

The Lully Viaduct, a Composite Bridge with Steel Tube Truss

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ABSTRACT

The innovative design of the Lully viaduct proposes a light and transparent structure made of a triangular cross-section fabricated entirely from unstiffened circular tubes. The result is twin space trusses, with a typical span of 42.75 m. Each transversal triangular cross-section is 2.9 m high and 4.0 m wide, and is supported by a single slender pier. The largest diameters and thickness of the tubes are over 500 mm and nearly 70 mm respectively. One major difficulty during the design of the connections was to define the stress distribution along the complex intersecting perimeters of the tubes, and to calculate the hot spot stresses. Geometry calculations, precision cutting and edge preparation of the tubes were necessary for performing full penetration welds. The mobile formwork, which also ensures the stability of the tubular trusses during concrete pouring, also required special consideration.

This paper describes the evolution of the project, from design, truss fabrication and welding, to construction on site.

KEYWORDS

Aesthetic, bridge, composite bridge, space truss, circular hollow tubes, fatigue, stress distribution, stress concentration factor, hot spot stress, weld.

INTRODUCTION

Located on the Swiss highway A1, the 1000 m long Lully viaduct, made of space tubular trusses, is the result of an engineer design contest. This project was chosen by the jury for its originality and aesthetic quality. In the past, no one dared to build a road bridge with welded tubular nodes due to the dynamic stress.

DESIGN CONTEST

Located near the village of Lully in the Canton of Fribourg, the viaduct is incorporated into highway A1 running from the East to West of Switzerland. Crossing a rural flat valley surrounded by wet land and trees, this bridge will complete a highway link between Murten and Yverdon. The owner, the Fribourg Cantonal Highway Office, had to choose between 3 projects submitted by selected experienced consulting firms.

The participants had to respect the following conditions:

- Total bridge length : approximately 1'000 m
- Width of the bridge deck : from 13.25 m to 16.00 m in each traffic direction
- For maintenance reasons, it was decided to build two separate roadways
- Longitudinal inclination : between 2.9 and 3.6 % in a concave circular arc with a radius of 40'000 m
- Horizontal curve : circle of 3'000 m between 2 transition radius
- Height over the valley : between 4 and 15 m.

The following projects were submitted to the 5 members of the jury :

- One prestressed concrete box girder with an average span of 44.60 m and a constant depth of 2.50 m.
- One prestressed concrete box girder with an average span of 42.50 m. This girder depth varied between 2 and 2.45 m.
- One composite space truss with an average span of 42.75 m and a constant height of 3.75 m.

The third project was recommended by the jury for its “lightness” and transparency allowing it to integrate into the countryside (Fig 1). The recommendation was approved by the responsible authority who accepted the challenge of innovative design. This project was presented by the consulting engineers group DIC-DMA (Dauner Ingénieurs Conseils SA and Devaud, Mongatti et Associées SA).

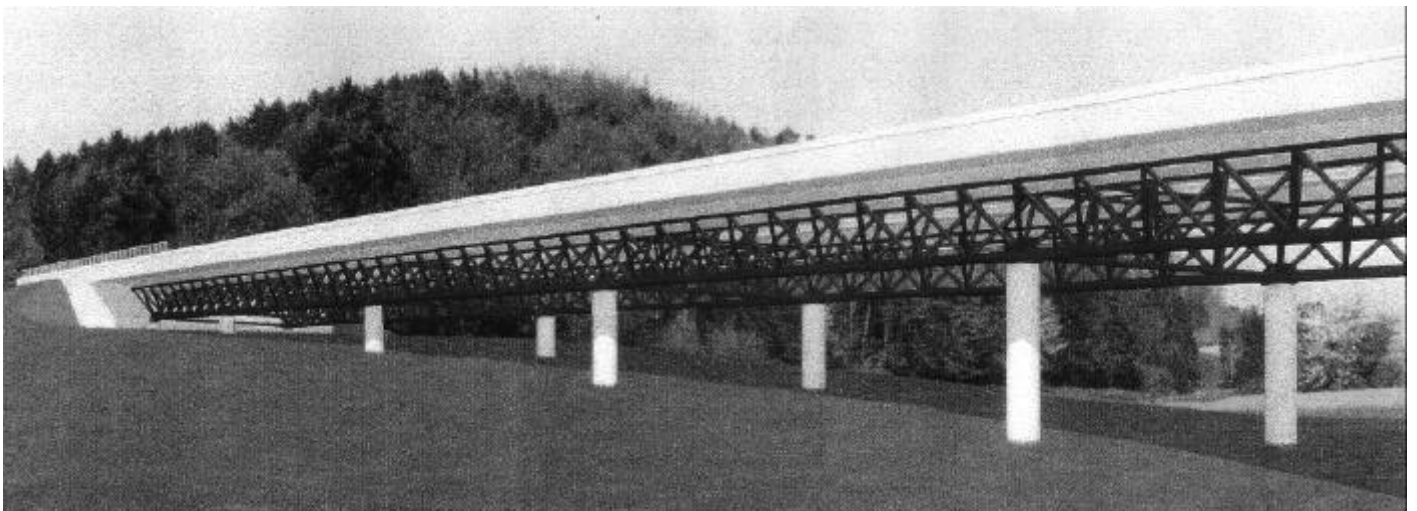


Fig 1 General view of the tubular composite bridge (photomontage)

CONCEPTUAL DESIGN

The shape of the surrounding trees inspired the author for the conceptual design [1]. A certain analogy can be made with the steel construction where the elements size can be easily changed and adapted to live load capacity [2].

Three different cross sections were considered :

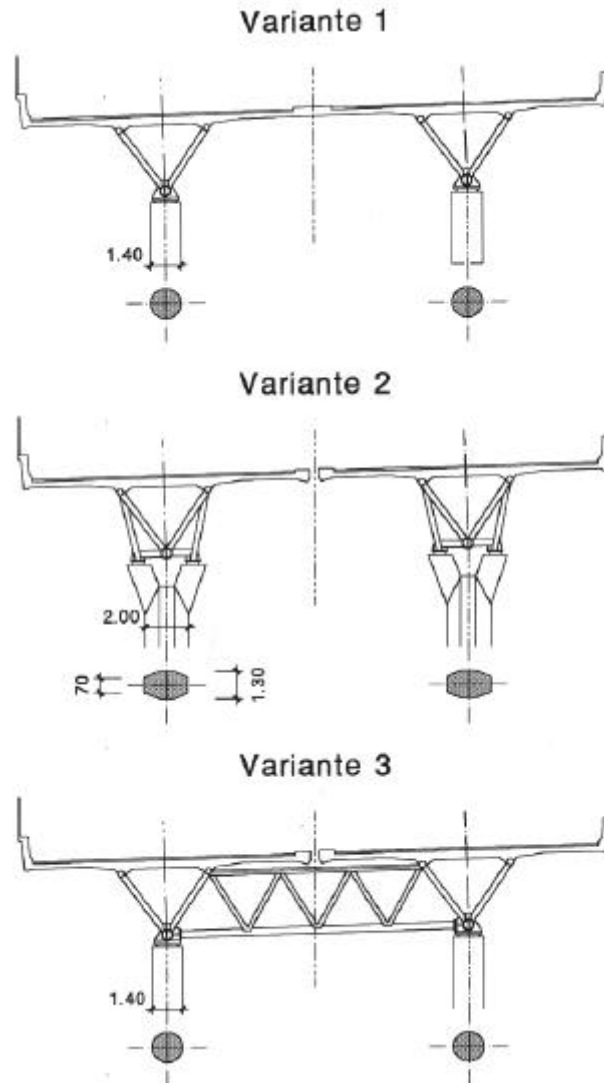


Fig. 2 Cross section – design evolution

The first alternative was the ideal design (Fig.2). It has slim cylindrical piers without bracing, but did not respect the maintenance condition of two separated roadways. The third solution has been design to avoid having piers dominating the surrounding trees. This included perpendicular truss connecting the two longitudinal girders at the cross section on the piers. The result was a three-dimensional tubular truss supporting structure.

PROJECT DESCRIPTION

Geometry

The dimensions of the trusses were based on equilateral triangles. Compared to a traditional box girder, the truss depth is 50% higher. The slenderness (L/H) of this lightweight superstructure is approximately 13 instead of 20 as for regular beam girder (Fig 3).

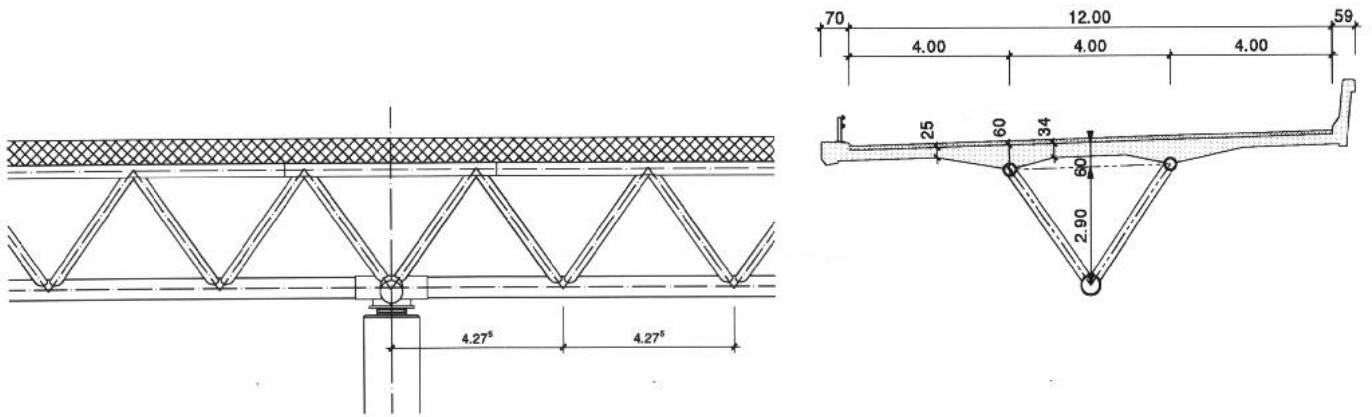


Fig. 3 Longitudinal view and standard cross-section

The truss geometry was determined first by considering the span length as well as the maximum transportable element. Then the tube diameters were given by preliminary calculations and the following considerations:

The diagonals governs the size of other members. Preliminary analysis leads to a diagonal diameter of 267 mm and wall-thickness between 11 and 50 mm. Adding to this the geometric conditions at the nodes, the smallest diameter for the lower chord was 508 mm.

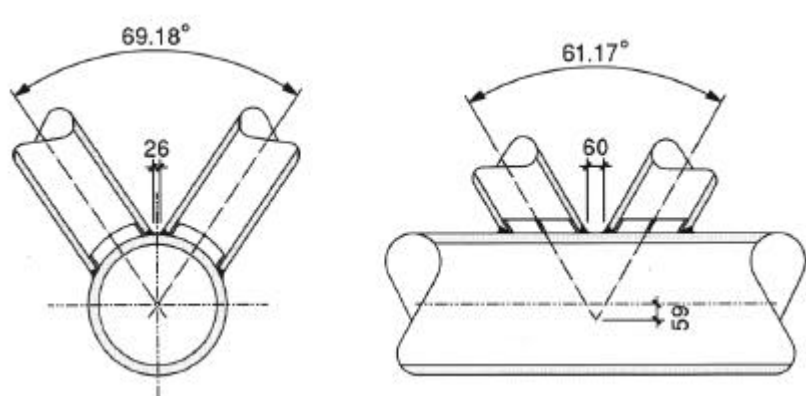


Fig. 4 KK-shaped joints geometry (lower chord)

It was beneficial to use the smallest possible tube in order to improve the force transfer between diagonals due to the thicker walls. The thickness of the lower chord tubes varied between 25 and 50 mm (Fig 4). In the support zone the thickness is increased from 50 to 70 mm. The thicker walled tube has a diameter of 559 mm and a length of 2 m centered on the bearing. Unsightly stiffeners could therefore be avoided.

The upper chords nodes are less complicated (K-shaped joint) (Fig 5). The choice of the tube size depended more on the considerations below than on the actual forces, which were carried mostly by the slab in the final stage.

1. Nodal forces had to be equilibrated without taking the concrete resistance into account. Overlapping of the diagonals was therefore chosen in order to transfer some of the vertical force directly between the diagonals.
2. The tube diameter had to be large enough to provide adequate space for the welded shear connectors and to allow minimum concrete cover.

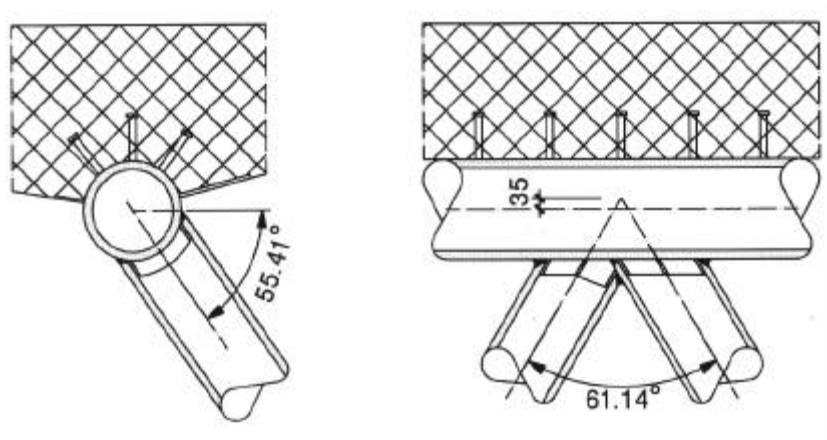


Fig. 5 K-shaped joint geometry (upper chord)

One of the difficulties encountered in the fabrication of the space tubular structure was the welding at the intersecting perimeters of the tubes and checking the penetration at the weld root. Even if the fatigue requirements are not too severe in road bridge construction, the fear of uncontrolled or bad quality weld roots in the tension tubes is understandable. For this reason, welding on backing shells was adopted. This increased the width of the inner weld toe.

The brace truss at the piers were built with smaller tubes (Fig 6 and 7), as they are subject to lower forces. The diameter of their chords varied between 219.1 and 323.9 mm, all diagonals are made of 168.3 mm diameter tube.



Fig 6 and 7 Brace truss during erection and after completion

The deck width varied between 12.0 and 14.65 m. In order to limit the long-term deflection of the relatively wide cantilever wings (4.0 to 5.33 m) and to minimize the weight of the deck, transversal tendons (600 mm^2) were used. The concrete deck was prestressed longitudinally with the same type of tendons. Longitudinally the prestressing force has been chosen to insure compression in every section under dead load (concrete and road surface).

Design

The bridge was designed according to the Swiss standard SIA 160.

Two different structural models were used for the calculation of the space truss:

1. Hinged diagonals with continuous chords for structural safety design
2. Rigid nodes for fatigue and serviceability design

The bridge deck had to be converted into a plane grid model (Vierendeel), made of two longitudinal chord and vertical members placed at the intersection with the diagonals of the space truss (Fig 8).

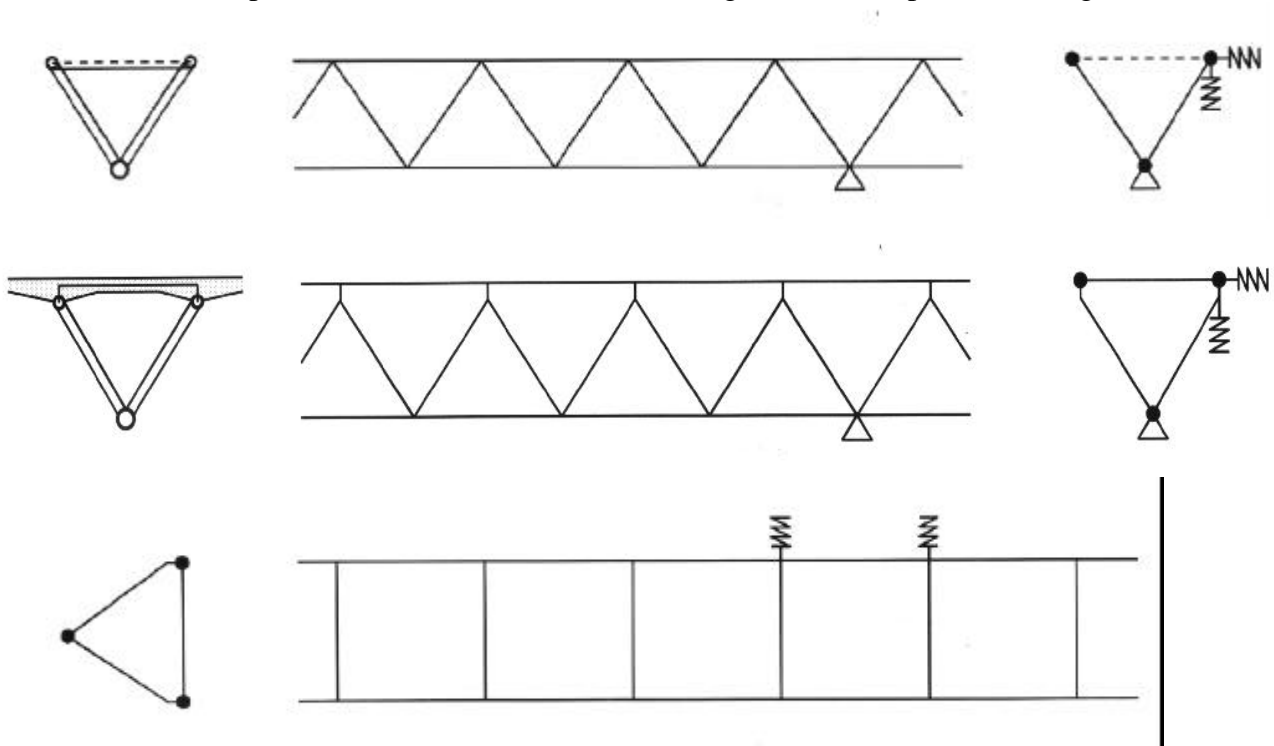


Fig 8 Space truss model for static analysis during erection and final stage

Equivalent displacement theory was used in order to determine the stiffness of the longitudinal chord. Deck stiffness, composite effect with upper chord and Saint-Venant torque resistance were considered in the vertical member properties.

Joints

The structural safety of the members and joints was verified with the internal forces of the hinged model (discussed in [3]).

No standard recommendations could be found to check fatigue resistance, or an empirical method for computing local stresses at the circular hollow tube intersection, nor a fatigue category considering the backing shell.

The following criteria were adopted after discussion with the owner and the expert engineers

1. Fatigue load according to SIA 160
2. Calculations of internal forces with
 - full and cracked composite section (with $n = E_{\text{steel}}/E_{\text{concrete}} = 10$)
 - rigid nodes

Fatigue

Fatigue effects are very important for this spatial tubular structure, especially at the welded joints. The comparison between stress range and fatigue resistance is given by the well known formula :

$$\Delta sE = \alpha \cdot \Delta s_{fat} \leq \frac{\Delta sR}{\gamma_R}$$

where Δs_{fat} : stress range under traffic loads
 α : reduction factor for traffic loads
 Δs_R : fatigue resistance for the considered detail
 γ_R : fatigue resistance factor

The problem specific to this bridge lays in the determination of the local stress range at the joints under traffic load. It involves complicated analyses taking into consideration :

- the geometry of the connection
- the diameters of the elements
- the width of the tube

All these factors are included in a "Stress Concentration Factor" (SCF), which multiplies the nominal stress s_N in the diagonal. It attempts to include both global geometrical stress concentration as well as the local stress concentration associated with the weld.

$$\Delta s_{fat} = SCF \cdot \Delta s_N$$

with $\Delta s_N = \Delta N/A$ nominal stress (without bending effects)

The stress concentration factor compares the hot spot stress at a joint with the nominal stress in the diagonal. The SCF is the ratio of the local stress at the weld toe to the nominal stress in the brace.

$$SCF = \frac{s_{HS}}{s_N}$$

with s_N : Nominal stress
 s_{HS} : Hot spot stress
SCF : Stress concentration factor

The SCF factor was obtained by comparing 3 different methods :

- empirical formulas
- finite element analysis
- stress measurement with strain gauges

Empirical formulas

Equations for the distribution of SCF around the intersection of tubular K joint can be found in [7], Fig 9.

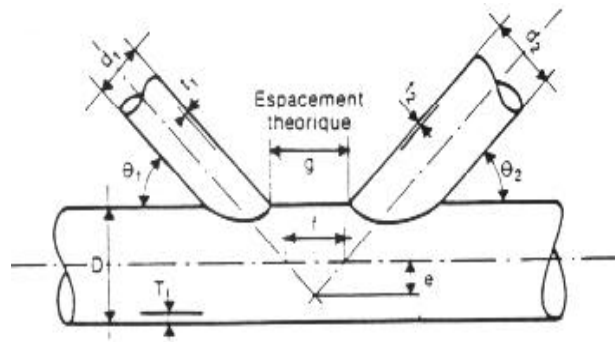


Fig 9 Geometry of a K-node

In the chord

$$SCF_K = 1,506 \beta^{-0,059} \gamma^{0,666} t^{1,104} \zeta^{0,067} \sin\theta^{1,521}$$

In the diagonal

$$SCF_K = 0,920 \beta^{-0,441} \gamma^{0,157} t^{0,560} \zeta^{0,058} e^{1,448 \cdot \sin\theta}$$

with

$$\mathbf{b} = \frac{d}{D} \quad \mathbf{g} = \frac{D}{2T} \quad \mathbf{t} = \frac{t}{T} \quad \mathbf{z} = \frac{g}{D}$$

No equations currently exist for stress distribution in KK joints [4]. For this type of connection the stress concentration factor is higher and a corrected factor γ was applied to the K-joints stress concentration factor.

$$SCF_{KK} = \gamma \cdot SCF_K$$

with $\gamma = 1,5$ for chords
 $\gamma = 1,1$ for diagonals

This hypothesis was verified by finite element analysis.

Finite Element Analysis

Elastic finite element analysis ([8] and [9]) was performed to study the distributions of stresses along the joint intersection. and through the thickness of the chord using thin shell element with no weld modelling. In this analysis the SCFs were calculated from the maximum principal stresses at the mid-surface intersection.

For the K joints between diagonals and upper chords, the results of the empirical formula were consistent with the FE analysis (Fig 10).

In the KK-joints the empirical formula results were consistent with the finite element analysis, especially for the diagonals. The stress distribution in the chords given by the FE model, gives a lower hot spot value for the circular tubes than for the rectangular tubes. Therefore the factor γ could be reduced (Fig 10).

These results have to be confirmed on the full scale structure, using the strain gauges.

<i>Type of joint</i>	<i>Element</i>	<i>Solicitation + Tension - Compression</i>	<i>SCF empirical</i>	<i>SCF FEA model</i>	<i>D (%)</i>	<i>?</i>
K	Diagonal	-	3,3	3,1	6 %	
		+	2,9	2,8	4 %	
	Chord	-	2,6	2,0	30 %	
		+	2,0	1,9	5 %	
KK	Diagonal	-	4,5	3,4		1.10
		+	4,2	3,2		1.14
	Chord	-	3,9	1,6		0.80
		+	3,4	1,6		0.84

Fig 10. Comparison between empirical formula and finite element analysis

<i>Type of joint</i>	<i>Element</i>	<i>Solicitation + Tension - Compression</i>	<i>SCF empirical</i>	<i>SCF FEA model</i>	<i>D (%)</i>
K	Diagonal	-	3,3	3,1	-6 %
		+	2,9	2,8	-4 %
	Chord	-	2,6	2,0	-30 %
		+	2,0	1,9	-5 %

Fig 10a Comparison between SCF empirical and FEA model for circular hollow sections and K joint

<i>Type of joint</i>	<i>Element</i>	<i>Solicitation + Tension - Compression</i>	<i>SCF FEA model K</i>	<i>SCF FEA model KK</i>	<i>? FEA</i>
KK	Diagonal	-	3.1	3,4	1.10
		+	2.8	3,2	1.14
	Chord	-	2.0	1,6	0.80
		+	1.9	1,6	0.84

Fig 10b Comparison between SCF FEA model for K and KK joint

Strain gauges measurement

To study the actual behaviour of the structure, the Fribourg Cantonal Highway Administration decided to instrument a bridge span with about 100 strain gauges. They were placed in order to measure the nominal stresses in the diagonals, and the stress distribution around the joints. Some gauges were also placed inside the tubes in order to obtain the stress distribution on both sides of the tube wall.

We are currently waiting for the results of stress measurement in order to compare the empirical approach and the finite element analysis.

Fabrication

The “heart and soul” of this bridge lay in the cutting edge diagonal facilities (Fig 11). The fabrication would not have been possible without this equipment which cuts tubes lengthwise and prepares contact and welding surfaces (Fig 12).

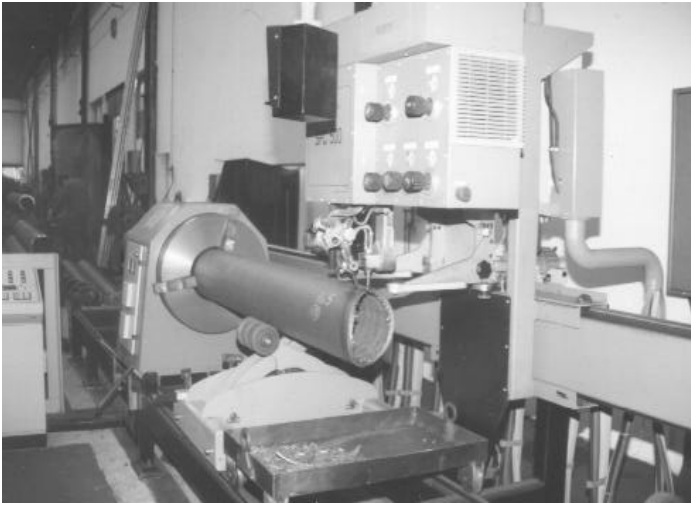


Fig 11. Hollow tubes cutting process

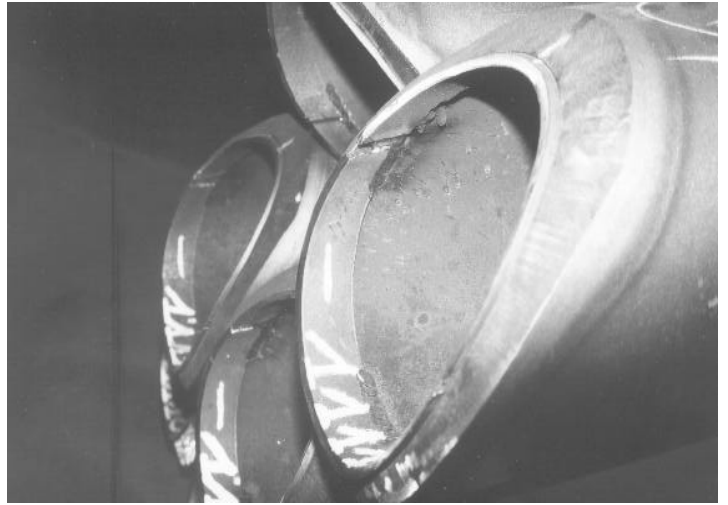


Fig 12. Diagonals with cut edges and backing shell

The truss was assembled on a frame enabling each segment length to be adapted according to the complex geometry, taking into account the camber, the horizontal transition radius and the vertical circle. Two adjacent sections could be preassembled on this frame. Diagonals joints were then welded manually and the position of the entire section could be adapted to suit the welding conditions.

On site construction

Every pier stands on 4 bored piles at depths of 12 to 20 m. After the completion of the western abutment and of 8 piers, the erection of the steel space truss could take place. The truss was assembled with crane trucks from ground level. The truss sections weighted between 25 and 35 tons. These were delivered on low bed trucks in lengths of 19 and 24 m. The building speed was of one full span every 2 weeks.

After positioning the truss up to the seventh pier, the mobile formwork was installed at the western abutment. Every stage was half a span long and took one week. The form and reinforcement placement added to the concrete pumping and hardening was moving at the same speed as the truss erection. The prestress of the transversal and longitudinal tendons was applied just after the form removal.



Fig 13. The Viaduct of Lully – lightness and transparency

CONCLUSIONS

The Lully viaduct using triangular tube-truss is a novelty in road bridge construction (Fig 13). Special thanks are due to all the companies who were involved for their contribution to this high quality bridge. No major problem occurred during the construction and some valuable experiences were gathered in tube truss assembly and construction of variable width deck. The analysis of stress measurement will be beneficial to the knowledge of the fatigue strength of KK shaped joint. Combined with new research in fatigue, it will certainly promote the use of tubes for road bridge structure. Let us hope that enough financial resources will be found in order to enable a real breakthrough in modern bridge construction.

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