Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 3 (1948)

Artikel: The bridges at Sandé

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DOI: https://doi.org/10.5169/seals-4026

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Les ponts de Sandö

Die Brücken bei Sandö

The bridges at Sandö

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Situation

The bridges over the Angerman River at Sandö, which were built during the years 1938-1943, form part of the Stockholm-Haparanda coastal road.

The river is divided at this point into two wide arms and one narrower one, separated from one another by the islands Killingholmen and Sandön. The main channel is bridged by a hingeless concrete arch with a span of 264 metres, whereas the other wide arm, the Klockestrand Sound is bridged by a continuous concrete girder with a maximum span of 71.5 metres. The width of the roadway is $12^{m}00$ of which $2 \times 1^{m}25$ are foot-paths.

Nature of the ground

The nature of the ground is comparatively unsatisfactory. There is a deep rock bed below the bottom of the river, covered by a very thick deposit consisting mainly of fine sand, changing upwards to loam and silt with a top stratum of loose clay. The depth of water in the river is 18 to 19 metres. At the banks, where the soil has been washed away, the rock is steep and irregular. Borings taken in the river's channel reached depths of 37 to 48 metres below the water level without rock being encountered.

On each side of the main channel the rock rises above the surface. It was thus possible to lay the greater part of the foundations for the approaches on rock. In the Klockestrand Sound the ground consists of fine sand loosely deposited to a considerable depth, and it was therefore necessary to use piling. The depth of water at this point is about 10 metres.

Loading

The roadway is designed to carry a load of one or more traffic lines of 9-ton motor vehicles, with or without an 8-ton two-axle trailer and with a 15-ton three-axle motor vehicle in each line with or without a 10-ton two-axle trailer, and a 15-ton road roller. Furthermore, the deck structures are designed for an 8-ton axle load having a wheel-base of 1.7 metres in each line. The pavements are designed to carry a uniformly distributed load of 400 kg/m^2.

Arch span over the main channel

The arch span over the main channel consists of a hingeless concrete arch having an effective span of 264 metres. The rise of the arch is 40 metres and the clear headway is 40 metres for a width of 50 metres.

The arch is of box form (see fig. 4) and it is divided by transverse walls distributing the loads from the columns to the four longitudinal

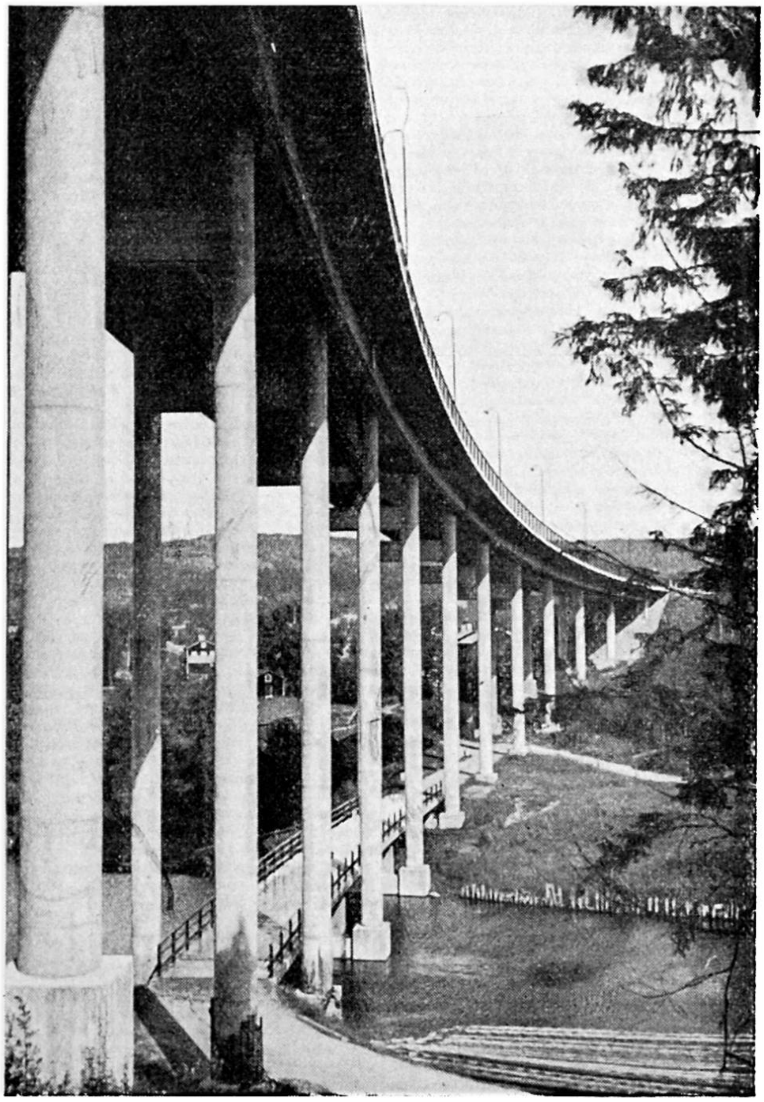

Fig. 1. Approaches.

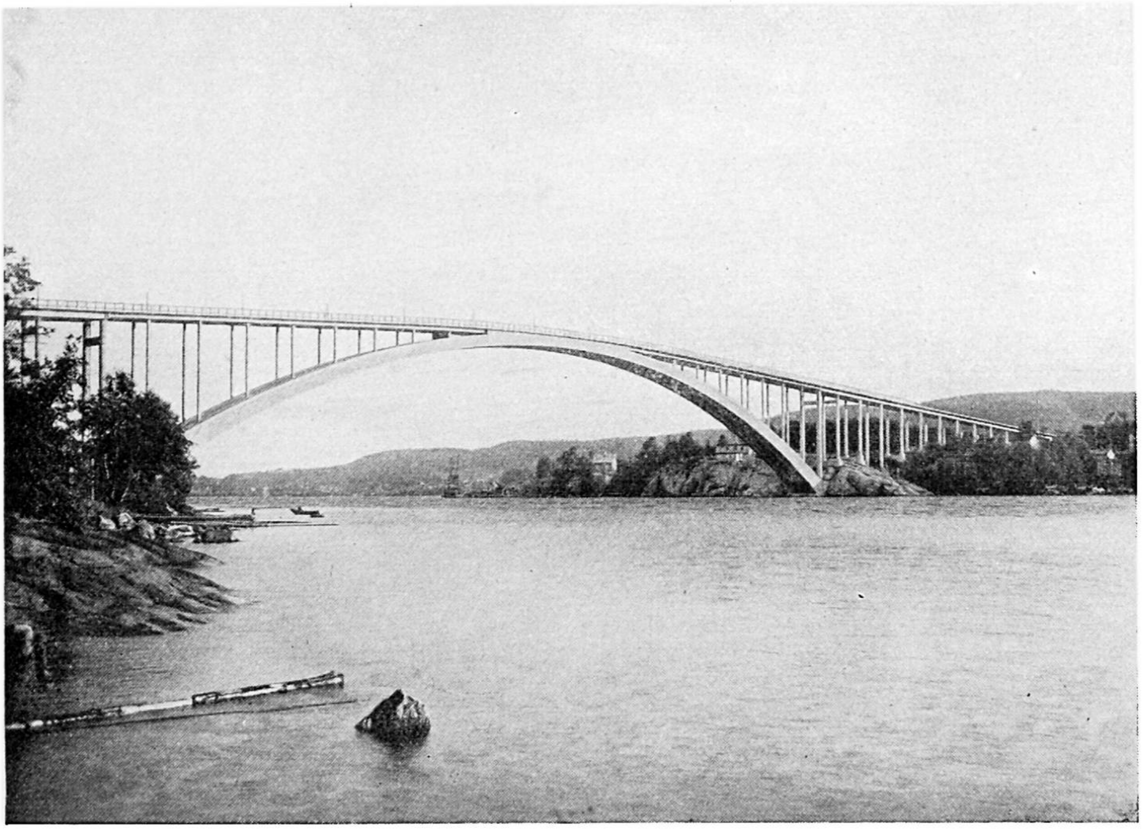

Fig. 2. Bridge over the main channel.

walls of the arch. The wall thickness is 30 cm except near the springings where the thickness is increased for a short distance. The constant width of the arch is 10.1 m and the height varies from 2.9 m at the crown to 5.0 m at the springings. The axial curve of the arch follows the equilibrium polygone for dead load.

The moment of inertia varies according to the formula :

$$I_x \cdot \cos \varphi_x = (1 + 8\ (K - 1) \cdot \xi^3) \cdot I_0$$

where

$$K = \frac{I_{10}}{I_0} \cdot \cos \varphi_{10} = 3\ 065$$

and I_0 and I_{10} are the moments of inertia at the crown and springings respectively.

In calculating the stresses in the arch set up by the live load and the temperature effect, the influence on the bending moments caused by the stiffening effect of the deck structure has not been considered but allowance has been made for the effect of the reinforcement; the ratio of the moduli of elasticity of steel and concrete being taken as 15.

The maximum stresses under normal and exceptional loads may be seen from fig. 5.

In calculating the temperature stresses it has been assumed that

$$t_{\max} = 18{,}5°\ C \text{ and } t_{\min} = -23°\ C\ .$$

The wind pressure is transmitted by the deck structure and the con-

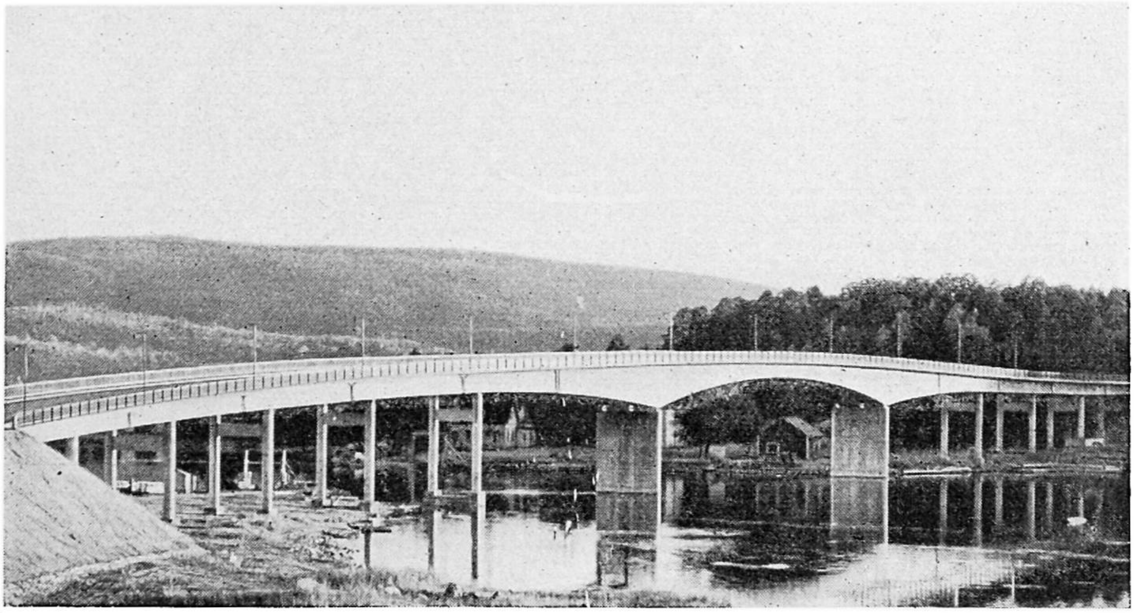

Fig. 3. Bridge over the Klockestrand Sound.

crete arch jointly and the proportion to be carried by the deck structure and the arch respectively is determined by the requirement that the deflection of the bridge roadway and the concrete arch shall be equal at the centre of the span. If the wind pressure at the crown of the arch is taken at 175 kg/m² and E = 375 000 kg/cm² with G = 0.385 E, the maximum deflection at the crown will be 50 mm and the maximum stress in the arch 18 kg/cm².

Safety against buckling in a vertical plane

Safety against buckling in a vertical direction was calculated according to Dischinger [1] the variable moment of inertia being taken into account. As the arch is comparatively flat, the influence of the horizontal displacements during buckling on the magnitude of the buckling load is small and has therefore not been taken into account. The modulus of elasticity E for loads of short duration has been assumed to be 375 000 kg/cm².

With unsymmetrical buckling, the critical horizontal thrust will be

$$H^a_{crit} = 27.7 \cdot \frac{E\,I_0}{a^2} \qquad (1)$$

where I_0 is the moment of inertia of the arch at the crown and a is half the span = 132 metres. Similarly for symmetrical buckling

$$H^s_{crit} = 41.7 \cdot \frac{E\,I_0}{a^2} \qquad (2)$$

With an unsymmetrical live load the factor of safety against buckling,

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(1) *Untersuchungen über die Knicksicherheit, die elastische Verformung und das Kriechen des Betons bei Bogenbrücken* (*Der Bauingenieur*, 1937, H 33).

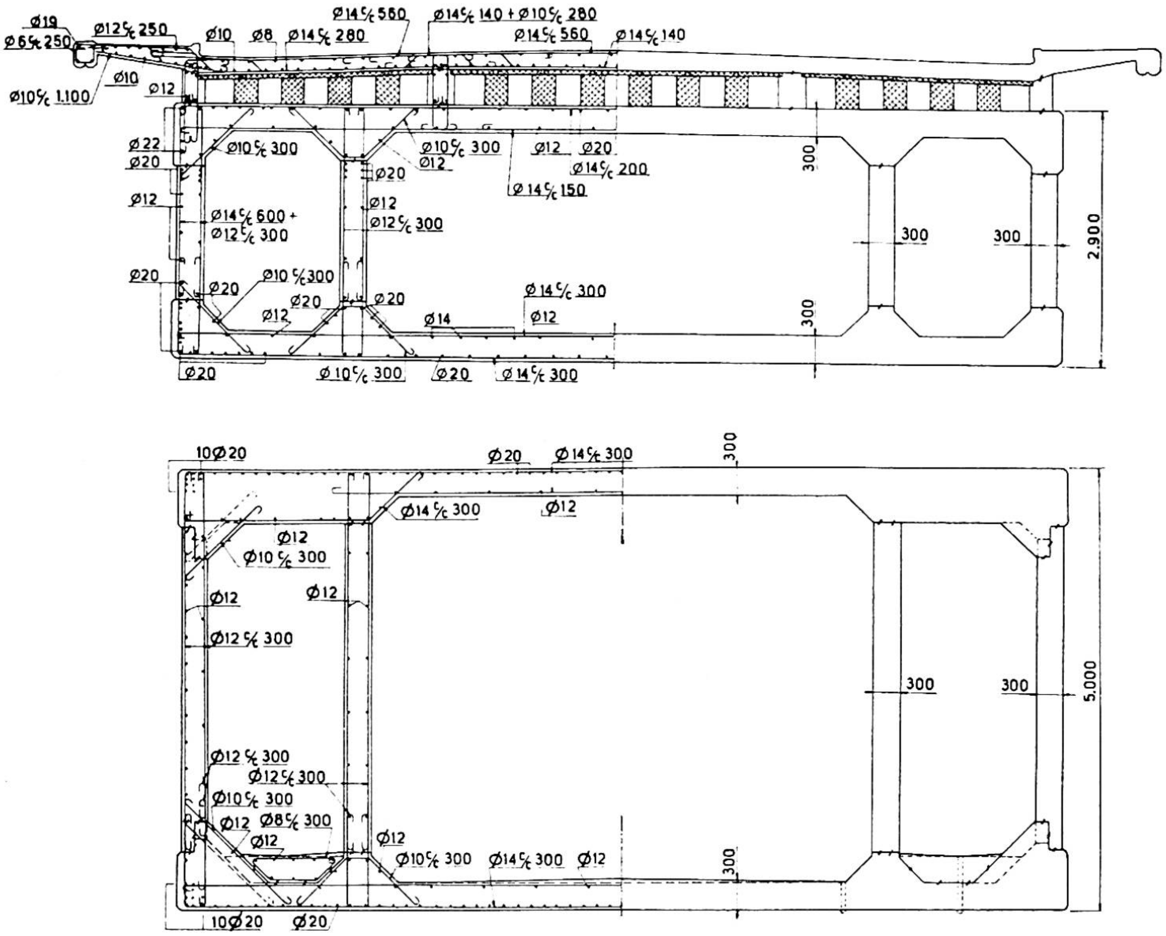


Fig. 4. Sections through the arch at the crown and in the springing.

according to Euler, will be 8.10 ($E = 375\ 000$ kg/cm²). The deflection will be 56 mm, corresponding to an additional stress of 4 kg/cm².

Centering

A considerable proportion of the cost of an arch is due to the falsework. In view of the considerable depth down to the solid ground encountered in this case, a freely suspended timber centering was found to be the cheapest.

This centering was constructed on land on a temporary trestle, as a framed arch 4 m high, with a span of 247.4 m and a rise of 36.5 m. The flanges were formed by slaps 12 m in width, consisting of 2″ × 8″ planks

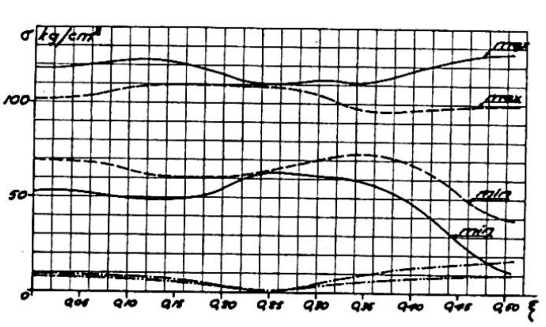


Fig. 5. Stresses in the arch.

— — — — Own weight + live load.
— · — · — Only wind.
— ·· — ·· — Only temperature.

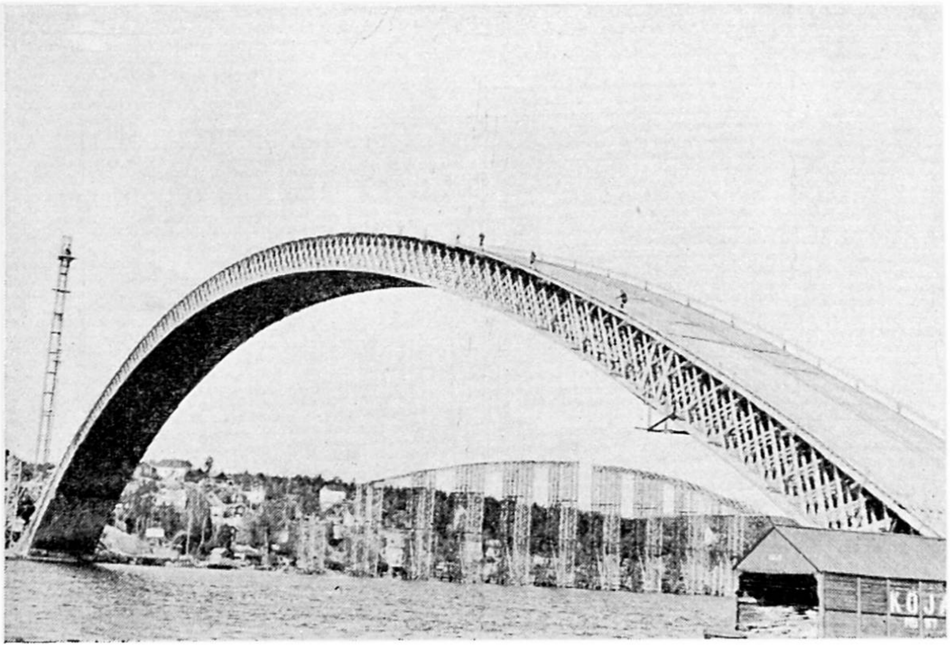


Fig. 6. Freely suspended timber centering.

set side by side, the widths vertical, and spiked together. The width of the slabs increased from 11.1 m at the crown to 13.1 m at the springings. They were joined together by 14 longitudinal webs, consisting of crossed diagonals 2″ × 6″. All timber was pine with a cubic weight of 0.35 and a moisture content of 15 %. All forces were transmitted by spiked connections. The spikes were 300 mm wire spikes 8 mm square having a tensile strength of 60 kg/mm^2. They were driven in without previous drilling by means of light pneumatic hammers.

Two barges with a loading capacity each of about 1 000 tons, which were first filled with water and then emptied were used in raising the timber arch weighing 1 000 tons from the temporary trestle and in transporting it to the site of the bridge. During this transport which took place in May 1939 the horizontal thrust was taken by steel ties.

In August 1939 the concreting of the bottom slab of the arch was started. Everything indicated that the work would proceed satisfactorily, especially as the measured deflections of the centering satisfactorily corresponded with those previously calculated (see fig. 7). During the last day of August, however, when the concreting of the bottom slab of the arch was nearly completed, the centering suddenly collapsed during an intermission in the work.

By subsequent calculations it was found that the stress in the bottom

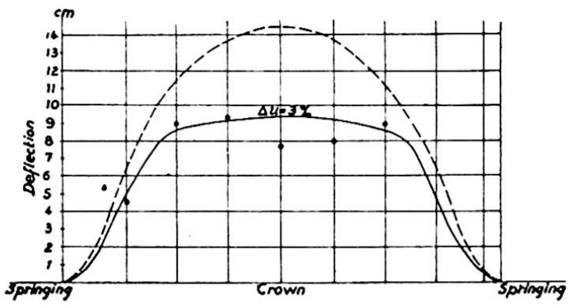


Fig. 7. Deflection at the freely suspended centering some hours before the collapse.

- - - - - - - - Calculated without regard to increased moisture.
———— Calculated with regard to increased moisture.
ooco Measured.

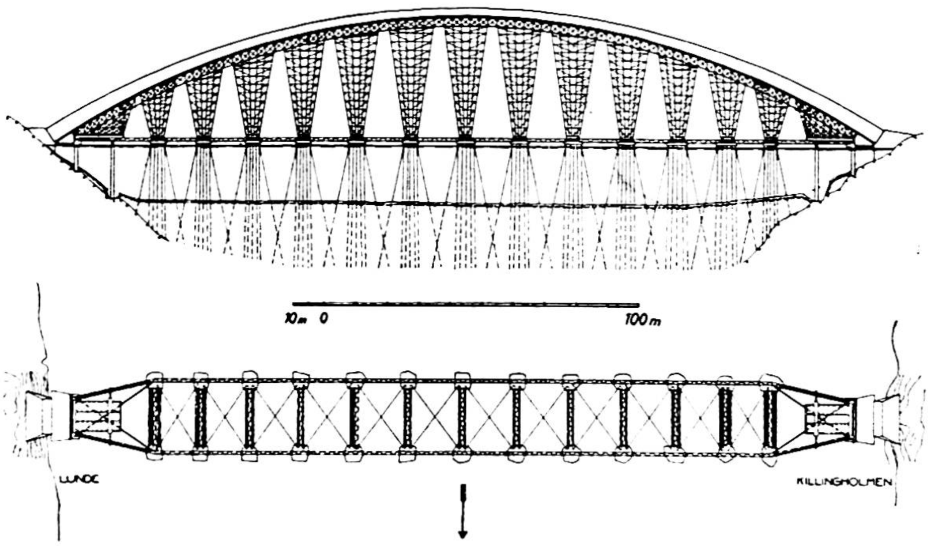


Fig. 8. The trestle timber centering. Plan and section.

flange at the crown due to the normal thrust from dead load, temperature and increased moisture content of the timber was 69 kg/cm². From the moment due to the bending of the planks, reduced by 33 % by creeping, and the eccentricity of the normal thrust in the bent flange the total bending stress was calculated to be 32 kg/cm² corresponding to an eccentricity of the normal thrust of 1.56 cm.

In view of the fact that very damp weather was experienced during concreting it was calculated that the moisture content of the timber had increased to 20 %. Repeated tests carried out later with this moisture content and using large specimens proved the compressive strength of the timber to be 195 kg/cm² ± 30 to 35 % for loads of short duration. For loads of longer duration the strength was reduced by about 40 % to 117 kg/cm², which at the eccentricity in question implies that the risk of failure exists when the mean stress in the cross-section is about 84 kg/cm² as against the actual corresponding stress of 69 kg/cm². When it is taken

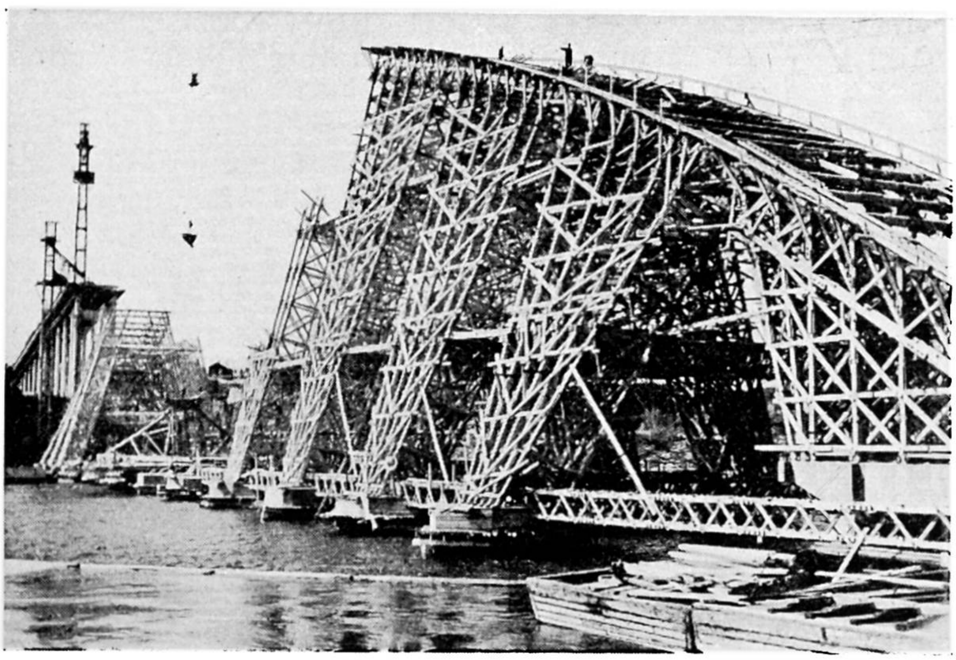


Fig. 9. The trestle timber centering, while building.

into account that the jointing and the spiking together of the planks reduce the strength still further—spiking alone probably accounts for about 10 %—and that the strength of the cross-section at the crown possibly may have been somewhat less than the mean strength of the timber, it is apparent that the collapse of the centering may probably be explained by the fact that the persistent damp weather and long loading period reduced the strength of the timber to such an extent that failure occurred at a stress which is generally considered to be safely below the ultimate strength.

Since the causes of the collapse of the centering could not be ascertained immediately, the new centering was constructed as a timber trestle (see fig. 8, 9) built up on piles 40 m in length (see fig. 10) which were driven down 20 m through the loose ground strata. Each pile consisted of 11 ordinary timber poles, 6″ × 10″ at the top and 11-16 m long. The piles were held together by 7/8″ bolts and 3/4″ jag washers 0.50 m on centers. The maximum load amounted to 26 and 40 tons for long and short periods respectively. In designing the piles it was assumed that the modulus of elasticity for wet timber was 90 000 and 63 000 kg/cm² for short and long loading periods respectively. It was also taken into consideration that a certain displacement might occur in the bolt connections. As a result the carrying capacity was reduced by about 24 % as compared with a homogeneous pile having the same cross-section, and assuming an initial deflection of 1/150. Loading tests carried out with a number of piles confirmed the accuracy of these calculations.

The piles were arranged in groups of 13, on top of which concrete piers were cast. The groups were arranged parallel to the bridge in two rows 23 m apart. In the longitudinal direction of the bridge the spacing was 14-17 m. The concrete piers encasing the tops of the piles were connected by horizontal timber trusses extending from shore to shore in which the chords and verticals were made up of timber framework and the diagonals of round steel bars.

The supports of the centering consisted of 8″ × 8″ posts of sawn timber with longitudinal and transverse braces of $2\frac{1}{2}$″ × 6″ planks. On top of the supports transverse crossheads were built 3 m on centers. These crossheads consisted of timber trusses with 8″ × 8″ top chords on which 4″ × 8″ longitudinal beams 0.5 to 0.7 m on centers were placed. On these beams a 1″ transverse sheeting was nailed on top of which a longitudinal sheeting of 1″ planed boards was laid for the arch soffit.

A description of the centering is published in *Betong* H 3. 1946.

The concreting of the arch was carried out in 4 stages : the bottom slab, the inner walls, the external walls and the top slab. After each stage 300-tons hydraulic jacks, in all 24, were mounted in a construction joint at the crown. The thrust effected by the jacks was gradually increased with the load on the centering up to a total of 6 700 tons. After an adjustment of the thrust line had been made ($M = -$ 1 025 tm $H =$ 6 270 t) the joint at the crown was filled in (see fig. 11).

Testing of the concrete

In order to maintain the specified cube strength of 500 kg/cm² a great number of test specimens were made. For every 50 m³ batch of concrete, one, two or three series of test cubes were prepared, which were crushed after 7, 28 and 90 days respectively. Furthermore, reinforced beams and prisms were also prepared, though to a lesser extent, for investigating the tensile strength, and modulus of elasticity. The results are given in the following table.

Age Days	Test N°	Average crushing strength kg/cm^2	Average error %	Negative deviation max. %
7	140	383	10.6	27
28	159	595	9.4	20
91	60	654	8.4	19

Cube strength of 20 cm concrete cubes

The modulus of elasticity for compression after 28 days, measured as the secant modulus for an increase in stress from 20 to 300 kg/cm^2, was 347 000 $\pm$ 7 %. The water-cement ratio varied between 0.39 and 0.44, and the weight per unit of volume between 2.41 and 2.52 kg/dm^3.

Shrinkage and creep of the concrete

The shrinkage and creep of the concrete were studied by means of unloaded and loaded concrete prisms 20 $\times$ 20 $\times$ 100 cm which were stored

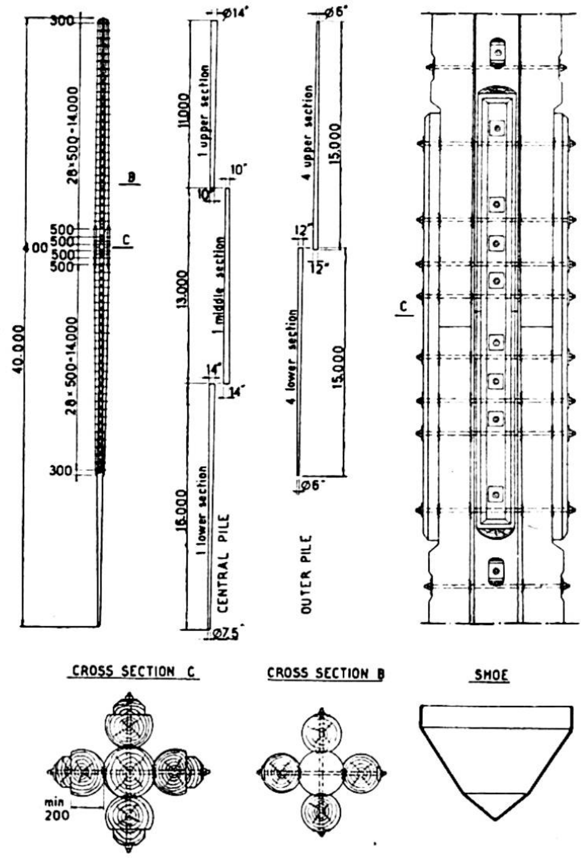


Fig. 10 (above). Pile of timber.
(Length : 40 m. — Loading : 26 t.)

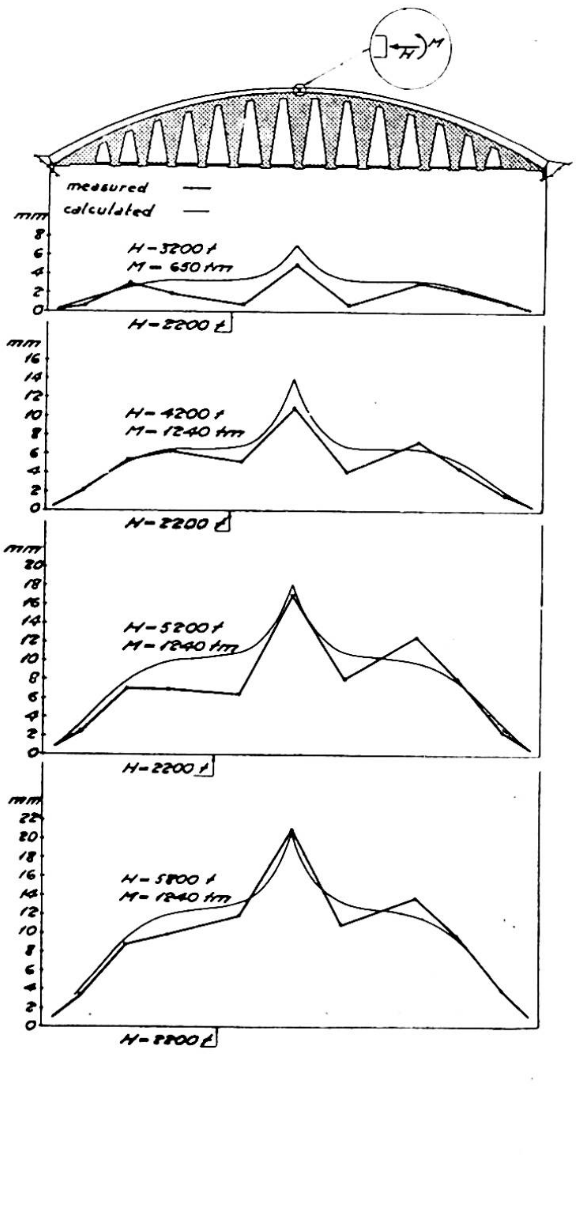


Fig. 11 (right). Movements at the raising of the arch from the centering.

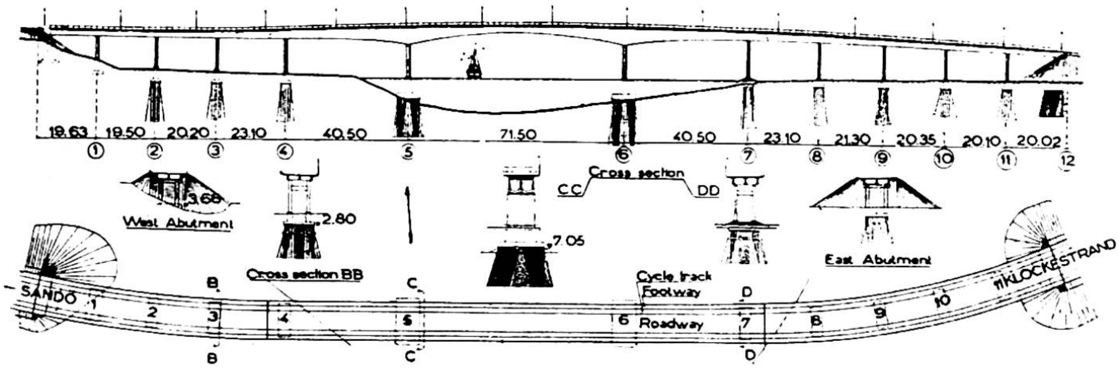


Fig. 12. Bridge over the Klockestrand Sound.

in air at a constant temperature of + 17° C and 60 % relative humidity. Based on the test results a calculation of the vertical deflection at the crown of the arch was made. It was assumed that shrinkage and creep depend on the diffusion of water vapour taking place through the concrete when the vapour pressures in or outside the concrete body varies and that these variations can be treated mathematically as a thermodynamic problem, or as the flow of water in the pores. In this way the fact may be taken into

Fig. 13. The shrinkage K and creeping f of the concrete as a function of the time **t**.

$K = \frac{100 - R}{100} k \cdot \mu_k$;

$f = \sigma \cdot F \mu \cdot f$

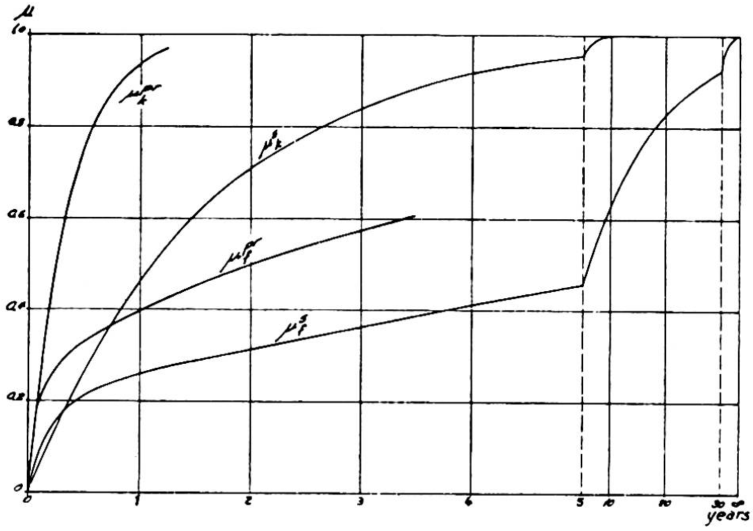
R = the relative moisture in the air.

k, F, d Material constants.

σ = kg/cm2.

μ^{pr} refers to a prisma 20 × 20 cm.

μ^{s} refers to a slab 30 cm thick.



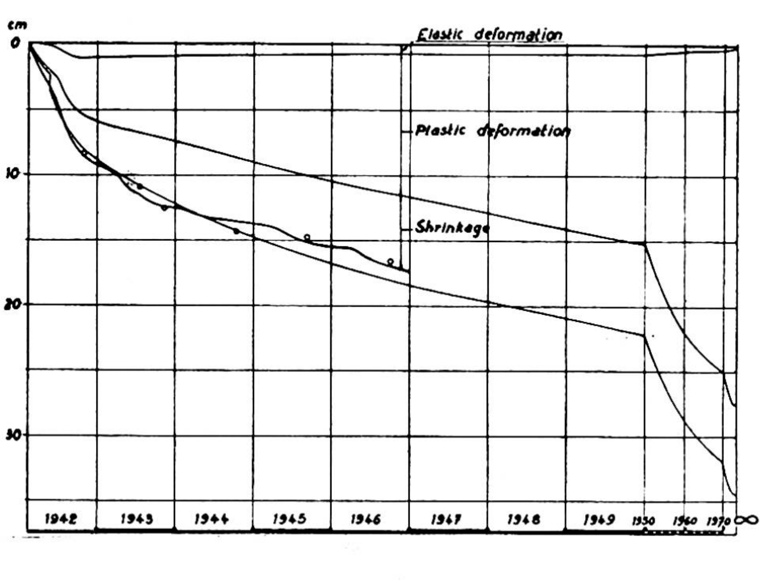


Fig. 14. Calculated and observed position at the crown of the arch with regard to the shrinkage and creeping of the concrete, calculated position with regard to the real moisture in the air.

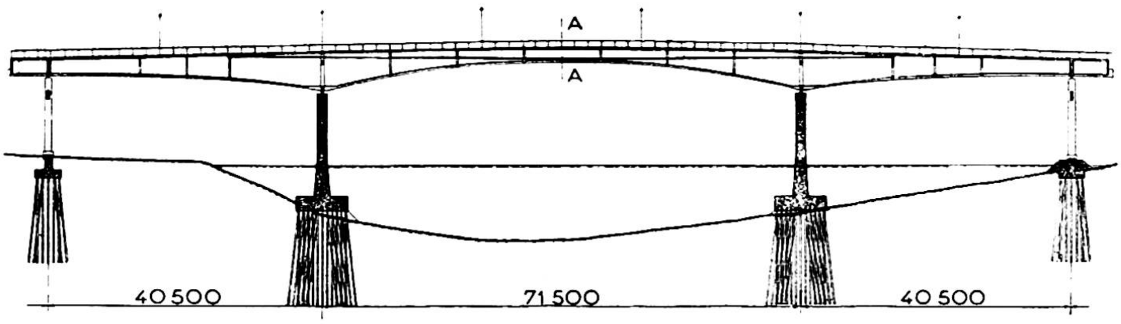


Fig. 15. Longitudinal section.

account that concrete bodies of different sizes though otherwise subjected to similar conditions shrink and creep at different rates (see fig. 13). Thus the test results obtained on the small concrete prisms in the laboratory can be converted to apply to the slabs of the arch. It will be seen from fig. 14 that satisfactory conformity was obtained between the measured and calculated values. The actual humidity of the air was also taken into account until the end of 1946. A point of special interest is the agreement between the measured and calculated position of the crown in the years 1945 and 1946 when humidity of the air was above the normal.

The girder span over the Klockestrand Sound

The three main spans of the bridge over the Klockestrand Sound are constructed as continuous reinforced concrete girders of varying depth in which an initial compression has been introduced. The centre opening has a span of 71.5 m, and the side openings 40.5 m. The depth of the girders varies between 2.55 m at the end and 5.74 m over the intermediate supports. The depth in the centre of the wide span is 2.11 m. The bridge consists of 3 longitudinal girders each 45 cm in width with a small longitudinal rib at the bottom.

The initial compression force of 1 070 tons is effected by 84 tie rods with a diameter of 30 mm of steel having a tensile strength of 52 kg/mm². The tie rods are anchored in the deck at the end spans and are freely suspended between the anchorage points below the deck and between the girders. They were stressed by hydraulic jacks up to 1 800 kg/cm². Owing to the curvature of the centre-of-gravity line of the girders and the eccentricity of the thrust, bending moments are produced in the girders which counteract the moments of the dead load. With the compression applied in this case the reduction is only 20 %, but by increasing the compression to about 3 200 tons it would have been possible to obtain a structure entirely subjected to compressive stresses. The use of steel with a high yield point in the tie rods would be of advantage.

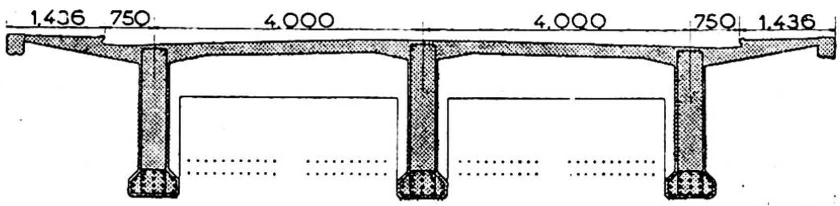


Fig. 16. Cross-section with the freely suspended tie-roads.

Due to the form of the girders the distance between the anchorage points of the tie-rods is independent of the initial force. Longitudinal deformations due to the normal thrust are approximately of the same magnitude as, but opposed to, the longitudinal deformations due to the moment of the normal thrust. This implies, therefore, that the initial force is independent of the creep of the concrete. Continuous series of measurements have also proved this.

Résumé

La construction des grandes voûtes en béton armé est intimement liée à la construction des échafaudages. Lorsque la nature du sol est mauvaise, il est préférable de concevoir un cintre reposant librement sur ses appuis. Exécuté en bois, il faut tenir compte d'une réduction de résistance dans le cas d'une atmosphère chargée d'humidité et lorsque le cintre reste longtemps en charge.

Les variations de forme de la voûte, dues au retrait et à la déformation plastique du béton, peuvent être déduites des constantes du matériau déterminées au laboratoire sur des éprouvettes.

Dans la construction des poutres en béton armé de grande portée, il s'est révélé avantageux de comprimer la construction par l'emploi de tirants en acier de haute qualité.

Zusammenfassung

Die Kunst, grosse Betonbogen zu erstellen, hängt mit der Ausführung der Lehrgerüste eng zusammen. Sind die Gründungsverhältnisse schwierig, so sind freitragende Bogengerüste vorzuziehen. Wenn sie aus Holz sind, muss man die zulässigen Beanspruchungen des Materials wegen der hohen Luftfeuchtigkeit und der langen Belastungszeit abmindern.

Nachdem man im Laboratorium anhand kleiner Probekörper die erforderlichen Materialkonstanten bestimmt hat, kann man die Formänderungen des Bogens in bezug auf Schwinden und Kriechen des Betons ausrechnen.

Bei Betontragwerken von grosser Spannweite hat es sich als wirtschaftlich erwiesen, mittels Zugbänder aus hochwertigem Stahl die Konstruktion mit einer Druckvorspannung zu versehen.

Summary

The task of building large concrete arches depend to a great extent on the construction of the centering. Where the nature of the ground is unsatisfactory, a freely suspended arch centering is superior to the trestle type. If it is constructed of timber, however, allowance must be made for the reduction of the strength of the timber when a high humidity of the air and a long loading period is to be reckoned with.

The deformations of the arch due to the shrinkage and creeping of the concrete may be calculated when the necessary constants for the material have been determined from small test specimens in the laboratory.

When constructing long span concrete girders, it has been found economical to compress the structure by freely suspended tie-rods of high quality steel.