delta &= \varepsilon \times l \\
&= 264 \times 10^{-6} \times 362 \\
&= 0.096 \text{m}
\end{align*}
\]

Therefore, the amount of movement at the expansion joint at the transition from cable-stayed to pontoon bridge will be 96mm. The longitudinal compressive stress which will be built up due to this wanted movement if the expansion joints clog is:

\[
\begin{align*}
\sigma &= \varepsilon \times E \\
&= 300 \times 10^{-6} \times 30,000 \\
&= 9 \text{ N/mm}^2
\end{align*}
\]

6. Loading Combinations For Ultimate Limit State

There are five loading combinations in total, for the case of this bridge I will be analysing the first two combinations, they are;

1. All permanent loads plus primary live loads
2. Combination 1 plus wind loads.

The nominal loads are now multiplied by the safety factors \( \gamma_{f1} \) and \( \gamma_{f3} \).

\[
\begin{align*}
\text{Combination 1;} \\
&= [135.4 \times 1.15 \times 1.10] + [12 \times 1.75 \times 1.10] + [47.5 \times 1.30 \times 1.10] \\
&= 262.3 \text{kN/m}
\end{align*}
\]

\[
\begin{align*}
\text{Combination 2;} \\
&= [135.4 \times 1.15 \times 1.10] + [12 \times 1.75 \times 1.10] + [9.18 \times 1.40 \times 1.10] + [47.5 \times 1.30 \times 1.10] \\
&= 330.2 \text{kN/m}
\end{align*}
\]

These are the critical cases for each load case, the wind can provide uplift on the deck; however that uplift will only lower the overall value of \( w \), and will not be a problem as the dead load of the deck far exceeds any uplift that may be achieved.

7. Other Loading Considerations

Being a concrete bridge, the deck and main pier would undergo Shrinkage and Creep, which would also need to be tested for. As the bridge has now been standing for over 10 years though, these effects should have stopped happening, and most likely stopped having a serious effect as early as when the bridge was opened. Differential settlement of supports could be another condition which would put the bridge under abnormal loading and earth pressure on the abutments, specifically on the landward side of the main pier would be important to consider. Snow loading can easily be considered over a bridge such
as this and is taken as a nominal 0.5 kN/m². In a climate such as the Norwegian one, it would be important to consider such loading. Also, scour may come into effect at some time in the future, as the foundations of the main pier, although on hard rock, are very near the water level and we are very aware of water levels rising due to the effects of global warming. Freeze-thaw effects could be significant given the latitude of the bridge as well. The nominal loads associated with all the above considerations would need to be found with the use of specialist literature.

Another loading condition which would be important to analyse is impact on the substructure, although the main pier of the bridge is not touching the water, there is still a possibility of boats colliding with the pontoon section of the bridge, and that could be a significant force.

**Structural Analysis**

For the analysis of the structure, load combination 2 has been used, as it is the critical load case. I have ignored the jelly effect in this case and have treated the stays as individual supports, thus the bending moment diagram achieved is:

The loading path is defined by the fact that the stays are all in tension and the main pier is in compression.

To analyse for maximum sagging, this critical load is applied to every other span, and only an unfactored dead load is applied over the intermediate spans.

To analyse for maximum hogging, the load is distributed over the bridge as shown below, with maximum load over the two spans where the critical hogging is being analysed, and max load at alternating spans outwards of that, again an unfactored dead load is applied to the other spans:

An approximate value for maximum bending moment in the bridge can be acquired by using the formula;

\[
M = \frac{wl^2}{8}
\]

\[
= \frac{330.2 \times 26.75^2}{8}
\]

\[
= 29,500 \text{ kNm}
\]

The vertical reactions applied by the cable supports can be calculated by assuming that each cable supports exactly half the span either side of it. For analysis of this, the critical state is used, where load combination 2 is applied over the entire bridge. As each of the stays are the same size, only the one likely to be under the greatest tension force need be analysed. In the case of this bridge, that critical stay is the final one at the end of the cable stayed section of the bridge before it joins the pontoon. This cable is not only supporting the largest span, but is also the shallowest cable, therefore providing the smallest proportion of its force as upthrust. From the elevation, I have assumed that this cable be inclined at 45° to the horizontal.

\[
T \sin 45 = 19.375 \times w
\]

so, \( T = 9050 \text{ kN} \)
The entire load on the main span section of the bridge is ultimately transferred into the main pier. There is evidently some eccentricity of loading about the central pier, thus it has been designed thick and strong, to be able to carry the bending moments this entails. In the horizontal direction, there is a wind load of 10.1kN/m, as calculated above, thus the bridge must put up with a bending moment of 18675kNm, in its lateral stiffness.

**Serviceability**

1. **Deflection**

The serviceability condition the bridge is in considers the deflection of the bridge under live loading. The critical span to consider is again the one closest to the pontoon section. I is assumed to be 0.5.

\[
\delta = \frac{5wl^4}{384EI} = \frac{[5 \times 47.5 \times 26.75^4]}{[384 \times 30 \times 10^6 \times 0.5]} = 0.021\text{m}
\]

This value is very small, thus we can deduce that dead load dominates the design criteria.

2. **Parapets**

Collisions occurring with the parapets are assumed to be 25 units of HB loading colliding with the parapet. The assumption made here is that a fully laden truck, weighing 40 tonnes, loses control and crashes into the parapet at an angle of 5 percent, with a velocity of 50 m/s. Assume the time that it is in contact with the parapet for is 0.1 seconds.

\[
F\Delta t = [0.2] \text{ m Av}
\]

\[
F \times 0.1 = 0.2 \times [40000] \times [50 \times 5/100]
\]

\[
F = 200 \text{ kN}
\]

Check \( v = u + at \)

\[
0 = 2.5 + a \times 0.1
\]

\[
a = -25 \text{ m/s}
\]

\[
s = ut + \frac{1}{2} at^2 = [2.5] \times [0.1] + \frac{1}{2} \times [-25] \times [0.1] = 0.125\text{m}
\]

Most of the force from the impact will be taken by the parapet longitudinally, this will cause a maximum latitudinal deflection of 0.125m as shown above. The parapet on the Nordhordland bridge appears to be more robust than one you may find on a similar bridge in the UK. This may be because the design was governed by a different philosophy to the one we have in the UK, and the parapets were designed to rebound the truck back onto the road bridge rather than taking the impact and requiring replacement. This is good for serviceability, but requires significantly more material in primary design.

3. **General**

The simple structure of this bridge allows for relatively easy access to all areas of the bridge that may need servicing. The obviously accessible stay anchors help with ease of serviceability.

**Foundations**

The foundations of each of the piers over the cable-stayed bridge are simply pad foundations set into the rock strata that make up the coastline of this part of Norway. This area of Norway contains a lot of Greenstone (soapstone), limestone and Precambrian gneiss, although I have
not been able to acquire detailed
gеotechnical data of the region.

**Durability and Maintenance**

This bridge needs to be able to withstand
day to day wear and tear, acts of
vandalism and effects such as fatigue.
As the bridge has been standing for a
number of years and appears to be still in
near perfect condition, from images I
have been able to view, (of which there
are, admittedly, rather few) I must
assume that the bridge has been designed
with durability considered and the
design has been effective.
As the bridge is only a road bridge,
vandalism is probably very rare,
however it could happen on a minor
scale from time to time and some
repainting may be required over the
years.
Probably the most susceptible part of the
bridge to corrosion are the cable stays,
they are coated with protective paint to
stop corrosion, however chips and cracks
in the paint could result in the steel being
open to the atmosphere and thus
susceptible to rapid corrosion. Thus,
regular checks and maintenance of the
stays is extremely important.

**Future Changes and Improvements**

With the Nordhordland Bridge, it is hard
to see any need for expansion in the near
future, and thus has not been designed to
be easily expandable. The only way to
really go about this would be to
construct a similar bridge next to it.
I feel that an improvement that I would
like to make to the bridge, if the money
was available, would be to expand the
proportion of the bridge that is cable
stayed to reach the end of the incline.
This would mean constructing a second
pier where the start of the pontoon
bridge is now and using stays to support
the tilting part of the deck. This would
be considerably more costly, however,
and the benefits would be merely
aesthetic.

**Conclusion**

In conclusion, there are many more
factors to be analysed for this bridge
which I have not had time to cover as
yet. The other load combinations, for
example, which include forces due to
skidding, friction forces at the supports,
and temperature loading on the structure.
Also, I would have liked to do an
analysis of the pontoon portion of the
bridge as well, and looked into the wave
forces governing a large portion of the
design criteria. Being curved, this
section of the bridge would also feel a
centrifugal force from traffic on it which
would need considering.

I conclude that the Nordhordland Bridge
has been very well engineered, with
cost-effectiveness, preservation of
materials, aesthetics and environmental
impact considered throughout the design
process.

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