A CRITICAL ANALYSIS OF THE ERASMUS BRIDGE:

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Abstract: The following paper gives information on the construction and design of the Erasmus Bridge, a critical analysis of the aesthetics and a consideration of the technical loading aspects to British standard codes.

Keywords: *Rotterdam, cable stayed bridge, Erasmus*

1 Background information:

1.1 General Introduction:

The Erasmus Bridge is an asymmetric cable stayed bridge located in the city of Rotterdam; the bridge is the last of three crossing points between the North and South areas of the city which are divided by the Nieuwe Maas River. Conceived in 1986 and costing 75 million Euros to construct [1], the bridge formed part of a larger redevelopment project which had seen the west of the city developed in the first stage leaving the area to the south largely untouched. In the next phase, a series of modern high rise buildings were to be built on the southern shore with a bridge required to link to the historic north, from the outset it was to be the central feature which would attract investment into the newly constructed area. The bridge was officially opened by Queen Beatrix on September 6th 1996 [2].

2 Design:

Multiple proposals for the bridge were initially put forward during the design stage; these included a four pole cable stay with similarities to the new Willems Bridge located just upstream, and a steel arch proposal. Eventually the decision came down to a city council vote where the single pylon design was chosen even though it cost 40 million Euros more.

Designed by the architect Ben van Berkel (BvB) in the early 1990's, the majority of the design work was carried out using computer software, specifically AutoCAD [3]. At the time this was a new way of working and allowed greater organisation both in the design and also integration into the fabrication stage.

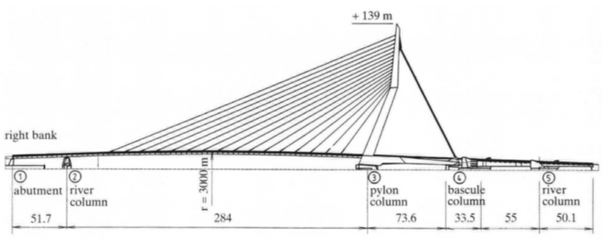

Figure 1: Elevation view [4]

In the following paper I will assume that the 284m section of cable stayed bridge is separate from the rest and will only carry out analysis on this part due to limited space. This assumption is justified due to the other decks spans not being made continuous and also the need for movement in the bascule part would necessitate no load bearing connections to that point.

2.1 Pylon:

In order for the bridge not to overwhelm the office block development to the south, it was felt that the pylon should not be made too tall. With early designs being up to 150m high, it became a key design issue to solve the height problem. By analysing the forces on a straight pylon it was found that if it is made to bend backwards then the bending moments are made more complex in nature but reduced in maximum intensity (see Fig. 2).

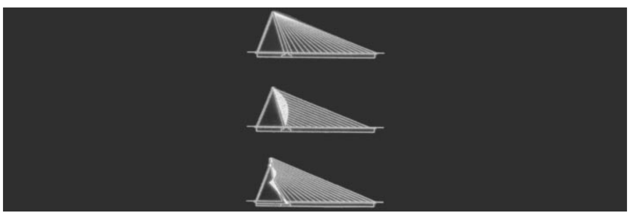
Figure 2: Bending moments

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This provides a shorted pylon which is more efficient in material usage. An initial feasibility study rejected the possibility of using a concrete pylon with the dead weight acting as the counter balance for the deck, similar in design to Santiago Calatrava's Alamillo Bridge. In this design the pylon was too short to provide the necessary balancing weight and it was also found that when considered with 60 tonne truck loading, there was a large difference between dead and live loading which would require that the pylon had back stays [4].

2.2 Deck:

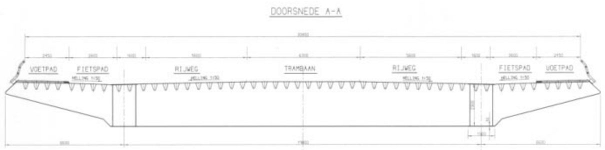

Figure 3: Cross section through deck

The very slender deck profile was not only aesthetically led, but there were imposed a number of design conditions which required it. There needed to be a clear shipping height in the centre of the span of 12.5m for at least 200m, yet the slope of the deck was limited to 1:28 to allow the trams and bicycles to pass.

The deck was therefore constructed from two 2.25m by 1.25m box sections joined every 4.9m by transverse sections. These transverse sections also cantilever out 6.7m either side for the pedestrian and cycle ways.

Figure 4: Profile from below [5]

The deck surface is constructed from a fully welded 18mm orthotropic S355 steel sheet with trapezoidal strengtheners at 600mm centres on the underside. Above this is an 8mm synthetic resin layer which replaces the asphalt mastic and provides significant weight savings [6].

2.3 Concrete Piers:

The piers were designed as a sculptural form rather than just what was required to take the loadings. The engineers designed steel tubes which would provide the support to the bridge and then the architects could mould the concrete form around these. As with the rest of the design this was all done in 3D, using computer modelling and the final form was chosen to both be sculptural but also to show the forces involved inside the pier.

Figure 5: Rendered computer model of pier 1

2.4 Cables:

Each stay cable is made from individually galvanised and polyethylene-coated strands. 30 to 45 of these strands are then put together in a high density polyethylene (HDPE) cover [7] and anchored at either end.

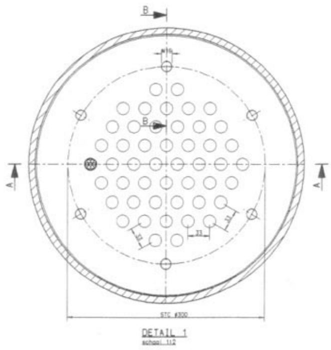

Figure 6: Section through cable

At the base of each cable is a steel cover which protects the cable from vandalism and vehicle collision (see Fig. 20).

2.5 Lighting:

In the Netherlands 1% of the funding for public projects must be used for art, in the case of the Erasmus Bridge the architects decided that this would be through lighting. The concept behind the lighting scheme was to reverse the form at night, this meant highlighting those elements which blended into the backdrop during the day and hiding through the use of shadow those which did not.

Figure 7: Lighting at night

3 Construction:

3.1 Pylon:

The choice of steel for the construction had many advantages, it was possible to construct the entire superstructure inside a factory where faster construction and higher quality workmanship was possible. The pylon is made from thermo-mechanically rolled S460ML high strength steel which was unusual in its use at the time [4].

The pylon assembly was fabricated in three separate pieces and then floated to the site from a factory around 100km down the coast. There was a 1800t central mast and the two 2000t backspans [3] which run along the bridge deck. The large weight and size of the pylon meant that the firm who was able to bid the lowest specialised in oil rig construction and although they had no experience of a bridge project they already had all the necessary heavy equipment.

Another relatively new technique used in the fabrication was the extensive use of cutting directly from the CAD drawings.

Figure 8: Transportation of the top mast by barge

Once the pylon was fabricated it was raised onto a large barge using an offshore semi-submersible crane with a capacity well over 10000t [4]. Once raised the pylon was welded together and had to be stabilised until enough stay cables could be fitted. This was achieved using a temporary tubular steel support structure between the back of the pylon and the deck, Fig. 9.

Figure 9: Pylon is raised and transported

Once this assembly had been floated to site during high tide it was positioned over the foundations and the supporting pontoon flooded.

3.2 Bridge Deck:

The deck was constructed by a separate contractor who built it offsite in 22, 15m sections. These were again shipped to site by barge and installed outwards from the pier. Each section was lifted by crane using a temporary truss girder placed underneath. With each subsequent section of deck added, the stay cable tension had to be altered to keep the structure in equilibrium. The backstay cables were built up in strands as each piece was added. By the fifth section it was possible to remove the temporary pylon supports.

Figure 10: Lifting a section into place

3.3 Foundations & Geotechnics:

In the bridge there were two different types of pier, the large bascule cellar which was needed to contain all the machinery and counterbalance for the raising mechanism and the other concrete piers which supported the deck at either end (see Fig.1).

Bascule Cellar: this was cast insitu on top of piles which sunk into the river bed under the weight of the underwater concrete. The bottom section contained a 9m deep base and on top of this the walls of the cellar were built. The pit was then drained leaving the finished pier.

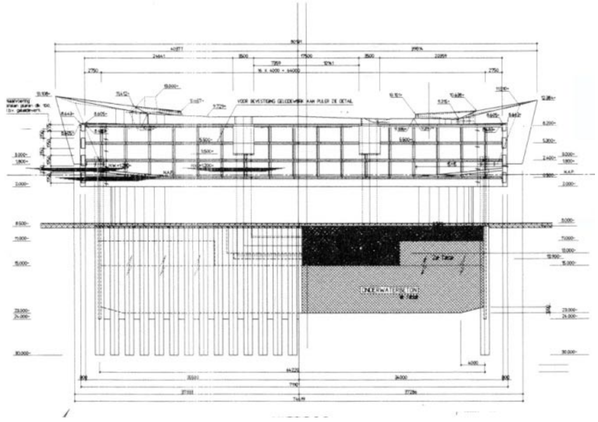

Figure 11: Bascule Cellar Section

Concrete piers: due to the relatively small size of the piers, these could be fabricated offsite and then transported by barge to the final location. The complex shape produced difficulties in creating the formwork but this was again solved by going back to the computer model and finding the formwork shapes required, from which large templates were made.

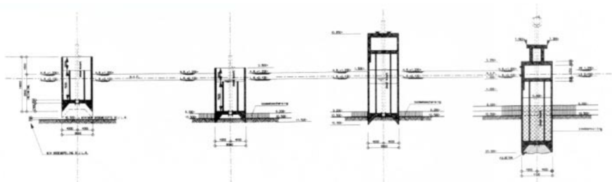

Figure 12: Placement of Pier

Each pier was constructed as a caisson at the bottom, this was then sunk into the seabed and concrete infill used to provide a solid base.

4 Aesthetics:

BvB had previously spent time working at the office of Santiago Calatrava and it is likely that he would have been influenced by the design of previous Calatrava projects through his critical analysis. He would most likely have been aware early on in the project of the design for the Alamillo Bridge, which was designed and completed during the design stage of the Erasmus Bridge. As such I feel it is possible to compare and contrast certain features of the Erasmus to what is considered by many to be a very aesthetically successful bridge.

4.1 Fulfilment of function:

The kinked pylon design gives a good appreciation of where it would seem most likely that the forces are acting. The stay cables are pulling the top forward so this is resisted by the back cables, the remaining force will then be a downwards forward facing thrust as shown by the change in direction of the pylon. At the base, the heel girder is very large, showing the resistance to overturning of the structure and also acting as an aesthetic addition to the pylon. As it turns out the 12m high rear span girders are far more than required and were constructed using stiff skirts, yet from a distance the massiveness of the pylon is reassuring.

The deck on the Erasmus is extremely thin to an extent that from a distance it appears that it could be flimsy to use when compared to the pylon structure.

4.2 Proportions:

With a span to height ratio of 2:1 between the pylon and deck span I feel this bridge achieves a more majestic appearance, in keeping with the area of cityscape than the 1:1 ratio of the Alamillo Bridge profile.

With a 2.25m box section, the bridge deck gives a span/depth ratio of 130 [4] over the clear shipping lane. This is already a very slender shape but it is further accentuated by the use of tapered edge beams which reduce the profile even more. This profile was used to keep a clear view out to the north from inside the city. While being critical of the apparent flimsiness of the deck I feel that the Erasmus deck proportions are very successful both in the non obscuration of the view and also in the impressive graceful appearance, more akin to a footbridge than one with 6 vehicle lanes and 4 of pedestrians.

4.3 Order:

The bridge is very ordered in appearance, the same deck profile and appearance is used throughout, even though it can be considered as two separate bridges with the bascule. A single asymmetric pylon makes it hard to have poor order as this becomes the central focus and there are no other pylons to compare to.

4.4 Refinement:

The architects dedicated much of the work on the bridge to the detailing and it is obvious when one looks in depth, as most elements are well thought through. The edge of the deck has an outwardly angled skirt (see Fig. 4) which gives a smooth soffit line by reflecting more light than the steel elements behind it. This soffit also continues up in a smooth transition to form the sloping balustrade along the footpath.

The design of the pylon is also impressively detailed into a sculptural piece with different angles used throughout each face, creating effects using areas of shadow and light.

Figure 13: Roadway at bottom of pylon [8]

One of the less successful elements is the transition between the pylons as the deck does not flow cleanly past but forms a dead spot behind them in the footpath.

4.5 Integration into Environment:

The environment in which the bridge exists is one of a modern cityscape due to much redevelopment after extensive bombing in WWII; the river is at a similar level to the surrounding area and there are several other bridges in reasonably close proximity. Many of Rotterdam's earlier bridges are truss girder designs, built in the 19th and early 20th century showing its industrial past. The most recent addition is another cable stayed bridge which quickly became a landmark feature. Into this environment the Erasmus fits perfectly, its modern design was specifically chosen to complement the business district to which it connects and the low cable stayed deck appears to float above the water.

4.6 Surface Texture:

The majority of the visual part of the bridge is formed in steel which is smooth and painted. Notable exposed parts are the bottom of the stay cables which are clad in stainless steel tubes and give a distinction in texture to the rest of the structure. The concrete piers are left in their original texture and to hint at the formwork, one of the bumpers in the bascule passage is made in heavy timber.

4.7 Colour:

As is quite common for cable stayed bridges, a "baby-blue" [3] colour was chosen. In this bridge the cables and the pylon are coloured like this which helps them to blend into the sky under certain conditions. This gives a greater range of appearances throughout the year from an almost transparent appearance to one which stands out boldly.

4.8 Character:

The Erasmus bridge has an undeniable character as many asymmetric cable stayed bridges do. Its design is somewhat more honest with its use of backstay cables to take the forwards force than the Alamillo, which suggests untruthfully that the loading is all taken by the pier leaning backwards. Since it was built, the Erasmus Bridge has become a distinctive landmark within the city and has picked up the nickname "The Swan" showing a local appreciation of the design aesthetics, from the people who matter most.

4.9 Complexity:

The design of the pylon adds complexity to the bridge through its use of a non natural and conventional shape. This gives it a more interesting and iconic form than the standard straight pylon design. The bascule part of the bridge is also technically very complex through its ability to raise the end part of the deck. While the normal observer will be unable to work out how this works the ability nonetheless adds interest through complexity.

5 Loading:

The bridge consists of two main parts, a 284m section which is cabled stayed and an 89m bascule section which when combined have an overall span of 802m when considered with the approach ramps [3]. As the two sections are independent of each other due to the movable nature of the bascule, they will behave as separate bridges and as such I will only carry out the analysis for the cable stayed part.

I have used BS5400-2:2006 which varies slightly from the 1978 edition in the loads which are taken.

5.1 Dead:

Dead loading covers all loading which will not change over the life of the bridge. The following dead load is calculated on assumptions, especially for the steel thickness as I have been able to find only limited technical dimensions. The nominal dead load is then calculated using BS648 material densities.

Steel truss:

$2.25m \times 1.25m, t = 25mm$

$volume = 0.1725m^2 \times 284m \times 2 = 98.0m^3$

$weight = 98.0 \times 7850 = 0.77 \times 10^6 kg$

Steel Beams:

$33m \times 2.25m, t = 15mm, 4.9m\ spacing$ [4]

$weight = 0.42 \times 10^6 kg$

$600mm\ diameter\ CHS, 50\ across\ deck$

$+18mm\ steel\ deck$

$weight = 2.08 \times 10^6 kg$

Total:

Total weight $3.45 \times 10^6 kg$

$$Total\ Force = 33.8 \times 10^3 kN$$
$$= 3.41 kN/m^2$$

This can be compared to a value of 170 tonnes per 15m section of deck [6] which gives 3.66kN/m^2, as it is not specified whether this includes superimposed dead loading, I will use my value of dead combined with a superimposed which will be the more conservative condition.

5.2 Superimposed Dead:

Blacktop:

$284m \times 33m, t = 100mm$

$volume = 937m^3$

$$weight = 937 \times 2400$$
$$= 2.25 \times 10^6 kg$$

Fittings:

$weight = 0.35 \times 10^6 kg$

Total:

$Total\ Force = 2.72 kN/m^2$

5.3 Primary Live loading (Traffic and pedestrians):

The bridge deck is 33m wide, this is then split symmetrically from outside to inside into a pair of 2.45m footways, 2.6m cycle paths, 5.6m carriageways and a central 6.3m tram line which carries two tram tracks (Fig. 3).

The carriageway corresponds to 2 notional loaded lanes in BS5400 which is the same number as it actually takes. Each of these lanes will be $5.6/2 = 2.6m$ wide. Two checks are specified for highway bridge live loads, HA which is a uniformly distributed load with either a knife edge load (KEL) or a single wheel load and HB which is a loading based on an assumed truck weight and wheel distribution.

HA: as the bridge is $50 < L < 1600m$ the HA load can be calculated using

$$W = 36\left(\frac{1}{L}\right)^{0.1}$$
$$= 20.46 kN/m$$

Over the notional lane the UDL is therefore $20.46/2.6 = 7.87 kN/m^2$. The KEL is then taken as 120kN.

HB: Normal HB loading is taken as 30 units of loading with each wheel representing 2.5kN/unit of load on a 16 wheel vehicle. This equates to 75kN per wheel.

Trams: As there is no provision in BS5400 for trams I have found a different source which specifies that the industry standard for light rail is a maximum axle load of 98kN [9] the trams will be up to 30m long and have a total load of 2000kN. This loading will therefore be treated as an HB load with the same variable lengths as given for trucks to take into account different lengths of carriages. The worst case will be two trams travelling in opposite directions on the bridge at the same time. Due to operating safety, there will never be more than two at any time.

Pedestrian live load: The bridge is over 36m long therefore:

$$Load = \left(\frac{HA\,UDL \times 10}{L + 270}\right) \times 5.0kN/m^2$$
$$= \left(\frac{20.46 \times 10}{284 + 270}\right) \times 5$$
$$= 1.84\,kN/m^2$$

As the combined footway and cycleway is over 2m wide the load may be reduced by 15% ($1.56\,kN/m^2$) for the first meter over 2m and 30% ($1.29\,kN/m^2$) for any further width. Over the width of 5.05m this gives an average loading of $1.56\,kN/m^2$.

5.4 Secondary load effects:

Centrifugal forces: The radius of the bridge curvature is 3000m [4], as it is over 1000m no extra longitudinal force is required.

Longitudinal loading: Worst of:

$8\,kN/m$ for the loaded length plus 250kN, up to a maximum of 750kN. For a 284m bridge this will be $(284 \times 8) + 250 = 2522 > 750, \therefore 750kN$. This is applied to an area one notional lane wide multiplied by the loaded length

25% of the total HB loading, applied between eight wheels of 2 axles spaced at 1.8m apart. This is 25% of 75kN/wheel = 18.75kN/wheel = 150kN for all eight wheels.

Accidental load due to skidding:

This is taken as 300kN nominal load in any direction with associated HA loading. This load effect is considered on its own as a secondary effect.

Parapet collision: 25 units of HB loading colliding with parapet. This is taken independent of other secondary effects.

Substructure collision: All the land piers are below the roadway and hence vehicular collision is not possible. The worst load case will therefore come from boats which could collide with the central pier or possibly with the bridge deck. The loadings will therefore depend on the maximum speed, size and risk of collision which would have to be determined by the relevant authority.

5.5 Wind:

The mean hourly wind speed for Rotterdam was not available and hence I have taken it as 20m/s in line with an average UK speed. The height above sea level and also the surrounding terrain is 17m [3].

$V_d = S_g V_s$
$S_g =$ gust factor
$= S_b' K_F T_g S_h'$
$= 1.55 \times 1.00 \times 0.94 \times 1.0$
$= 1.457$
$V_s =$ site hourly mean wind speed
$= V_b S_p S_a S_d$
$V_b =$ basic hourly mean wind speed
$= 20$ m/s
$S_p =$ probability factor
$= 1.05$
$S_a =$ altitude factor
$= 1 + 0.001 \times$ altitude
$= 1 + 0.001 \times 17$m
$= 1.017$
$S_d =$ direction factor
$= 1.00$
$V_s = 20 \times 1.05 \times 1.017 \times 1.00$
$= 21.36$ m/s
$V_d = 1.457 \times 21.36$
$= 31.12$ m/s

The closed parapet height is taken as 2.3m as there is only railings above this due to the designer's decision to keep the bridge profile as low as possible.

Bridge Deck:
$P_t = qA_1C_D$
$q = 0.613V_d^2\ N/m^2\ (adverse\ affects)$
$= 593.7$
$A_1 =$ solid area
$= 2.3\text{m} \times 284\text{m}$
$= 653\text{m}^2$
$C_D =$ drag coefficient
As distance between beams $> 7D_1$, $C_D = 1.5 \times single\ box\ C_D$
$b/d = 0.55$
$\therefore C_D = 1.5 \times 2.4$
$= 3.6$
$P_t = 1396kN$

Uplift: This is the other important wind loading.
$P_v = qA_3C_L$
$q = 593.7$
$A_3 = plan\ area$
$= 9372m^2$
$C_L = Lift\ coefficient$
$= 0.75\ as\ the\ bridge\ is\ not\ inclined$
$P_v = \pm 4168kN$

5.6 Temperature:

The entirely steel construction of the Erasmus will mean that temperature changes will have a greater effect on the bridge than concrete structures. As this bridge takes normal highway usage, a 1 in 120 year minimum and maximum temperature is used. Although there are no readily available 120 year isothermal maps for the Netherlands; looking at available climate data a temperature range of -5 to +30 Celsius seems a reasonable assumption especially given Rotterdam's close proximity to the sea which tends to reduce peaks.

With no data on the temperature at which the bridge was completed I will therefore assume a temperature change of 25ºC.

$\varepsilon = \alpha \times \Delta T$
Where $\alpha = 12 \times 10^{-6}/°C$, steels thermal coefficient
$\varepsilon = 12 \times 10^{-6} \times 25$
$= 300\mu\varepsilon$

This can then be multiplied by the length of the bridge deck to find the expansion which it would undergo should it be entirely unrestrained (length = 284m, Fig. 1):

$$\begin{aligned}\delta &= \varepsilon \times l \\ &= 300 \times 10^{-6} \times 284 \\ &= 0.0852m\end{aligned}$$

I have been unable to discover any information on the movement joint of the bridge, therefore making the assumption that either the movement joint becomes blocked or that the design contains no such joint, the stress in the steel would be:

$$\begin{aligned}\sigma^{c}_{apparent} &= \varepsilon \times E \\ &= 300 \times 10^{-6} \times 200000 \\ &= 60\, N/mm^2\end{aligned}$$

This apparent stress would be easily taken by the S355 steel used for the deck construction; hence the design is safe even if the movement joint becomes blocked.

5.7 Other Load Effects:

Creep: As the deck in constructed entirely from steel there will be no movement due to shrinkage or creep as this is only a problem in concrete. It would be necessary to consider stresses in the steel due to rolling, welding or lack of fit.

Where differential settlements can be expected it is necessary to take these into account. This analysis can only be done with geotechnical data from which expected settlements can be calculated. At the base of the pylons there are jacks which can be used to replace the supports and accommodate differential settlement, for this reason I do not feel that it is necessary to consider the effect.

Fatigue: Checks for fatigue are set out in BS5400 which would need to be considered for the bridge design, however it was deemed by the engineer on the project that fatigue was not the most critical factor for the pylon [4].

6 Strength:

Five bridge loading combinations are specified in BSS5400, these consist of three principal and two secondary combinations and take into account possible loading scenarios. Each loading combination has different associated partial load factors (γ_{FL}) all loads are also factored by $\gamma_{F3} = 1.10$ for steel design.

6.1 Load case 1:

All permanent loads + primary live loads:

Table 1: Case 1 Loads

Load:	γ_{FL}	
Dead	1.05	$3.41KN/m^2$
Super Imposed Dead	1.75	$2.72KN/m^2$
Live		
HA	1.50	$7.87kN/m^2$, KEL=120kN
HB and Tram	1.30	HB: 75kN per wheel Tram: 2000kN total
Pedestrian	1.50	$1.56\, kN/m^2$

HA loading: for a loaded length of >112m and <6 notional lanes BS5400 specifies a load factor of β_1=1.0HA for the first, β_2=0.67HA for the second and $\beta_{3/n}$=0.6HA for all subsequent lanes.

Factored Loads ($W \times \gamma_{FL} \times \gamma_{F3}$):
$W_{dl} = 4.12KN/m^2$
$W_{sdl} = 5.24KN/m^2$
$W_{live(vh)} = 12.99kN/m^2$
$W_{live(ped)} = 2.57kN/m^2$

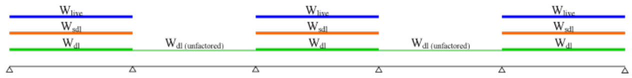

Figure 14: Max Sagging combination:

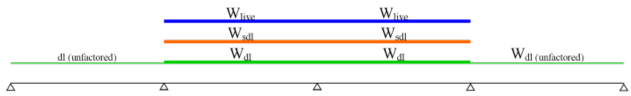

Figure 15: Max Hogging:

The live load combinations can be made up from HA and HB loading as given in specific occurrences in BS5400. For my analysis I will assume that the pedestrian and cycleway live loadings are always applied to the areas where it is worst and that there are always two trams positioned next to each other which span over two of the 15m spans. With only two lanes in either direction it is reasonable to assume that any HB loading will be straddled over both hence BS5400 type 2a loading combination will be taken.

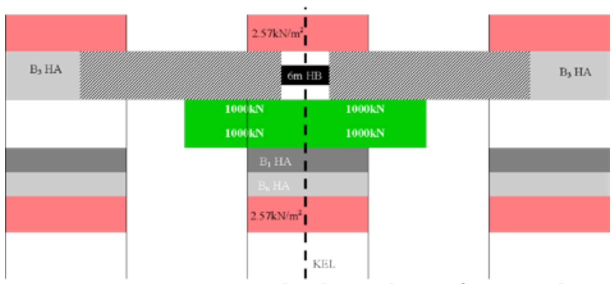

Figure 16: Max sagging, loadings shown for central 15m spans

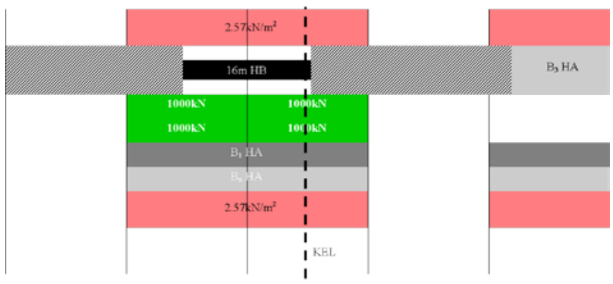

Figure 17: Max hogging, loading in centre

Per 15m segment this equates to UDL loadings of:

Pedestrian (Red): 12.98kN/m
Tram (Green): 67kN/m

β_1 HA (Light Grey): 33.77kN/m
β_n HA (Dark Grey): 20.26kN/m
Factored dead for whole width: 136kN/m
Unfactored dead: 112.5kN/m
Superimposed dead: 172.9kN/m

470kN/m on loaded spans
136kN/m on unloaded
Plus loadings in centre as shown in Fig. 16 and Fig. 17.

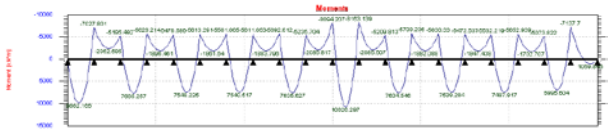
Figure 18: Bending moment diagram for maximum sagging condition

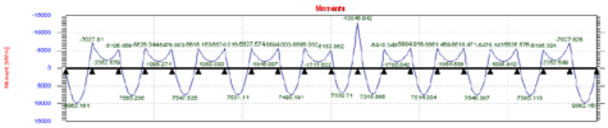
Figure 19: BM diagram for maximum hogging

The bending moments show a maximum sagging of 10826kN.m and a hogging of 12946kN.m.

6.2 Cables:

Assuming that each set of stay cables carries a 15m section of deck, it is possible to resolve the maximum force in each cable. As the HB and tram loading will travel along the bridge, the maximum load condition is shown in Fig. 16for the central section. This gives a load of 10370kN per section, which is 5185kN per cable. The worst cable is the end one which has the most shallow angle to vertical and is also the longest.

$$Tension = \frac{5183}{sin22.4} = 13601kN$$

I know that this cable is 300m long, 225mm in diameter and has a mass of 70kg/m [7] hence I am able to check whether this design seems correct using my loading.

Taking a minimum tensile strength of 1770N/mm^2 for the steel cable, to take the load of the end cable would require a cable $13601/1.77 = 7684mm^2 = 50mm\ radius$; this would have a weight of 61.5kg/m. The large discrepancy between the diameters of the cable will be due to the multi-wire arrangement used on the bridge cables rather than a large solid core. The actual weight of steel found in my calculation is surprisingly close to the actual amount used on the bridge with the many assumptions which I made to find the loading on the structure.

The following load cases are the remaining ones which would need to be checked to validate the safety of the bridge under all given conditions in BS5400. Unfortunately due to the lack of space in this paper and the further complexity of them I will not be able to carry strength calculations for them.

6.3 Load case 2:

Case 1 + wind:

a) P_t alone:
b) P_t in combination with $\pm P_v$:
c) P_L alone:
d) 0.5Pt in combination with PL $\pm 0.5P_v$:

6.4 Load case 3:

Case 1+ temperature + temporary loads

6.5 Load case 4:

All permanent loads + secondary live loads:

6.6 Load case 5:

All permanent loads + loads due to friction at supports:

7 Serviceability:

As well as the ultimate limit state it is also necessary to check the bridge under serviceability limit state to ensure that deflections will be acceptable when in use.

7.1 Deflections:

Making the extremely simplified assumption that, with the cables supporting the continuous beam every 15m, deflections will occur in a similar manner to a 15m simply supported beam with the UDL of 690kN/m and point load of 120kN for the KEL. This applies the worst case loading from Fig. 16 even though this uses ULS factors and hence is conservative.

$$\begin{aligned}\delta &= \frac{5wl^4}{384EI} + \frac{Pl^3}{48EI} \\ &= \frac{5 \times 690 \times 15^4}{384 \times 200 \times 10^6 \times 0.24} + \frac{120 \times 15^3}{48 \times 200 \times 10^6 \times 0.24} \\ &= 9.4mm + 0.2mm \\ &= 9.6mm\end{aligned}$$

This small deflection even under extreme loading conditions shows that the section is sized for strength rather than a deflection limit

7.2 Crash Barrier Collision:

On the outside pedestrian lane there are conventional handrails which will stop people falling off accidentally and with only a 12.5m drop into water, suicide barriers are unnecessary. On the inside vehicle lane it is very important that the crash barriers stop traffic as the stay cables are exposed at this point and also there is the pedestrian lane. Figure 20 shows the deep steel barrier used, which would, by inspection, deflect the majority of the force with little deflection. The sloped design is most likely to deflect the wind as the barriers are completely closed. BS5400 barrier calculation is done assuming that 25 units of HB loading collides with them, independent of other secondary effects.

It is known that the bridge was designed using 60t trucks, hence assuming a speed of 50m/s, an 80% momentum loss due to crumple and 5% of force transferred into the vertical direction:

$$F\Delta t = m\Delta v$$
$$F \times 0.1 = (80\%\ of\ 60000) \times (5\%\ of\ 50\, m/s)$$
$$F = 300\text{kN}$$
$$v = u + at$$
$$a = -25\, m/s^2$$
$$s = ut + 0.5at^2$$
$$s = 0.125m$$

A deflection of 125mm for the worst case collision will be fine as the cables are beyond that point; it is also likely that should a vehicle crash over the top of the barrier the steel covers on the cables are designed with enough strength to stop it from doing catastrophic damage.

7.3 Natural frequency:

To find a rough estimate of the natural frequency it is possible to use the Raleigh-Ritz method based on Euler's differential equation.

$$\omega_n = (\beta_n l)^2 \sqrt{EI/ml^4}$$
$$m = mass\ per\ unit\ length$$
$$= 20000\, kg/m\ \ (as\ calculated\ previously)$$
$$l = length\ of\ span$$
$$= 284m$$
$$E = 200 \times 10^9$$

With the assumption that the majority of the resistance comes from the two beams, and estimating that their steel thickness as 25mm I can find the second moment of area:

$$I = \frac{bd^3}{12} = 2\left(\left(\frac{1.25 \times 2.25^3}{12}\right) - \left(\frac{1.20 \times 2.20^3}{12}\right)\right)$$
$$= 0.2424m^4$$

The spans will act as clamped-clamped joints due to the continuously wended nature of the deck:

$$\therefore (\beta_n l)^2 = 22.37$$
$$\omega_n = 22.37 \sqrt{\frac{200 \times 10^9 \times 0.2424}{20000 \times 284^4}}$$
$$= 0.43Hz$$

Due to a vibration induced problem, described below, an engineering study was carried out which also calculated the deck natural frequency, as shown in Table 2. In this case their results are very close to my result for the first mode of vibration giving me confidence that my assumptions are reasonably correct. Limits of acceptable acceleration are set out in BS5400 but I do not feel it is necessary to analyse the bridge to these criteria due to the changes which were subsequently made.

Table 2: Calculated natural frequencies of bridge deck [7]

Natural frequency	**Bending**	**Torsion**	**Combined**
0.45Hz	1st mode		
0.67Hz	2nd mode	2nd mode	
0.84Hz			1st mode
0.94Hz	3rd mode		

Less than two months after the bridge was opened, on a day with a low 14m/s wind speed, the stay cables started vibrating and the deck moved noticeably. Due to the tolerances in the steel sheaths over the cables these produced an unsettling knocking noise. For this reason the bridge was closed to traffic until a solution could be engineered. At the time the problem was found to be the little understood rain-wind-induced vibrations. As a temporary solution to allow the bridge to open again the polypropylene ties which had been used in construction to stay the cables were reinstated and Rotterdam commissioned an extensive research project into the problem. The outcome of this work was a much better understanding of the problem such that a detailed finite element model of the structure which could accurately predict the vibrations of the cables, for example the longest cable had three modes of 0.38, 0.76 and 1.14Hz. To fix the excessive vibrations, tuned hydraulic dampers were installed which have solved the problem permanently.

7.4 Durability:

As with any exposed steel structure, corrosion can be a problem. The cables are all encased in a HDPE protective sleeve which will stop water from reaching them. The steelwork which is exposed for aesthetic reasons at deck level, such as the cable protectors and handrails are all made from stainless steel which will not need any maintenance to stop corrosion. The top of the deck is coated with an epoxy resin to stop water from seeping through the road surface and reaching the steel underneath. Lastly the bulk of the steelwork, including the pylon and underside of the exposed deck structure is painted, as the steelwork used is not a weather resistant variety (the steel used was S355K2G3 and S460ML [6]). Due to the factory fabrication this is most likely an Elastomeric Urethane coating or such like, which when applied correctly will last over 30 years with little maintenance required.

7.5 Vandalism:

A steel cover at the base of the cables is used to protect the cables from vandalism [7].

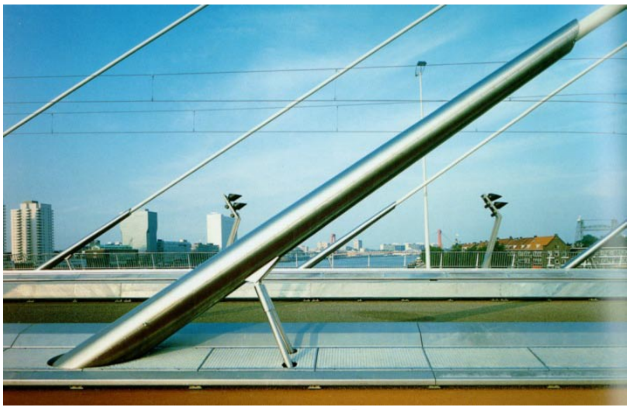

Figure 20: steel covers

At either end of the foot and cycle way, steel bollards stop vehicles from driving over the bridge as these could cause significant damage to areas not designed to carry them.

8 Future changes:

Currently the bridge supports 26000 vehicles per day and while the pedestrian walkways on either side would provide ample apace for an extra lane in each direction to be installed, subject to the capability to support the extra load, I do not think that this would ever happen. Rotterdam is split by a river and as such has much experience in building new bridges when extra capacity is required, this also tends to occur when major development in the city takes place both increasing the capacity required but also providing a time for construction when it will not be too disturbing.

In the current environmentally conscious climate Rotterdam would be more likely to wish to increase usage and capacity of its tram system or to persuade more people to cycle and walk. The Erasmus has been a success in the last two aspects, with more people using it to travel by foot than any previous bridge.

9 Inspection

The risk assessment carried out by the engineer on the project required special provisions be made to make all critical parts easy to access for inspection or repair. One of the most vital areas is the inside of the pylon and the outside cable anchorages. To inspect the pylon, ladders and an elevator in the west leg were installed however given the shape of the pylon it was not possible to include working platforms to carry out inspection of the outside. As permanent openings were also not possible a detachable pylon top is employed which can raise 2m and allow a rail guided inspection platform to be lowered out, see Fig. 21 [4].

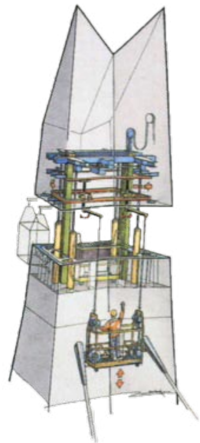

Figure 21: Detachable Pylon Top and Inspection Platform [4]

On the underside of the bridge the transverse sections are not as deep as the box girders to leave space for an inspection rail to run along.

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