Dimensioning of the cable-stayed Helgeland bridge

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Dimensionnement du pont à haubans de Helgeland

Bemessung der Helgeland Schrägkabelbrücke

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SUMMARY

The dimensioning of this cable-stayed bridge with a main span of 425 m and a very slender beam was governed not only by static loads, but also to a great extent by the high dynamic wind loads during construction and after completion. Non-linear effects played an important role in the determination of the stiffness of the structural concrete members.

RÉSUMÉ

Le dimensionnement de ce pont haubané en béton dont la portée principale est de 425 m et dont le tablier est très élancé, a été effectué en tenant compte non seulement des sollicitation de type statique, mais essentiellement des charges dynamiques dues au vent. Les effets non-linéaires jouent un rôle très important sur la détermination de la rigidité des éléments structuraux en béton.

ZUSAMMENFASSUNG

Die Bemessung dieser Beton-Schrägseilbrücke mit einer Hauptspannweite von 425 m und einem sehr schlanken Balken wurde nicht nur durch die statischen Lasten bestimmt, sondern weitgehend auch durch die hohen dynamischen Windlasten während des Baus und nach Fertigstellung. Nichtlineare Effekte spielten eine grosse Rolle bei der Bestimmung der Steifigkeit der tragenden Betonteile.

1. INTRODUCTION

The bridge is located near the town of Sandnessjoen, close to the Polar Circle in Northern Norway. Construction started in April 1989, completion is anticipated for September 1991. A general description is provided in Ref. [1], the layout is shown in Fig. 1.

Fig. 1: General Layout

2. DIMENSIONING AND DETAILING

2.1 Towerheads

Three basic tower shapes were investigated: H-, diamond- and A-shapes. The modfied diamond shape was found most suitable, combining economy, esthetics and structural characteristics.
The upper legs are rectangular boxes with 40 cm wall thickness, temporarily post-tensioned during erection with external tendons.

For the cable anchorage area at the tower head a comparison between solid cross sections with overcrossing backstays and forestays, a composite structure, and boxes indicated that concrete box sections with wall thicknesses of 0.42 m and 1.00 m are most economical and have the advantages of anchor head protection from the environment and easy inspection access, see Fig. 2.

At the tower head a large number of concentrated cable forces are introduced. The different models for distributing these forces must be evaluated very carefully, all forces must be followed until they are equally distributed over the cross section, all deviation and tie forces must be taken by steel - rebars or post-tensioned bars - all concrete struts must be checked for compression. Special care must be taken for the design of the nodal zones where the forces are introduced or where several struts and/or ties are intersecting.

Fig. 2: Section through Towerhead

The system shown in Fig. 2 has been proven to be easy to construct. The number of post-tensioning anchorages is reduced to a minimum, thus reducing the usual congestion in this region. The cable forces are confined by loop tendons, spaced at the same distances as the stays (1.50 m). The question, how much tension is lost along the tight loops due to friction has been investigated in a 1:1 model. With a smooth lubricated duct the standard friction and wobble factors have been confirmed.

The design has been done for different models:
a) truss action (see Fig. 3 and 4)
b) arch action (see Fig. 5)
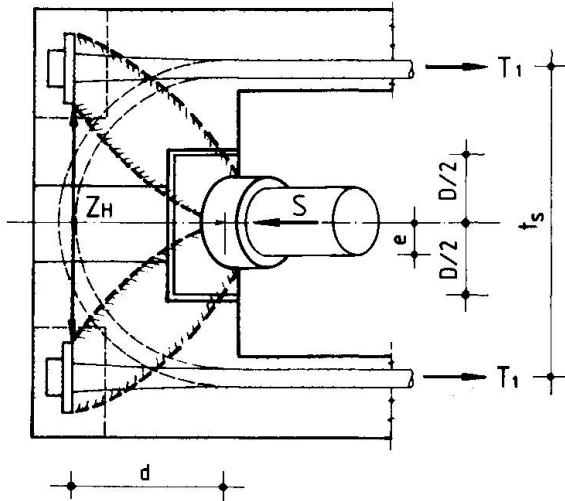c) frame action
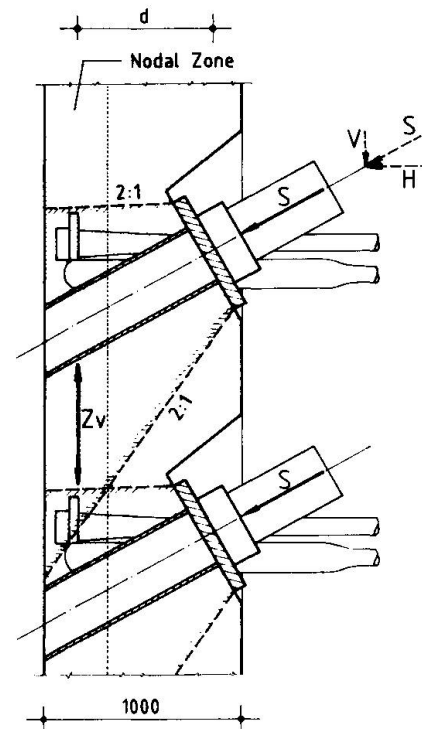d) punching.

Fig. 3: Truss Action at Towerhead in plan

Fig. 4: Truss Action at Towerhead in elevation

a) and b) give each a contribution to the required mild reinforcement, a) more to the horizontal and vertical part outside the loops, b) to the vertical part inside the loop. Local forces have to be covered by separate reinforcement like tie back reinforcement at the loops. The nodal zone is designed as vertical beam acting in tension and bending. It ties together the anchors and gives the possibility to distribute the splitting reinforcement over the total depth of the punching cone. c) and d) are additional investigations which took into account e.g. the double box structure in the upper region.

The reinforcement is designed to satisfy all models a) to d). After the cable forces are introduced into the short walls they must be distributed into the long wall. This design has been done with a multiple truss model.

It was decided by the contractor to use slipforming for all parts of the tower, including the anchorage regions. This was necessary due to the severe wind conditions with very strong, unexpected gusts, during which nobody wanted to take the risk of lifting the more exposed jumping forms.
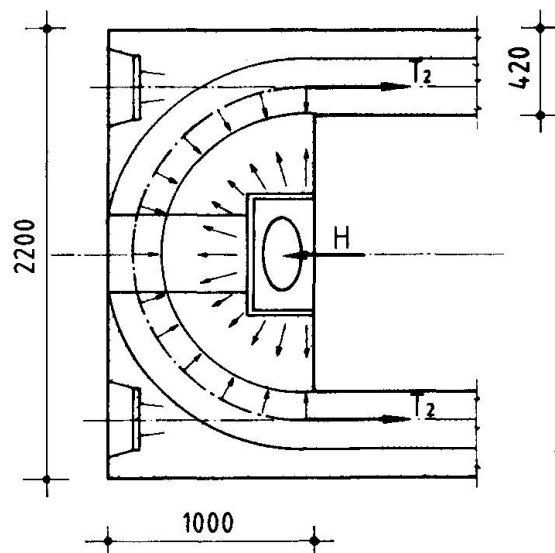

Fig. 5: Arch Action at Towerhead

This slipforming had a very stromg influence on the final rebar arrangement and detailing, especially in the anchorage area. The anchorage pipes have been welded on the ground to a steel frame, lifted up to the tower and bolted to the previous frame in the short walls. All the reinforcement had to be placed around these frames. This was achieved by a very accurate planning of each single bar location.

2.2 Deck Edge Beams

The required two lanes of traffic and a walkway resulted in a beam width of only 11.95 m, which leads to the remarkable slenderness in plan of 1:35.6. For the severe wind conditions the shallow, aerodynamically shaped 1.20 m deep cross-section was developed with a slenderness in elevation of 1:354. Non-linear effects in both directions were investigated using realistic non-linear stress-strain relationships.

Partial prestress was chosen for the beam in both directions to enhance its ductility. Straight tendons in the edge beams are continuous over the full length of the bridge and are coupled in every construction joint. Additional continuity tendons are threaded into empty ducts after completion of the beam across the mid-quarter of the main span. Full depth cross girders are provided at the cable anchorpoints. They contain the only transverse prestress. The reinforced 0.40 m thick roadway slab spans 7.25 m transversely and 12.2 m longitudinally.

The final design was governed by
a) dead plus live load action forces
b) action forces from dynamic wind
c) action forces during construction.

In a) the non-linear effect was taken into account with an estimated reduction of the E-Modulus by 2/3 because in ULS the stress level is above the linear branch of the stress-strain curve. Due to the slenderness of the bridge deck, the non-linear increase of the bending moments reaches 50 % of the linear live load moments, see Fig. 6.

For the first iteration of the wind analysis, an uncracked section without non-linear effects has been investigated. The resulting forces of this study gave the governing moments for the design of the beam almost all along the bridge and lead to the dense reinforcement shown in Fig. 7. The four tendons with 9 x 0.5" strands per edge beam have been provided mainly for erection. Due to partial prestressing the ratio between reinforcement and post-tensioning could be optimized with regard to structural and construction requirements. The combined bar and duct arrangement provides easy access for concreting and vibrating, convenient and economical splices and a simple post-tensioning arrangement.

Fig. 6: Non-linear Beam Moments

The design for wind and live load was done in ULS, however the serviceablility had to be checked in SLS. The crack width is limited to 0.2 mm at the rebars and 0.1 mm at the tendons under dead plus 60 % live load. In the cracked section the steel stress is limited to 200 N/mm² for this load case. The post-tensioning is sufficient to avoid tension in longitudinal direction under permanent load conditions.
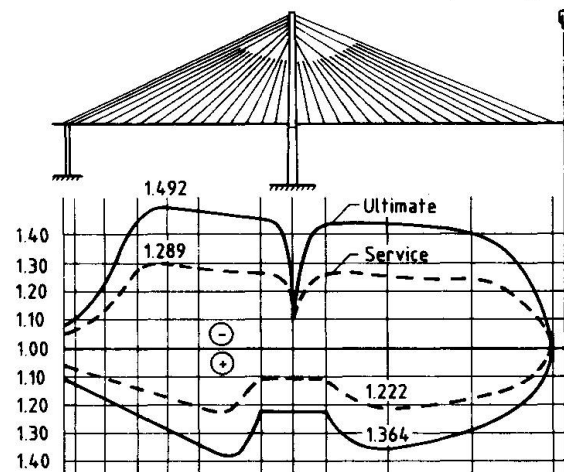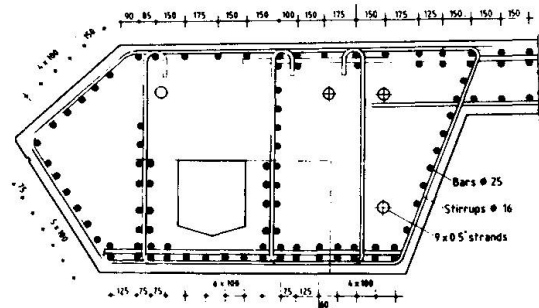

Fig. 7: Reinforcement in Edge Beam

3 AERODYNAMIC INVESTIGATION

The wind climate at the bridge site is very severe. The 50-year design wind speed (10 min. mean) at deck level is 50 m/s, with turbulence intensities horizontally of 10 m/s and vertically of 4 m/s. These rather large turbulences are due to the adjacent mountains.

An analytical time-history wind investigation was performed which is outlined in more detail in Ref. [2].

The non-linear force-deformation characteristics of the beam and the interaction between horizontal and vertical bending as well as torsion in the ultimate limit state were taken into account. For calculations in ULS action force versus deformation behaviour of the structure was followed up through a parametric analysis by a biaxial bending program, under consideration of the governing longitudinal force as well as of cracking of the tensile zone, see Fig. 8. These bending stiffnesses were essentially dependent on the actual reinforcement of the cross section.

A remaining tension stiffness of the concrete between the cracks of up to 1/3 was assumed. Calculations showed that nonlinear behaviour and bending interactions are satisfactorily described through the mathematical model shown in Fig. 9.
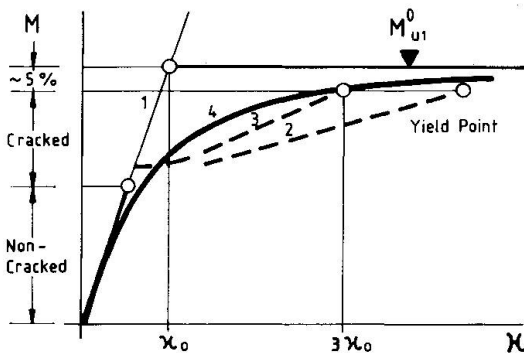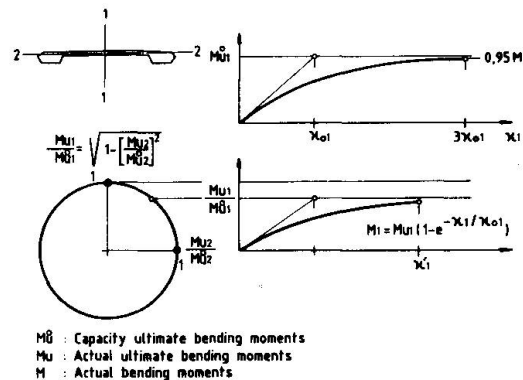

Fig. 8: Beam Stiffness Diagram

Fig. 9: Moment Interaction, ULS

The nonlinear behaviour in case of one-axial bending was estimated by

$$M_1 = M_{U1}^0 \left(1 - e^{-\varkappa_1/\varkappa_{01}} \right)$$

with M1   actual bending moment in the main plane 1
     MU1   corresponding bending moment capacity
     $\varkappa_1$ actual rotation
     $\varkappa_{01}$ fictive roation by bending moment capacity, assuming full-elasticity

In view of a conservative supplementary checking of the ultimate load carrying capacity, the deformations were limited to the elastic limit of the reinforcement; this was found at about
A simultaneous moment in the ortogonal direction was found to be effective essentially through a reduction of the bending moment capacity MU1. In dependence of the reduction factor,

$$\mathcal{E} = \sqrt{\left(\frac{M_1}{M_{U1}^0} \right)^2 + \left(\frac{M_2}{M_{U2}^0} \right)^2}$$

the reduced bending capacity becomes

$$M_{U1} = |M_1| / \mathcal{E}.$$

The stiffness receives the same reduction automatically, see definition in Fig. 9. The allowable roatation by achieving the elasticity limit of the reinforcement was found to be at about

$$ \alpha_1' = 3\alpha_{01}\left(1 - \alpha_2/3\alpha_{02}\right). $$

The interaction effect on the torsional stiffness was set to

$$ GJ_T = GJ_{T0}\left(1 - \frac{|M_1|}{M_{U1}^0}\right)\left(1 - \frac{|M_2|}{M_{U2}^0}\right). $$

The St. Venant torsional stiffness dropped in the ULS calculations occasionally to 10 - 30 % of the initial value.

The non-linear transverse moments at ULS at mid-span are reduced as a result of redistribution due to degressive stiffness, compared with the moments at service state multiplied by the safety factor of 1.6 as illustrated in Fig. 10.

4. BEAM CONSTRUCTION

In order to facilitate the exact positioning of the stay cables and to provide an economical solution for the transmittal of the horizontal stay force component during casting, the concrete corbels and adjacent parts of the beam with the steel pipe are precast, see Fig. 11.

Governing stages in the erection phase were stressing the cables to final length for max. pos. moments and placing the precast elements and rebars into the catilevered formtraveller for max. neg. moments. The steel stress in the outermost bars was limited to 200 MPa, keeping the crack width at the exterior steel layers smaller 0,1 mm. The erection post-tensioing and the rebars had to be adjusted several times during the final design phase to take care of the changes in the equipment weight.

5. ACKNOWLEDGEMENT

The owner of the Helgeland Bridge is the Norwegian Road Administration, Nordland, with Mr Wilhelm B. Klaveness in charge. Main consultants are A. Aas Jakobsen, Oslo, Norway. Leonhardt, Andrä and Partners, Stuttgart, Germany, are special consultants for the cable-stayed bridge, including the aerodynamic investigation.
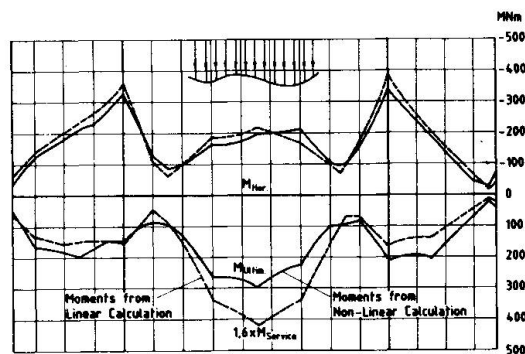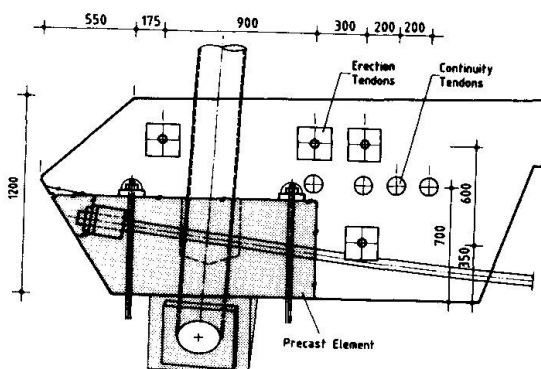

Fig. 10: Transverse Beam Moments, ULS

Fig. 11: Edge beam near cable anchorage

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[2]   KOVACS, I., SVENSSON, H.S., and JORDET, E.A.: Analytical investigation of the cable-stayed Helgeland Bridge in turbulent wind. To appear soon in the Journal of Structural Engineering, ASCE.