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Herstellungs- und Montageprobleme

Fabrication and Erection Problems

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Réalisation du pont à haubans de Saint-Nazaire

Stahlschrägseilbrücke in Saint-Nazaire

Cable-Stayed Steel Box Girder Bridge in Saint-Nazaire

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Le pont à haubans intégré au viaduc qui franchit l'estuaire de la LOIRE entre SAINT-NAZAIRE et SAINT-BREVIN, est un ouvrage métallique de 720 m de longueur : une travée centrale de 404 m de portée flanquée de 2 travées latérales de 158 m. Il dégage un gabarit de navigation maritime de 6I m de hauteur sur une largeur de 300 m (Figure 1).
Le profil en long de la chaussée est une parabole de paramètre 6 428 m. Il supporte une chaussée de 12 m et deux trottoirs. Le tablier a une largeur hors tout de 15 m.

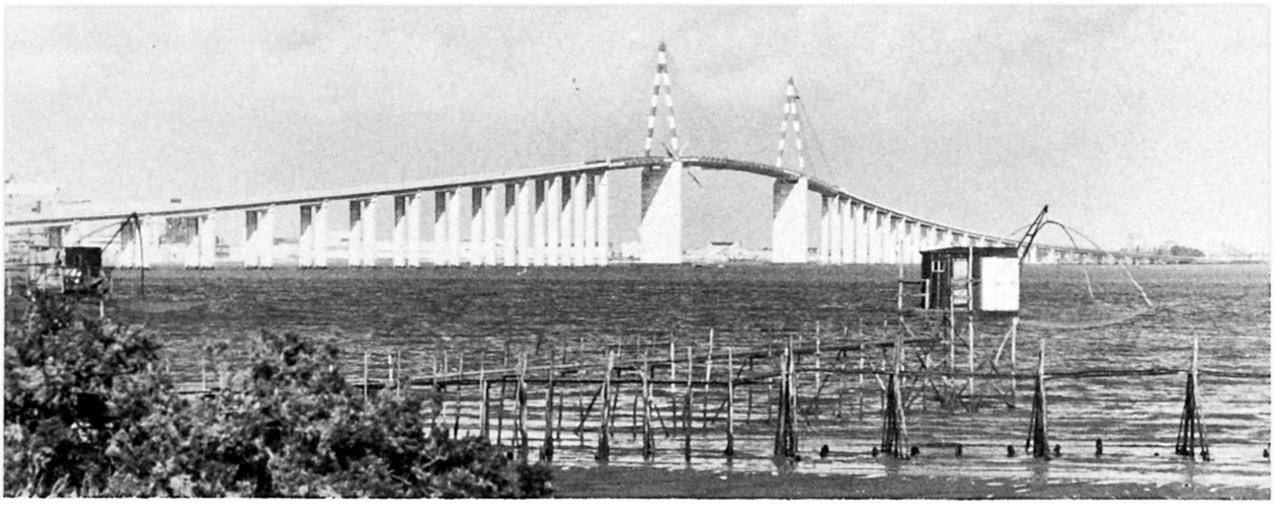


Figure 1 - Pont de SAINT-NAZAIRE

Un pont à haubans a été préféré à un pont suspendu qui aurait nécessité des massifs d'ancrage coûteux en raison de la grande hauteur des piles, environ 50 m et de la profondeur des fondations : 5 m d'eau plus 40 m de vase et sable.

Pour assurer un bon comportement aérodynamique de l'ouvrage, il a été choisi :

- un tablier en forme de caisson assez mince dont la hauteur dans l'axe de la chaussée n'est que de 3,20 m et dont les âmes sont inclinées

- des pylônes triangulaires et un haubanage fixé aux parois latérales des caissons et quasi convergent en tête des pylônes.
  Une telle architecture donne à la structure une grande rigidité de torsion.

Le développement de l'étude aérodynamique a confirmé que le pont ainsi conçu présente une vitesse critique de "flutter" très largement supérieure aux vitesses de vent possible en son site. Par contre, il a fallu le munir de déflecteurs permettant de réduire l'amplitude des oscillations verticales dues aux échappements tourbillonnaires.

La construction de l'ouvrage selon le procédé classique : montage des travées latérales en prenant appui sur des piles intermédiaires provisoires, a été jugé trop coûteux en raison de la hauteur des piles et de la profondeur des fondations. Le processus de construction retenu tire profit du fait que les travées latérales sont en site aquatique et que la C.F.E.M. possède un atelier en bord de la mer Méditerranée à FOS. Quatre tronçons de tablier, longs de 96 m y ont été construits puis acheminés à l'aide de barges marines jusqu'au port de SAINT-NAZAIRE (Figure 2). Là, assemblés deux à deux pour constituer les travées de rive et les amorces de la travée centrale du pont, ils ont été amenés sur le site et placés, en profitant de la marée, sur un système élévateur reposant sur l'embase des piles (Figure 3).

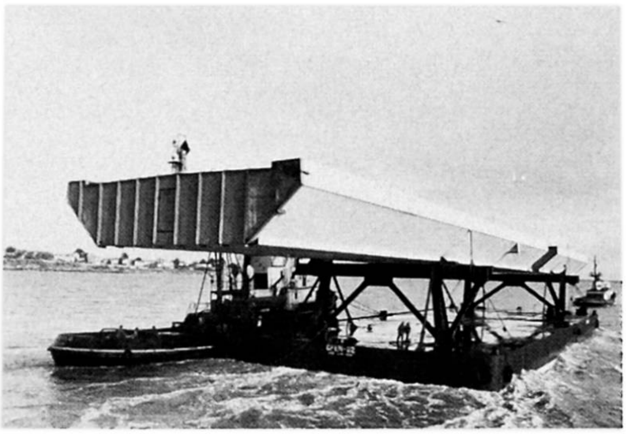


Figure 2 - Transport de tronçon de tablier

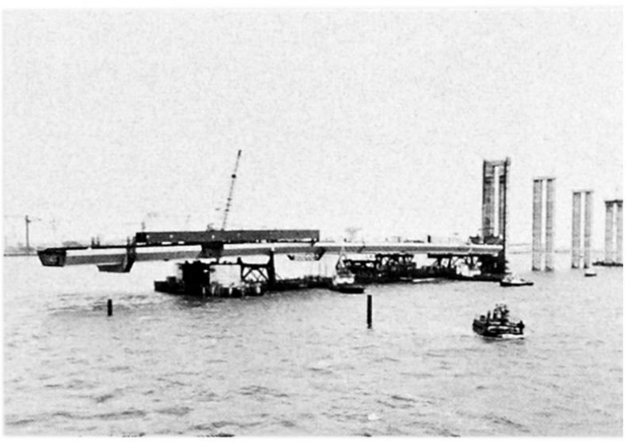


Figure 3 - Mise en place d'une travée latérale

Après construction des fûts des piles principales à l'aide de coffrages glissants, le tablier du pont a été hissé jusqu'à son niveau définitif (Figure 4). Le dispositif de hissage est analogue à celui déjà utilisé par la C.F.E.M. en 1971 lors de la construction du viaduc de MARTIGUES. Le tablier a servi de monte charge et porté le pylône couché, divers engins de levage, une baraque de chantier, etc...
Après que les pylônes aient été redressés, la travée centrale a été montée à l'avancement en porte à faux par tronçons complets de 16 m approvisionnés sous le pont et levés à l'aide d'une chèvre (Figure 5), puis assemblés au tronçon précédent par boulonnage et soudage.

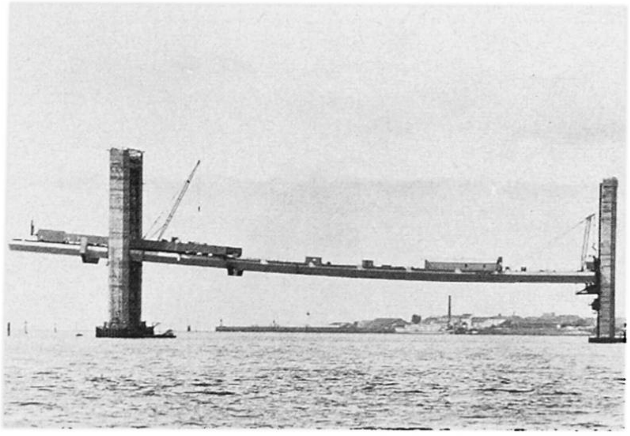


Figure 4 - Travée latérale en cours de hissage

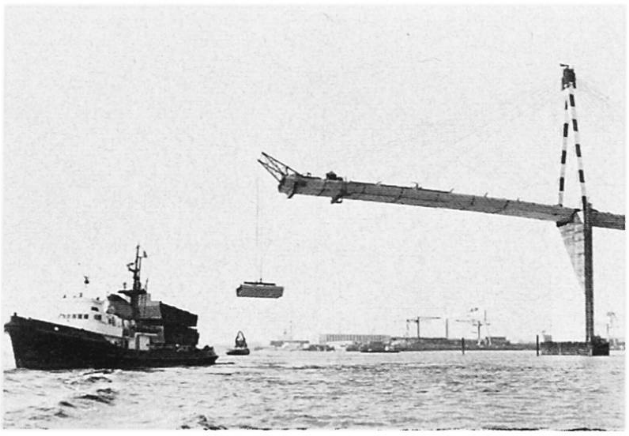


Figure 5 - Levage d'un tronçon de la travée centrale

Des aciers soudables à hautes caractéristiques mécaniques ont été utilisés en certaines parties de l'ouvrage.

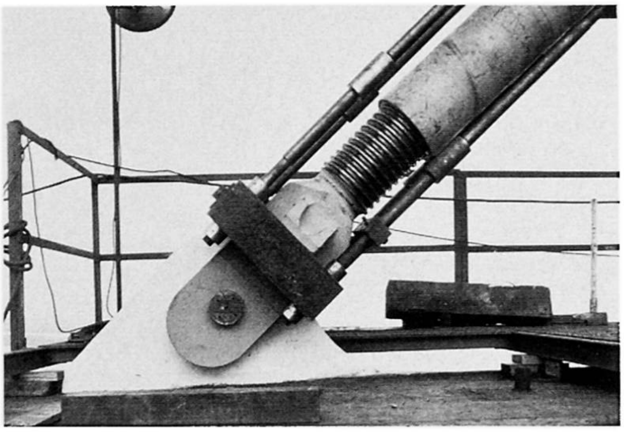
Les haubans sont fixés sur les parois latérales de la tête du pylône et sur des appendices des âmes du caisson. La fixation est faite par l'intermédiaire de chapes, d'axes et de rotules sphériques. La résistance effective de l'ouvrage dépend bien sûr, essentiellement de la bonne tenue des parois latérales de la tête du pylône et des appendices du caisson. Ces pièces assemblées à l'ossature par soudage devaient avoir une épaisseur de 80 mm ; Elles sont soumises à des contraintes d'ensemble modérées mais localement des contraintes élevées peuvent apparaître qu'aucun calcul ne peut prévoir avec précision. Il fallait les réaliser en acier très ductile ; Nous avons choisi un acier élaboré au four électrique par la Société MARREL : l'acier AMCO calmé à l'aluminium et dégazé sous vide. Ses caractéristiques sont les suivantes :

- Composition chimique (teneur en %)

0,11	C	0,16
1,45	Mm	1,65
0,10	Si	0,35
0,20	Ni	0,40
	Cv	0,25
	Mo	0,10

- Caractéristiques mécaniques

	E Travers kg/mm2	R Travers kg/mm2	A %	KCV-50° en long kg/cm2
Valeur minimale exigée	30	-	-	3,5
Moyenne des essais	37,1	53,2	31,4	16,9
Moyenne moins 2 fois l'écart type	33,7	49,8	26,7	4,5



Dispositif d'ancrage des haubans sur les caissons et
Dispositif de mise en torsion des haubans

Les chapes ont été pour les mêmes raisons réalisées en acier moulé à haute résistance. En fonction des efforts développés par les haubans, 2 familles de chapes ont été réalisées avec des aciers de caractéristiques différentes :

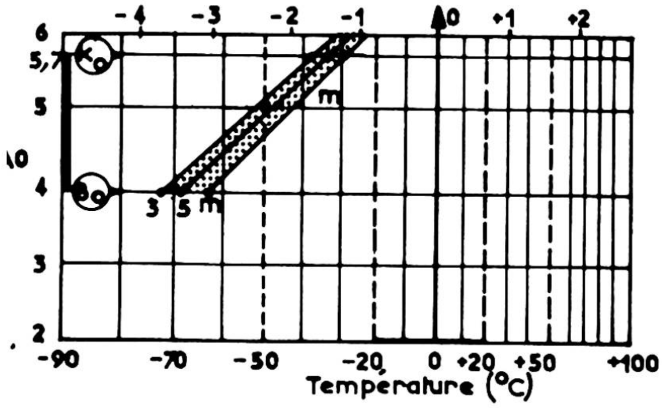
Nuance d'acier moulé	E kg/mm2	R kg/mm2	A %	KCV-20° en long kg/mm2
30 NCD 8 M				
Valeur minimale exigée	45	80	15	3,5
Moyenne des essais	75,2	87,8	19,9	11,6
Moyenne moins 2 fois l'écart type	69,9	84,1	14,8	9,6
25 MD 25 M				
Valeur minimale exigée	36	65	15	3,5
Moyenne des essais	51,9	69	22	7,9
Moyenne moins 2 fois l'écart type	47,5	64,7	17,8	4,5

En situation de hissage, les travées de rive ont été considérablement plus sollicitées qu'elles ne peuvent l'être dans l'ouvrage terminé.
Une étude a montré qu'il était plus économique de construire le fond du caisson en acier de limite d'élasticité égale à 42 kg/mm2 que d'adopter une tôle de plus forte épaisseur dont le supplément de poids aurait contribué à augmenter sensiblement les sollicitations.
Il fallait toutefois, que l'acier choisi soit soudable sans qu'il soit nécessaire de prendre de précautions coûteuses.
Une étude confiée à M. SCHNADT nous a permi de sélectionner des aciers présentant avant et après soudage des caractéristiques mécaniques en long et travers satisfaisantes. Le tableau ci-dessous résume les caractéristiques mécaniques exigées et celles obtenues lors des essais de réception des aciers DILLINAL 55/43 utilisés.

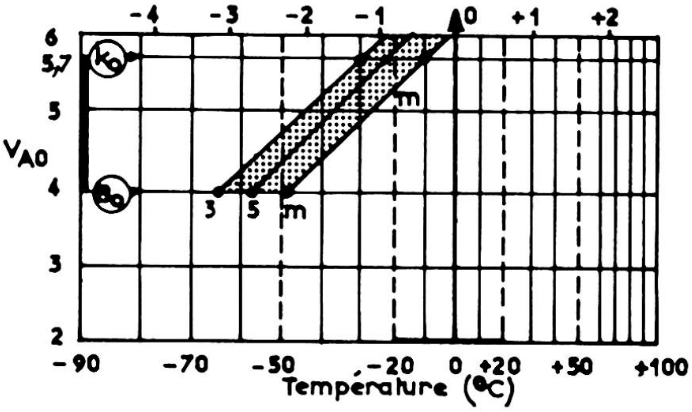
	E long kg/mm2	R long kg/mm2	A %	KCV-20° long kg/cm2	KCV-20° Travers kg/cm2
Valeur minimale exigée	43	55	19	7	4
Moyenne des essais	48	59,5	28,1	13,1	5,9
Moyenne moins 2 fois l'écart type	43,1	54,8	23,9	9,3	4,2

Les zones thermo vectomiques d'un échantillon de cet acier prélevé dans une tôle de 14 mm sont :

- en long



- en travers



Bien que ce ne soit pas nécessaire pour la résistance de l'ouvrage, il fut décidé de construire les jambes des pylônes en acier Usiten LE 43 Nb Lc. C'est un acier de composition chimique identique à celle de l'acier E 36-4 au niobium calmé à l'aluminium, mais le laminage contrôlé permet d'en relever les caractéristiques mécaniques.

	E Long kg/mm2	R Long kg/mm2	A %	KCV-20° Long kg/cm2	KCV-20° Travers kg/cm2
Valeur minimale exigée	43	55	19	7	3,5
Moyenne des essais	47,1	57,8	26,2	12,4	5,7
Moyenne moins 2 fois l'écart type	43,1	54,9	22,5	7,3	3,75

Une étude faite sur des échantillons de tôles de même composition chimique, a montré que les aciers livrés à l'état de laminage contrôlé n'étaient pas plus sensibles au vieillissement que les aciers à l'état normalisé provenant de la même coulée.

Les épaisseurs des tôles en acier DILLINAL 55/43 et en acier USITEN LE 43 Nb Lc n'excédaient pas 16 mm. Leur soudage a pu être effectué sans préchauffage selon des techniques similaires à celles utilisées pour l'acier E-36.

RESUME

Le viaduc sur la Loire entre Saint-Nazaire et Saint-Brevin comporte un pont métallique haubanné dont la travée principale a une portée de 404 m. Des aciers moulés et des aciers soudables de hautes caractéristiques mécaniques ont été utilisés pour la construction de certaines parties de l'ouvrage.

ZUSAMMENFASSUNG

Die Ueberbrückung der Loire zwischen Saint-Nazaire und Saint-Brevin besteht aus einer Stahlschrägseilbrücke, deren Hauptfeld eine 404 m lange Stützweite hat. Für die Herstellung einiger Bauteile wurden hochwertiger Stahlguss und hochfeste schweissbare Stähle benützt.

SUMMARY

The main span of the cable-stayed steel box girder bridge as a part of the viaduct joining Saint-Nazaire to Saint-Brevin over the Loire is 404 m long. Cast high strength steel and welding high strength steel have been used in building some parts of the bridge.

Application of High Strength Steels to a Long Span Truss Bridge — Osaka Port Bridge

Application des aciers à résistance à un pont à poutres en treillis de longue portée — Pont du port d'Osaka

Anwendung hochfester Stähle für eine weitgespannte Fachwerkbrücke — Osaka Hafenbrücke

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1. INTRODUCTION

The Osaka Port Bridge is a cantilever truss bridge having a total length of 980m (=235m+510m+235m), including a suspended span of 186m. This bridge has double decks with four lanes each.

The type of the bridge was determined considering location of piers, navigation channel and soil condition.

High-strength steels of $70kg/mm^2$ and $80kg/mm^2$ classes up to 75mm in thickness which had high weldability were newly produced for the construction of this bridge. Out of the total bridge weight of 40,000 tons, the following amount of the high-strength steels was used.

$80kg/mm^2$ class high-strength steel (HT80): 4,197tons
$70kg/mm^2$ class high-strength steel (HT70): 1,075tons

In the application of high-strength steels to a long-span truss bridge, due considerations should be given to design, materials, fabrication and erection.

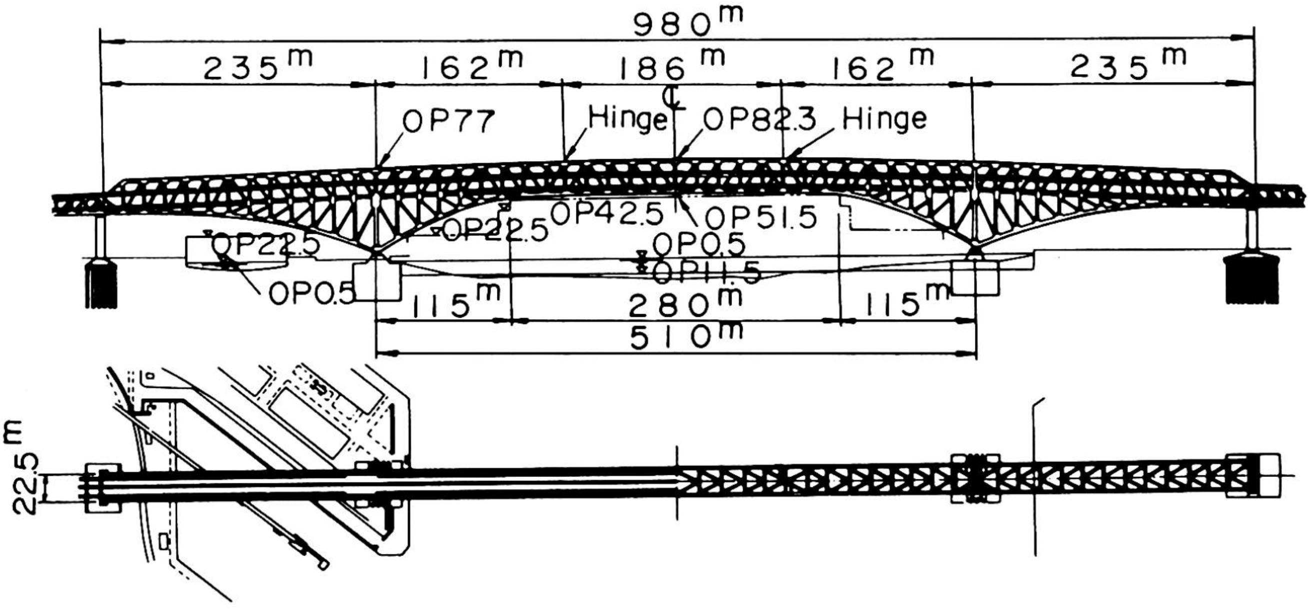

Fig.1 Osaka Port Bridge

2. DESIGN

The conventional truss bridge designs are based on an assumption that the both ends of a truss member are pin-jointed, thus causing axial force alone as section force. In recent years, however, there have been changes in the joint details of the actual truss, from the pin joint to the rivet joint and further to the high-strength bolt joint, which results in the prevention of free rotation of the members at the joints due to the increased rigidity of the connection.

In case of the Osaka Port Bridge, analysis shows that secondary stresses due to the additional bending mements and shear forces caused by connecting members rigidly at panel points are beyond negligible values.

One of the effective ways to minimize the secondary stresses is the use of members with lower flexural rigidity. But, there is a limit to this when a member is subject to higher stresses. In América, "Prebend" method is usually employed to reduce secondary stresses due to the dead load in a long span truss. In this method, however, difficulty lies in the implementation of procedure control and in the method of checking the residual stresses.

It was concluded in the design of this bridge that the secondary stresses due to the bending moments and shear forces caused at the panel points were to be analyzed and evaluated as one of the design stresses, and that thick plates with higher strength were to be used as the material for the chord members for the purpose of obtaining higher member strength without increasing the flexural rigidity of the member sections.

Box sections were employed as the chord members of this bridge. (Fig.2) The depth of each web was limited to one-tenths of the panel length, and the ratio of the sectional area of each flange to that of the entire cross section was made as low as practicable, which led to the use of 75mm thick plate of HT70 and HT80 steels as the web plates.

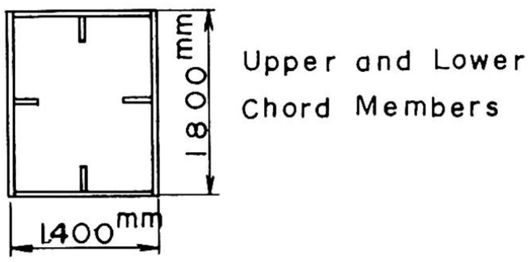

Fig.2 Sectional Form

Longer panel length can present a better proportion between the main truss height and panel length in a long truss bridge, which will result in less slenderness ratio l/r of the members and hence in less effects of the secondary stresses. Therefore, in this bridge the panel length was taken to be 18m to 19m, which is much longer than that of the conventional bridge.

3. MATERIALS

High-strength steels of 70kg/mm^2 and 80kg/mm^2 which were then available in Japan were considered to have too many unresolved problems to be applied for the construction of this bridge. In preparing a new specification it will be an important matter to our technical judgement which limit should be paid attention to - limit of steel manufacture or that of bridge fabrication. In the case of the Osaka Port Bridge, judging the matter in the balance of the above two limits, it was decided, in view of the highly advanced techniques of steel manufacturing in Japan to leave the solution of the problem to steel material wherever possible, and then requirements for the steel manufacture were set forth. In determing the mechanical properties and chemical compositions of the tempered high-strength steels to be employed for this bridge,

Table 1 Specification of HT70 and HT80 steels

	Thickness (mm)	Chemical Composition (%)						Mechanical Property					Toughness
		C	Si	Mn	P	S	Ceq	Y.P. (kg/mm^2)	T.S. (kg/mm^2)	E L. Thickness (mm)	E L. Test Specimen (JIS)	E L. %	
HT70	$6 \leqq t \leqq 50$	$\leqq 0.15$	$\leqq 0.55$	$\leqq 1.50$	$\leqq 0.03$	$\leqq 0.03$	$\leqq 0.49$	$\geqq 63$	70~85	$6 \leqq t \leqq 16$	NO.5	$\geqq 17$	$vE_{-15} \geqq 4.8^{kg \cdot m}$ (V Notch Charpy Value)
										$t > 16$	NO.5	$\geqq 23$	
	$50 < t \leqq 100$	$\leqq 0.17$	$\leqq 0.55$	$\leqq 1.50$	$\leqq 0.03$	$\leqq 0.03$	$\leqq 0.53$	$\geqq 60$	68~73	$t > 20$	NO.4	$\geqq 17$	
HT 80	$6 \leqq t \leqq 50$	$\leqq 0.14$	$\leqq 0.55$	$\leqq 1.50$	$\leqq 0.03$	$\leqq 0.03$	$\leqq 0.53$	$\geqq 70$	80~95	$6 \leqq t \leqq 16$	NO.5	$\geqq 16$	$vTrE \leqq -35°C$ (Transition Temperature)
										$t > 16$	NO 5	$\geqq 22$	
	$50 < t \leqq 100$	$\leqq 0.17$	$\leqq 0.55$	$\leqq 1.50$	$\leqq 0.03$	$\leqq 0.03$	$\leqq 0.57$	$\geqq 68$	78~93	$t > 20$	NO. 4	$\geqq 16$	

Y.P. : Yield Point T.S. : Tensile Strength E L. : Elongation JIS : Japan Industrial Standards

special considerations were given to the occurrence of cracking, and softening and embrittlement in the bond of weld joints of HT70 and HT80 steels to prevent the possible occurrence of these phenomena in practical use.

The material specification of the steels to be used for the Osaka Port Bridge was set forth in accordance with the following basic conditions; (Table 1.)

(1) The maximum plate thickness of each class of steel shall be as follows.
75mm for $60kg/mm^2$ class steel
100mm for $70kg/mm^2$ and $80kg/mm^2$ class steels
(2) The steels shall have such fracture toughness that they will not be brittle fractured at the service metal temperature of -15°C.
(3) The heat input by welding shall be determined so that the requirements in item (2) above can be met. (50KJoule/cm)
(4) The temperature of preheat is dependent on the grades and thickness of materials. As for the materials to be used for this bridge, it shall be aimed that welding of $80kg/mm^2$ class steel, 100mm in thickness with preheat temperature of 150°C and under, may not result in any weld defects.
(5) The flatness of the steel plate shall not exceed 2mm/m to maintain subsequent fabrication accuracy.

A series of tests were conducted on plate thickness, 25, 50, 75 and 100mm of both HT70 and HT80 in six major mill makers in Japan to confirm the characteristics of materials concerning these phenomena. These tests were intended to investigate chemical compositions, shape and dimension accuracy, mechanical properties, weldability and weld joint performance.

From these tests it was proved that HT70 and HT80 steels can be satisfactorily applied to this bridge. It was also found out that materials produced by the current mass production process to be used for the construction of this bridge had excellent properties.

4. FABRICATION

As a procedure test to determine welding condition, fabrication accuracy, and inspection procedure, the following tests were conducted prior to fabrication;
(1) lamellar tear test,
(2) restrained cracking test,
(3) tests on the performance of corner weld joints,

(4) tests to check residual stresses due to welding, and
(5) tests to investigate the various chracteristics of the actual members using full scale models.

Welding conditions including minimum preheat temperature, maximum preheat temperature, interpass temperature, and preheat temperature for tack welds and baking conditions of welding materials were determined from the results of these tests, under the condition that over 50KJoule/cm of heat input was in no case permitted in the fabrication of this bridge.

Major characteristic procedures employed for the fabrication of this bridge are as mentioned below. Unlike the conventional preheat control method, the following three types of preheating methods were used;
(1) electric preheating type with automatic control,
(2) fixed burner type, and
(3) manual burner type.

The separate preheating method was specified for each type of weld joints. In welding major members, symmetrical preheating and symmetrical welding methods were employed to secure higher fabrication accuracy and minimize residual stresses. To prevent the occurrence of cracking, the welding material having the strength lower than the base metal that is "soft joint" was employed. It was also specified that the fabricators should submit reports of preheat temperature, amount of heat input, amout of angular deformations in butt joints, and dimentional accuracy of members after tack welding and final welding, respectively.

As for the drilling of holes at joints, discussions were made on efficiency and accuracy of drilling, partly because the grip lengths of joints were large due to the use of thick plates and partly because the high-strength steel with high hardness was used. In this bridge, an interchangeable method in which drilling of holes was performed using a template with a bush inserted in each hole was employed, wherever practicable.

5. ERECTION

This bridge has long panel length, which resulted in increase in the weight of single members. Larger members weigh 75 to 110 tons each. This was the biggest factor in determining the erection method. Erection methods are shown in Fig. 3. Erection loads which would be resulted from the below-mentioned erection method were investigated and evaluation of stresses caused by these loads was carried out for each stage of erection.

The two panels at the tower part was fabricated into an assembly of about 540 tons which was then installed by means of 3000 tons floating crane, in order to secure accuracy as well as safety and erection time saving, since these two panels would be the reference points for the erection accuracy.

The anchor and cantilever spans were erected by cantilever method and the maximum weight of a single member at these spans was limited to 80 tons so that it could be erected by a traveller crane.

The suspended span whose length was 186m and whose weight was about 4500 tons was preassembled into one block at the shop and towed to the site by 15,000 tons capacity barge. Finally on February, 26, 1974 suspended truss span was lifted to the height of approximately 60m above sea level using lifting equipment to the final position. For this lifting it took only about 3.5 hours.

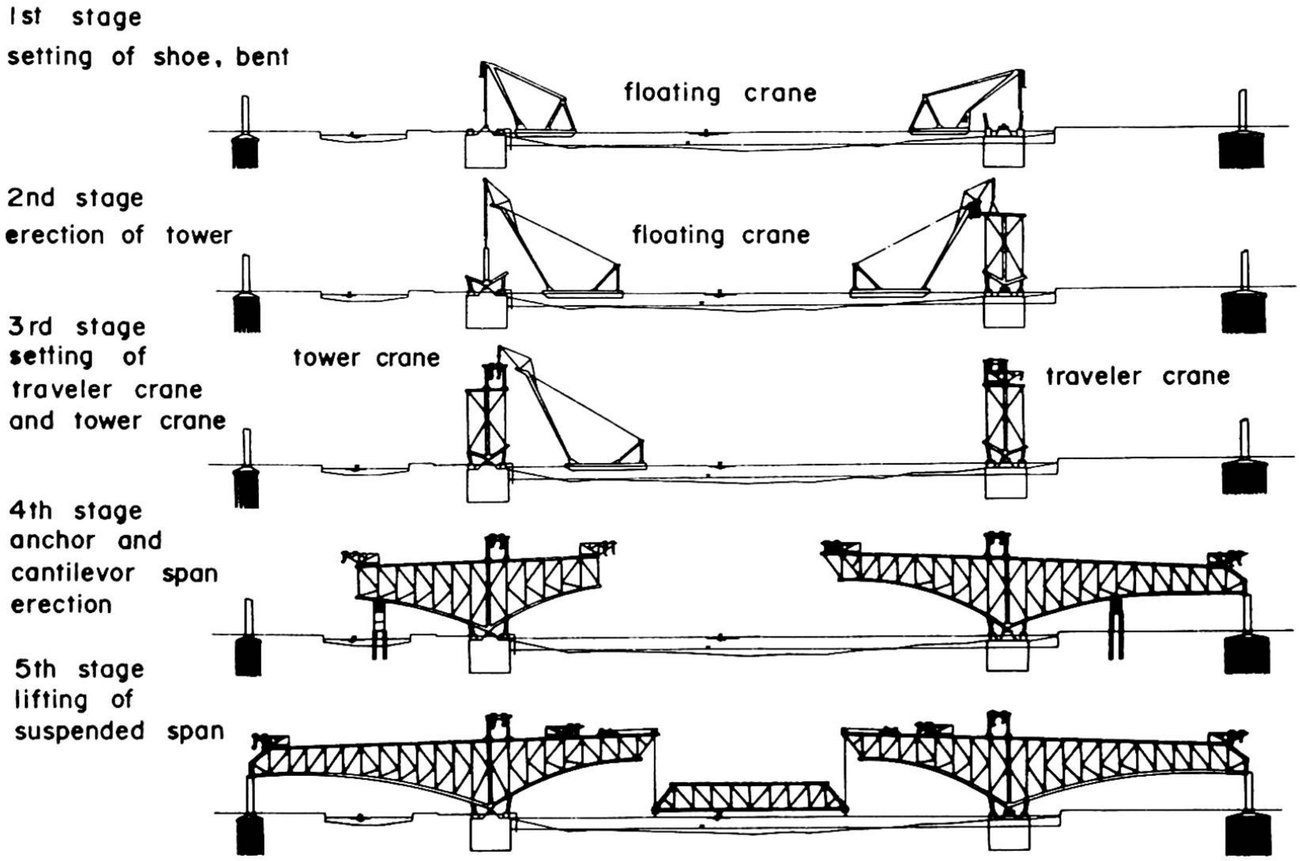

Fig.3 Erection Methods

Photo 1 Lifting of Suspended Span

SUMMARY

High-strength steel was applied in the Osaka Port Bridge after sufficient investigations. The choice of this material lead to compact cross sections and permitted to reduce the secondary stresses at panel points. This choice was confirmed by a serie of tests for different plates of HT70 and HT80 with thickness of 25, 50, 75 and 100 mm. Due attention was paid to the problem of cracking and embrittlement of welded joints during fabrication. This bridge will be ranked as the third longest modern cantilever truss bridge.

RESUME

Les aciers à haute résistance ont été appliqués au pont du port d'Osaka après des recherches approfondies. Ce choix a rendu les sections de membrures compactes et a réduit les contraintes secondaires aux points de jonction. Les matériaux ont été choisis avec soin; ils ont été mis à l'epreuve d'endurance sur des plaques de HT70 et HT80, d'épaisseur 25, 50, 75 et 100 mm. Il a été fait attention que ni fissure ni rupture de fragilité dans les joints soudés ne se produisent en cours de fabrication. Ce pont est le troisième pont moderne cantilever le plus long dans le monde.

ZUSAMMENFASSUNG

Nach verschiedenen ausführlichen Untersuchungen wurden hochfeste Stähle für die Osaka Hafenbrücke verwendet. Damit wurde es möglich, die Querschnitte einzelner Bauteile zu verkleinern, und dadurch die Nebenspannungen an den Knoten zu verringern. Bei der Fertigung wurde besonders die Vermeidung von Rissen und Sprödbruch im Bereich der Schweissstellen berücksichtigt. Für HT70 und HT80 wurde eine Serie von Eignungsprüfungen, jeweils mit Blechstärken von 25, 50, 75 und 100 mm, durchgeführt. Diese Brücke ist nun die drittlängste moderne Fachwerkbrücke mit Kragarmkonstruktion in der Welt.

Choice of Steel Quality of Steel Bridge Girders with Regard to Support Forces during Launching

Choix de l'acier de poutres de ponts métalliques en relation avec les forces d'appui lors du lancement

Wahl der Festigkeitsklasse von Brückenträgern im Zusammenhang mit den Auflagerdrückern beim Einschieben

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Large steel girder bridges in one or several spans are often erected by launching. The loading cases which occur during the launching procedure are very different from those of the final structures.

The support forces at the launching rolls are concentrated loads acting on the bottom flanges of the main girders of the bridge. During launching these loads move along the whole length of the girder, which means that very large concentrated loads are acting on the flange and an at least partly unstiffened web (as there is of course no economical possibility to arrange stiffeners in extremly small distances). It follows that there may be a risk for local deformation of the flange at the launching support combined with web crippling. This conclusion holds both for deep I-girders and box girders.

The design has to check the safety against both (a) local yielding and (b) buckling, as well as against combined influences.

Web crippling is of course included in various standard specifications and several authors have improved the solution, especially the calculations for combined influence, see e.g. [1]. The limit load is usually considered to be the one for which either the yield stress or the idealized plate buckling load is reached. As in modern steel construction more slender webs are used, the post-buckling range of a plate has to be considered. C A Granholm [2] performed full-scale tests in order to find the web crippling load and suggested that this load (with due safety factor) should be used as limit load. Further improvements [3] have led to the observation that even for girders where buckling is governing the influence of the yield point of the steel σ_y is of great importance (and not only the modulus of elasticity E), which means that the crippling load might be increased by the use of high strength steel.

The various kinds of influences – stresses and buckling – that have to be considered are shown in fig. 1 and are listed below :

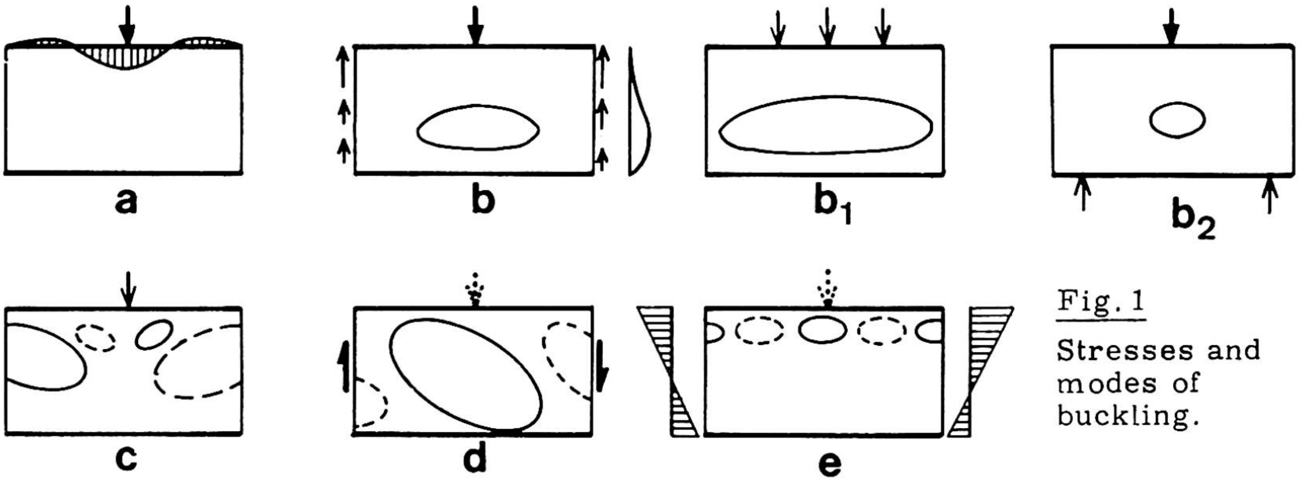


Fig. 1

Stresses and modes of buckling.

a. Local vertical stresses σ_z

b. Plate buckling because of vertical stresses

   b1. Plate buckling of a long part of the web (nearly equal to column buckling) because of distributed load or very stiff flanges

   b2. Plate buckling nearly equal to column buckling because the load is concentrated to a small part of the web due to opposite forces.

c. Shear buckling locally at the load

d. Shear buckling of the girder web

e. Buckling of the girder web due to bending stresses

It is obvious that local effects, such as a) c) and probably e) are interacting mainly with respect to local web crippling. An interaction of the effects b) and d) with respect to buckling occurs in a larger part of the web. The decrease of the bearing capacity due to effect b2) is sometimes greater than the decrease due to the effect e). This explains why it is sometimes found in tests that a large span girder can bear a greater concentrated load than a very short span girder.

Numerous tests have confirmed the obvious result that even if the bearing capacity for a web under a concentrated load mainly depends on the dimensions of the web, also the stiffness of the flange has a definite influence. Preliminary results [4] indicated an increase proportional to the variable ($1+0.4\ t/d$) when $t/d > 2$ approximately, where t/d = flange thickness/web thickness. When $t/d < \sim 2$ the tests indicated even a faster increase. Numerous test [5] in mainly the range $1.5 < t/d < 5$ confirmed the preliminary results and the following formulas are proposed

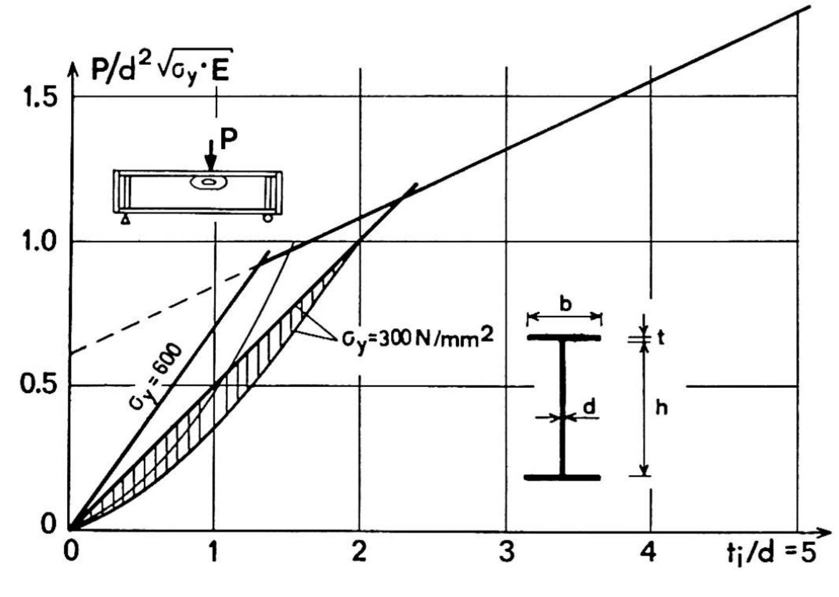


Fig. 2

Yield or crippling load according to Eq. (1).

$$\begin{cases} P^{yield}_{deform} \approx 13 \cdot \eta \cdot t \cdot d \cdot \sigma_y \quad ; \quad t/d <\sim 2 \quad ; & \ldots (1a) \\ P^{crippl.}_{failure} = 0.6 \cdot d^2 \sqrt{\sigma_y \cdot E}\,(1+0.4 \cdot t/d) & \ldots (1b) \end{cases}$$

t/d	= 0.5	1.0	1.5	2
η	= 0.55	0.65	0.85	1

Here P^{yield}_{deform} indicate a load above which the deformation begins to increase rather rapidly. If the reduction factor η is omitted (that is changed to 1 or in a certain range a little more) the transformed Eq. (1a) gives a theoretical yield failure load. It is seen that not only the yield load but also the crippling load (combined yielding and buckling) depend on the yield point σ_y, but to a different extent.

A theoretical deduction of Eq. (1a) is given in [4], and the corresponding failure load (inserting $\eta \approx 1$ in Eq. (1a)) is deduced in [5]. Both these deductions are based on an approximate model where the flange is regarded as a beam on an elastic spring bed (the bed being the web). More accurate results may be obtained by existing programmes for computations with the finite element method. A closed formula, however, is preferrable for the designer.

The simple formulae (1a) and (1b) hold for a flange width-to-thickness ratio $b/t \approx 25$. If the deviation from the value is great or if the flange is not rectangular, t in Eq. (1a), (1b) should be replaced by t_i from Eq. (2). The web "slenderness ratio" h/d is assumed to be of the order 200 but even great deviations from this value influence the coefficient only very little. If the load is not absolutely concentrated but distributed over the length c the bearing capacity is increased by multiplying with a coefficient f (c) the value of which is in principle given by Eq. (3), up to a maximum of $f(c) \approx 1.3$.

$$t_i = \sqrt[4]{\frac{12}{25}\cdot I_{fl}} \approx t\sqrt[4]{b/25\cdot t} \quad \ldots (2); \qquad f(c) = \frac{\gamma}{1-e^{-\gamma}\cdot\cos\gamma}\,, \quad \text{where } \gamma = c/2L \text{ and } L \approx 6.7\cdot t \quad \ldots (3)$$

Eq.(1a) corresponds mainly to influence of the kind shown in fig. 1a, while Eq.(1b) includes also influences as shown in fig. 1c and 1b. To this ought to be added influences as shown by fig. 1e, but the tests indicate that this does not give any reduction for bending moments M smaller than 0.6 M_f.

The failure moment M_f is to be calculated with regard to the reduction caused by web buckling. This reduction may be performed e.g. following a simplified method given in [6]. In this method the compressed part of the web is taken into account only by the portion $s = 1.95\cdot d\cdot\sqrt{E/\sigma}$ and from this portion 2/3 are placed near the neutral axis and 1/3 is placed near the compressed flange.

As an example the bridge in fig. 3 will be discussed, which was to some extent damaged during launching. The bridge has a length of about 200 m in 6 spans. A part of the bridge of somewhat more than 3 spans (ca 100 m) was erected by launching in order not to disturb the traffic on a number of under-

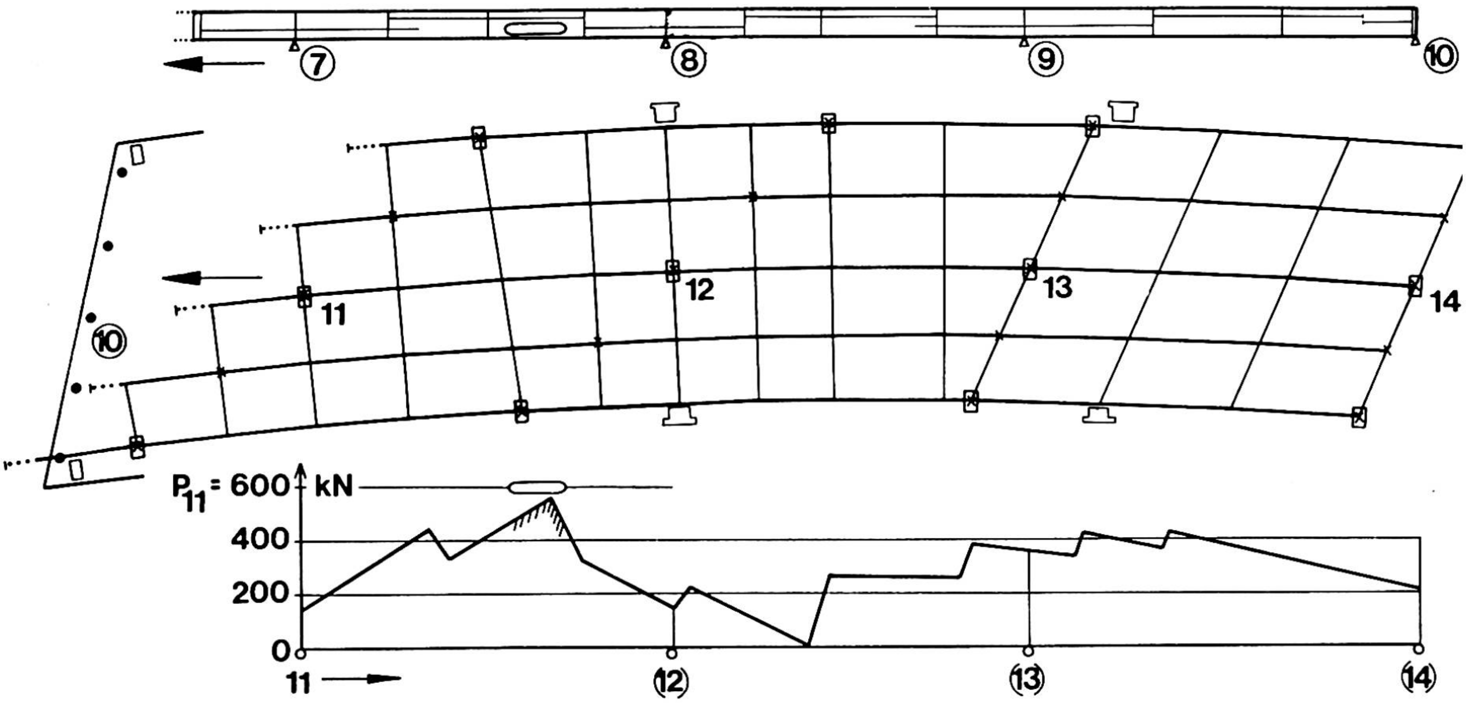
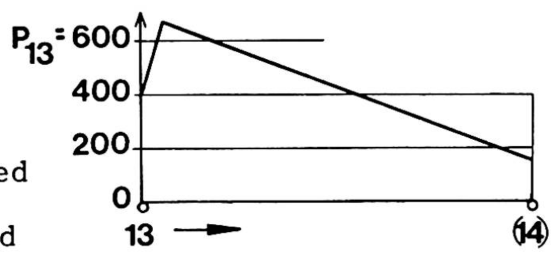


Fig. 3

Steel girder bridge erected by launching.
On top : The central main girder of the launched part (supports 7 to 10) and plan.
Diagrams : Variation of support forces P_{11} and P_{13} during launching.



lying railways. The supports had to be placed rather irregularly because of the railway tracks and as the bridge was slightly curved horizontally, it was very laborious to perform a complete analysis of all support forces during the various launching stages. The bridge has 5 main girders and in the final position of the permanent structure all girders are supported at every support line (the launching rolls were 2-roll bogies). In the provisional bridge bed from where the launching started, there were only 3 launching rolls in every support line (using single rolls). The reason for the different roll arrangements was of course the great risks if something unforeseen happened when launching above the railway lines.

Before the launching the supporting forces were only roughly checked mainly using the method of [1]. During launching a buckle was observed in the web in a region of the central main girder web, which is marked with an oval in the girder in fig. 3. Because of the buckle a computation of the supporting force was made afterwards for every roll (or bogie) for every stage of the launching. Here only two diagrams are shown of which one gives a reason for the buckle. (The other one gives a greater max. load but there is no risk for the web because of the thick flange).

Fig. 3 shows the girder of the bridge laying on the starting bed before the launching. The provisional supporting rolls are marked with squares and the positions of the rolls over the permanent supports are marked with filled circles (only these of the first permanent support line are seen quite to the left). The diagrams of the supporting loads give the load at the start in the utmost left and then the variation as the launching goes on.

The dangerous point, where also the buckle was afterwards observed, is marked with an oval in the force diagram for P_{11} in Fig. 3. In this section t/d was 16 mm/12 mm and Eq. (1a) is governing. The steel quality of the bridge girder had σ_y = 260 N/mm^2. This gives P^{yield}_{deform} $= 13 \cdot 0.8 \cdot 16 \cdot 12 \cdot 260$ N = 520kN = = 52 Mp.

The supporting force was 56 Mp in this point as seen in the diagram in fig. 3 and thus larger than the load giving large deformations. The force was, however, less than the failure load (where $\eta \approx 1$) which is $P_{failure} \approx 65$ Mp. At such high loads, the deformations ought to be very large, and the relatively stiff transverse girders would distribute the supporting force to the neighbouring girders. Even with such a load distribution the margin to the failure load was small.

After the launching a part of the flange and the web at the deformed region was cut out and replaced. Inspection showed that the web had small deformations indicating incipient web crippling at some other locations too, and there some small adjustments only were undertaken before the bridge was taken into traffic.

— When the concentrated load is too great there are several possibilities. As already mentioned one can use launching rolls designed with bogies, one can distribute the load by arranging a crane rail below the flange, or one can use a thicker web or use a higher quality for the steel of the web.

For t/d < 2, where Eq. (1a) is valid, the advantages of the two latter solutions are immediate apparent. For t/d > 2; and Eq. (1b), the best illustration is given by an example. For the bridge just described there were parts having t/d = 35/12 and in order to compare the web thickness neeeded for two steels with σ_y = 260 and 400 N/mm^2 respectively, you use Eq. (1b) :

$$0.6 \cdot 12^2 \sqrt{400\ E}\,(1+0.4 \cdot \frac{35}{12}) = 0.6 \cdot d_1^2 \sqrt{260\ E}\,(1+0.4 \cdot \frac{35}{d_1})$$

From this we find d_1 = 13.9 mm, that is 16 % thicker than the 12 mm web. If the difference in steel prize is less than 16 % it is thus cheaper to use the steel of higher quality.

*

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SUMMARY

A formula is given for the support forces due to launching that a steel girder can resist. The formula is applied to a bridge with damaged (crippled) web. The use of high strength steel is discussed.

RESUME

Une formule est donnée montrant la force d'appui qu'une poutre peut supporter pendant une construction par lancement. La formule est appliquée à un pont où l'âme est voilée. L'influence de différentes qualités d'acier est discutée.

ZUSAMMENFASSUNG

Es wird eine Formel angegeben für den Auflagerdruck, den ein Stahlträger während des Einschiebens aufzunehmen vermag. Diese Formel wird für eine bestimmte Brücke angewendet, deren Steg beschädigt wurde (Beulen). Die Verwendung von hochwertigem Stahl wird diskutiert.