# Fabrication and erection

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# The Antrenas Tubular Arch Bridge

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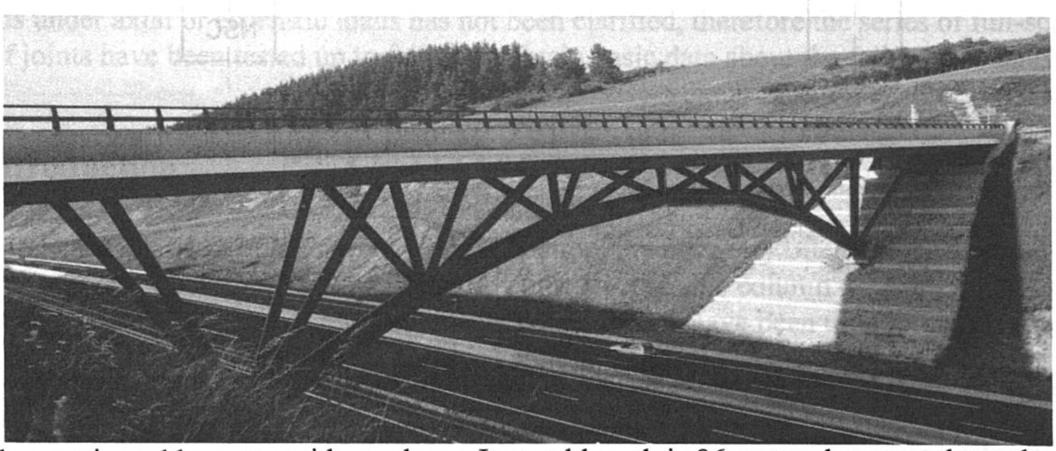
## 1. Presentation

Antrenas bridge is original and innovative. The tubular steel skeleton is an arch connected to a prestressed concrete slab to achieve a composite structure which is also a truss. Steel arch principal tube is filled with reinforced concrete, which again creates local composite elements.

The bridge is set in a remarkable landscape where the A75 motorway crosses the Gévaudan. The A75 cuts a dissymetrical gap through the granite ridge to a depth of about 15 metres for a width of 85 metres at the top. The western slope of A75 is one metre higher than the eastern slope.



The bridge carries a 11 metres wide roadway. Its total length is 86 metres between the end support axes. It is designed to give the passage to exceptionally heavy 110-ton trucks.

The bridge alignment is straight; it is perpendicular to the motorway axis. The tube keeps a 4.85 metres height clearance for traffic over the entire width of the motorway as well as over the access road. Its longitudinal profile has a one per cent slope.

## 2. General design

The bridge consists of a single tubular steel arch with a 56 metres long span, whose overall shape is parabolic, with a mean radius of approximately 60 metres. The deck is a ribbed concrete slab. Two longitudinal concrete ribs are supported by steel struts joining arch and deck.

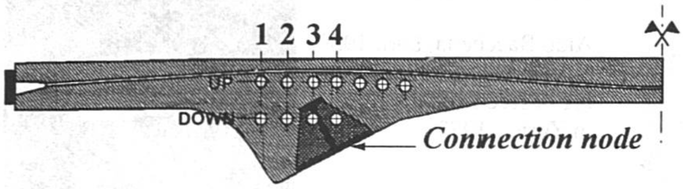
The steel tube of the arch has a circular section with a 1.2-metre diameter. Its sheet is 32 millimetres thick. This tube is located in the transversal symmetry plan of the structure, and rests on foundation blocks embedded in the slopes.

A steel propping system was developed to support the formwork for concreting the deck. Transferring the load from the temporary steel scaffolding props to the steel arch was a delicate operation requiring various jacking sequences.

To compensate the shortening effects of the arch under permanent loads, it was decided to jack it 25 millimetres at each of its springs. This reduced horizontal slipping thrusts in the punctual connection nodes.

The deck has a total width of 11 metres. The slab has a mean thickness of approximately 35 centimetres. At the ribs the maximum thickness is 95 centimetres.

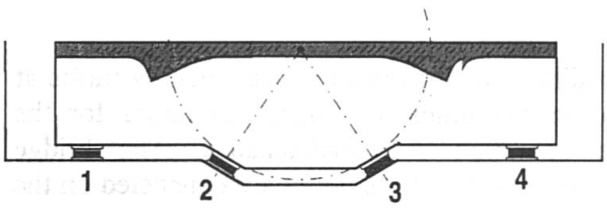
The deck is longitudinally prestressed by seven pairs of 12_T15_S tendons running from end to end. This slab is also prestressed in the transverse direction by 4_T15_S tendons. Two tendons are positioned over each local steel node connecting concrete ribs and tubular skeleton's struts. As shown below, those punctual nodes are sunken in the concrete ribs.

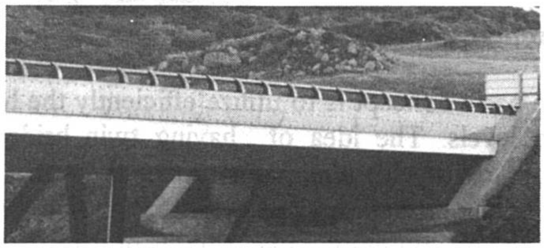



***Longitudinal 7 tendons disposition :***

***4 of them are undulating tendons:***
***UP : position at connection nodes***
***DOWN : position between punctual connection nodes supporting deck.***

The bearing system at the abutments provides sideways locking as well as taking up the vertical forces. Each cross-beam at the abutments is a counterweight on four neoprene bearings. Shear forces are very low in those neoprene bearings, even in the inclined ones.



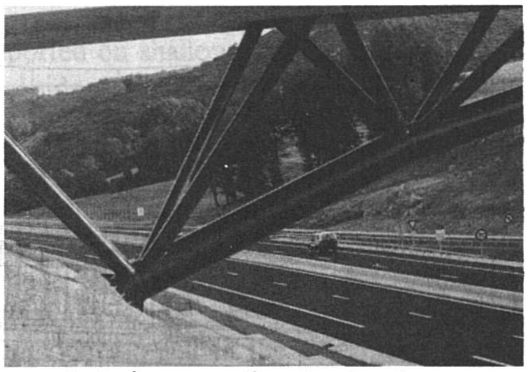



***Roles of bearings :*** ***1 - 2 - 3 - 4 : vertical load blocking***
***1 and 4 : torsion blocking - 2 and 3 : lateral load blocking.***

## 3. Design of the tubular steel framework

Because most of the bridge mass is in the slab, and not in the arch, it is preferable to give the arch a polygonal profile, almost perfectly funicular in transmitting permanent loads. Furthermore, from the economic standpoint, a non-developable toric shape could not be achieved under acceptable conditions with a 32 millimetres thick steel sheet.

Straight tube sections were therefore positioned between two successive butt welded joints at nodes.



It was decided to fill the bottom parts of the arch with concrete to improve the structure's resistance to collisions with outsize vehicles.

## 4. Conclusion

The contract, for a value of 11.3 million francs and an overall execution time of 16 months, was signed with the contractors ***GTM*** and ***Richard-Ducros.*** At a slightly higher cost than for an ordinary bridge, the Project Manager achieved an exceptional structure, well-suited to the landscape topology. In 1995, the Antrenas Bridge, designed by ***SETRA*** received the Silver Ribbon Award from the French Ministry of Public Works. Michel Virlogeux and the consulting architects Dezeuze and Zirk were also commended by the French association of steel-building companies called "Syndicat de la Construction Métallique de France" for an Architecture Prize awarded in the road bridges category.

# The Dreirosen Bridge over the Rhine at Basel

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**Summary**

The basic concept is to utilize efficiently the height difference for the structure given by traffic at two levels. The idea of having twin bridges offers favourable economic conditions for the structure itself, the constructional work, the operation and maintenance. The bridge superstructure consists of two-storey composite constructions, whose decks are connected on the outside by continuous steel trusses of transparent form.

## 1. Basic Concept

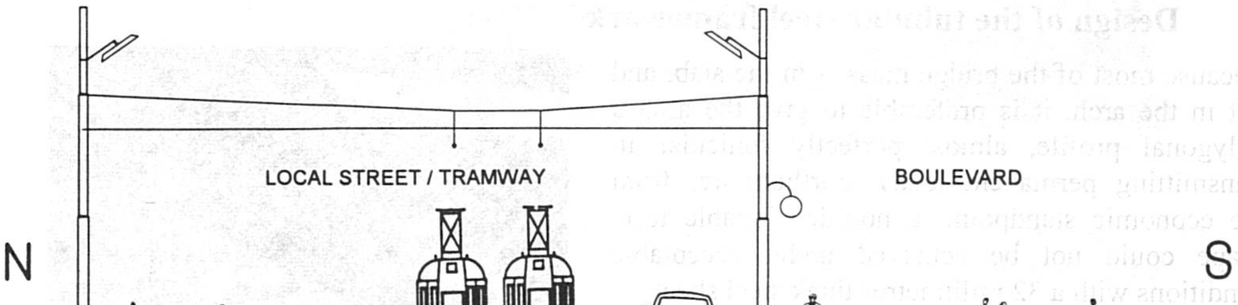


**Fig. 1 Cross-section of bridge with the traffic areas**

From the point of view of urban construction the rather simple three span structure is impressive visually on account of its delicate truss work providing sufficient transparency to allow viewing both into and through the structure. Two parallel concrete chords, which carry the steel trusses,

emphasize the spanning of the River Rhine. The main elements of the bridge are continued harmoniously into the shore area of Kleinbasel.
The choice of an equal width of the upper and lower levels for constructional and noise protection reasons and a shift of the traffic lanes on the upper level to the northern side leaves on the southern side some considerable space for an attractive boulevard.

## 2. Structure

### 2.1 Bridge Superstructure

By keeping the existing pier axes the spans amount to 77, 105 and 84 m. The total height of 8.05 m and the constructional height between the axes of the upper and lower chords of 6.50 m can thus be kept to a minimum. The slenderness ratio ($h_k$ : 1) is 1:16. In order to ensure transparency and throughviewing the truss diagonals were selected to consist of slender concrete-filled steel tubes, which together with the concrete chords form the bridge truss. The dimensions of the concrete chords were chosen such that the joints of the steel truss are perfectly and permanently encased in concrete. The bridge decks comprise longitudinally and transversely prestressed ribbed slabs with a rib spacing of 7.0 m and a span of 14.95 m. The total width of the superstructure is 33.0 m.

### 2.2 Piers and Abutments

The massive piers for the new bridge (40 x 4 m) were constructed at the same place as those for the existing bridge. Only the upper parts of the old piers will be removed. The pier shafts and the caissons will be integrated in the new pier foundations (constructed using bored piles) and the new piers. Both abutments are built up of three parts. They consist of two external and one massive internal pier.

### 2.3 Approach Structures

The approach structures consist of frame structures supported on shallow foundations, of 126 m length on the Grossbasel side and 132 m on the Kleinbasel side.

### 2.4 Constructional Work

The constructional work is based on a division into two independent parts. In a first phase the existing Dreirosen bridge, moved a distance of 15 m upstream, takes the whole of the traffic. In this way one can proceed with the construction of the one half of the bridge at its permanent place. Afterwards the traffic will pass over the new half of the bridge and the second upstream half of the bridge will be constructed in a slightly shifted position after the demolition of the existing bridge. Finally, the new upstream half of the bridge will be manoeuvred hydraulically in to an adjacent position next to the new downstream half of the bridge.

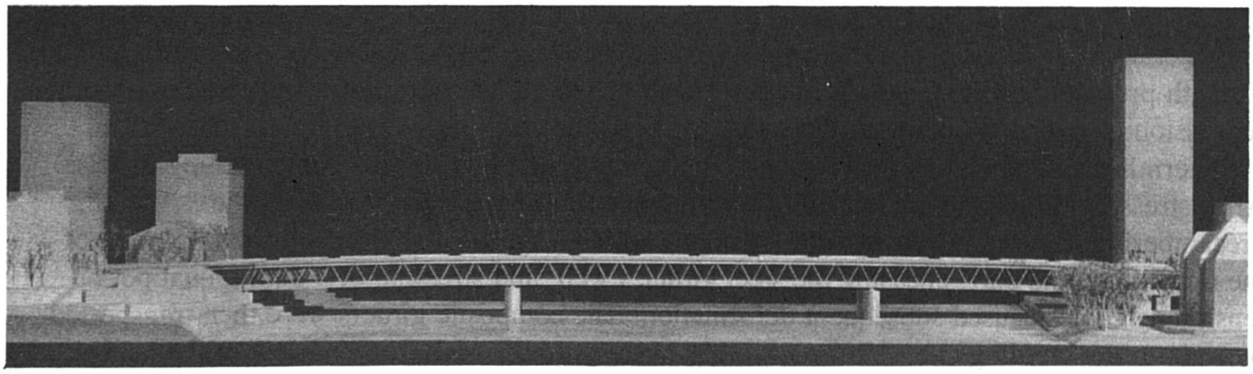

Fig. 2 Photo of Model

# Deformation-Compatibility of Steel Truss and High-Strength Concrete Girders

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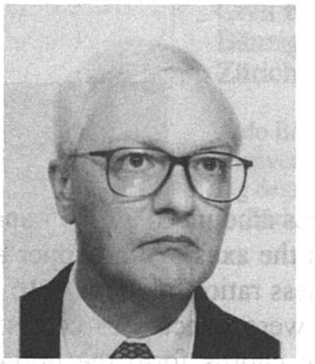

Philippe Van Bogaert, born 1951, received his civil engineering degree from Ghent University in 1974 and PhD in 1988. He is currently head of bridge design office with TUC Rail Cy - Brussels and professor of Bridge Engineering at Ghent Univ.

**Summary**

The railway fly-over at Lot, was built for the crossing of the high-speed railway line from Brussels to Paris over domestic tracks. Its superstructure consisting of lateral composite girders with steel truss elements bolted to the upper flanges, required geometric compatibility of the composite girders and the truss elements. The influence of a set of parameters was examined. From the recordings of all deformation steps, it was found that the composite girders with high-strength concrete showed little time-dependent deformations.

## 1. Structural concept

The railway fly-over at Lot (see fig 1) is located some 7 km to the south of Brussels. It was built for the crossing of the high-speed railway line from Brussels to Paris over domestic tracks. It consists of 16 spans of 42.60 m, the total length of the fly-over becoming 682 m. The fly-over had to be prefabricated entirely, since it was designed to be built over tracks remaining in service. The design of the superstructure is remarkable in this sense that a complete composite structure was built (see the superstructure cross section fig 2). The piers have alternatively a triangular or straight shape. The triangular piers comply with the truss shape of the superstructure and are composite members, resisting the braking and acceleration forces of trains. Two lateral composite girders are equipped with an intermediate reinforced concrete deck plate and transverse stiffening ribs. Steel truss elements are then bolted to the upper flanges of the composite girders.

The lateral girders consist of welded steel I-beams of 2.2 m depth. These are encased in high-strength precast concrete C 80/95. While producing the girders, they were subjected to a succession of stress and deformation states. At first, the steel beams were fabricated with a predetermined initial rise and precambered by concentrated forces, thus compensating the rise. After stressing of bonded tendons, a lower concrete flange was cast. Releasing the precambering forces and cutting of the tendons initiated again a rise of the girders. Casting of the upper concrete part, encasing the steel girder's web, and stressing of 4 additionnal post-tensioning cables caused subsequent deformation. The composite girders were then transported by train to the building site. After concreting of the stiffened slab the geometry of the bridge decks had to similar to the truss geometry, which were allready fabricated.

## 2. Deformation-compatibility and effect of high-strength concrete

The most difficult part was to predict the deformation state of the composite girders at the construction stage where the truss elements were presented. From laboratory tests a secant deformation modulus of the concrete was determined. However, all parameters, governing creep and deformation were not kept under control. As the girders were placed on the piers, the concrete age varied from 43 to 210 days. The values of concrete resistance varied from 93 to 119 MPa. In addition, the relative concrete strength at which the precambering was loosened, the strands where cut, or the post-tensioning was applied varied considerably too. From the recordings of all deformation steps, the influence of these parameters was examined (see fig 3 for charts of deformations as a function of $f_{c28}$ and concrete age). Due to the use of high-strength concrete the time factor and other creep factors were almost insignificant. Eventually the fabrication tolerances of the steel beams were found to be the most significant parameter for determining the deformations of the girders. Thanks to this the deformation steps were predicted accurately, thus achieving the required geometric compatibility.

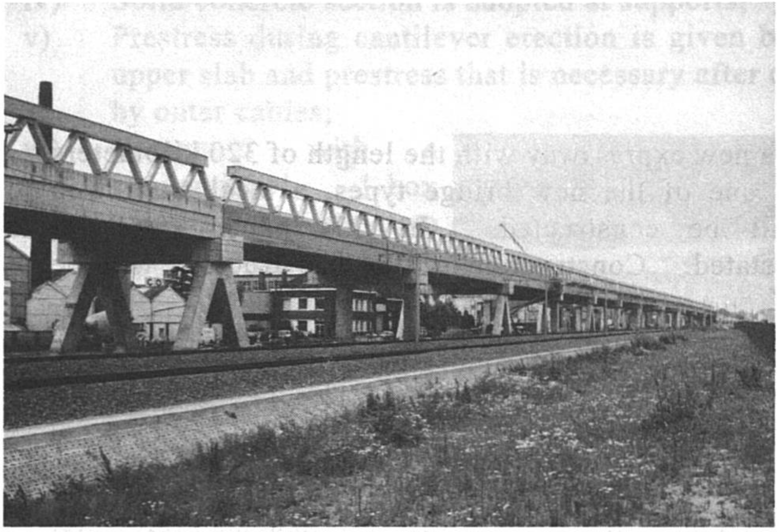

*Fig. 1 Overall-view of fly-over*

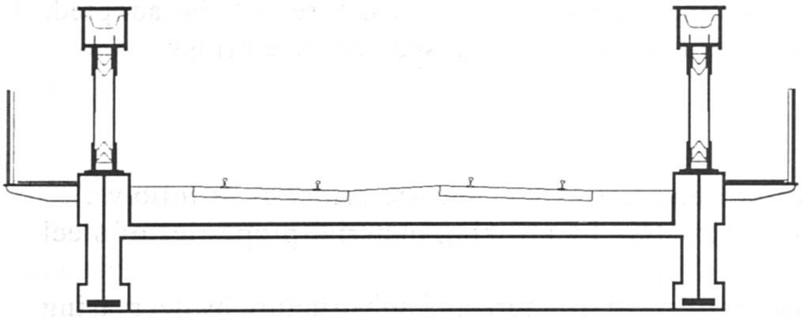

*Fig. 2 Superstructure cross section*

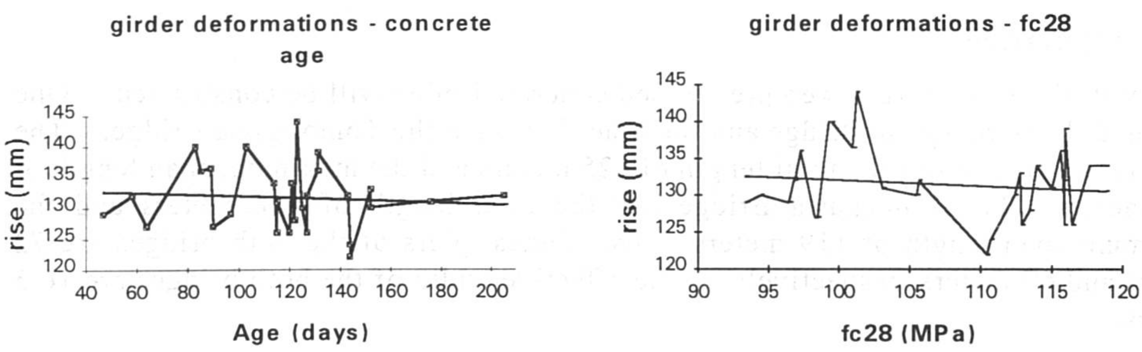


*Fig 3 : Composite girder deformations versus factors*

# Planning of Steel Truss Web Prestressed Concrete Bridge

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**Summary**
The Second Tomei Expressway is a new expressway with the length of 320 kilometers linking Tokyo and Nagoya. As one of the new bridge types, a steel truss web prestressed concrete bridge will be constructed. In this paper structural characteristics of this bridge is stated. Construction of the substructure will be starting from 1997.

## 1. Introduction

The Second Tomei Expressway is a new expressway with the length of 320 kilometers linking Tokyo and Nagoya. A section of the expressway in Shizuoka Prefecture, located about the center between Tokyo and Nagoya, runs through mountainous area. The section has the mainline length of 134 kilometers. About 33 % of the total section length, namely 44.3 kilometers, will be bridges and viaducts. As many bridges and viaducts should be constructed, various new structure will be adopted. One of new type of structures is a steel truss web prestressed concrete bridge.

## 2. Purpose

The purposes of adopting steel truss web prestressed concrete bridge is as follows:
i) To have a rational composite structure by utilizing material properties of steel and concrete;
ii) To have a rational combination of superstructure and substructure by decreasing weight of superstructure and width of bottom slab;
iii) To have a structure type, even with noise barriers, that gives less impression of massiveness.

## 3. Structure

Two unit of the steel truss web prestressed concrete bridge will be constructed. One is named the Sarutagawa Bridge and the other is named the Tomoegawa Bridge. The Sarutagawa Bridge has the total length of 625 meters and the maximum span length of 110 meters. The Tomoegawa Bridge has the total length of 478 meters and the maximum span length of 119 meters. The highest piers of the both bridges are 72 meters and 69 meters, respectively. The effective width of the both bridges are 16.5 meters.

Main characteristics of this structure are as follows:

i) The upper and bottom slabs are cast-situ concrete constructed by cantilever erection method;

ii) Section of truss members and span of upper slab are decreased by adopting 4-plane main truss structure;

iii) Fabrication cost of truss members is decreased by using box shape steels;

iv) Solid concrete section is adopted at supports;

v) Prestress during cantilever erection is given by prestressing steel arranged in upper slab and prestress that is necessary after completion of the girder is given by outer cables;

vi) Compared with a conventional 1-box prestressed concrete girder, weight of the girder and the total dead load become about 86% and 88% respectively.

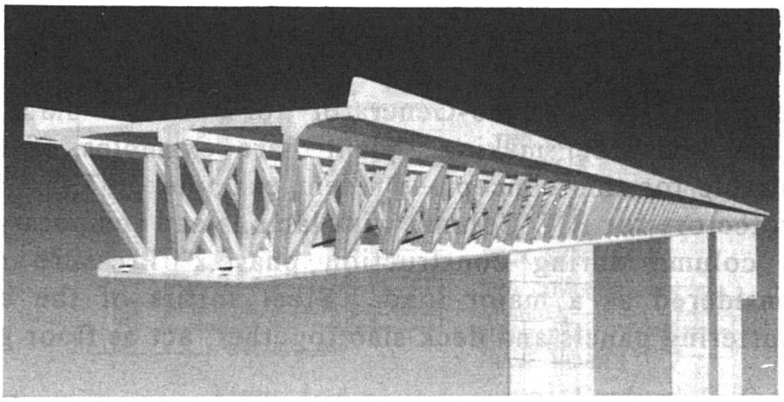

Fig. 1 Cross Section

Structural details of connecting panels are one of the important points of this structure. It is required that panels can be connected easily between concrete blocks. It is also required that the panels can absorb errors during cantilever erection. Various types of connecting panels were compared and the type with cast iron cones is proposed.

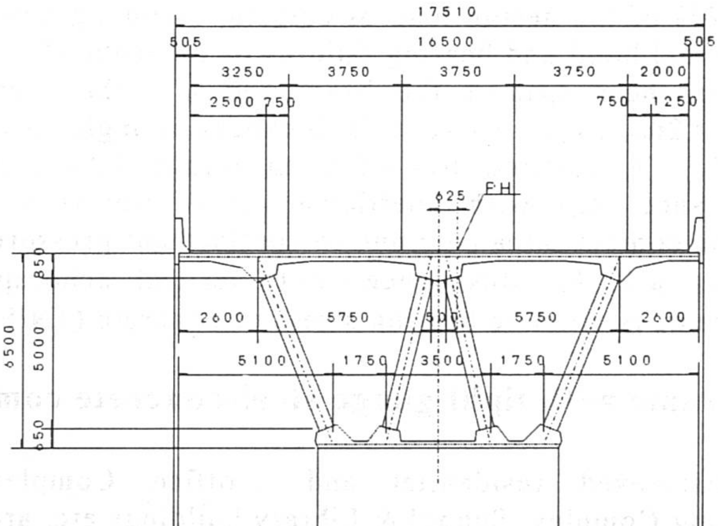


Fig. 2 Cross Section at the Pier

The upper slab is supported discretely by top ends of the truss members. This support mechanism is different from the ordinary slab that is supported continuously by solid webs. Finite element analyses were done using a finite element model and design bending moments and shear forcesfor the upper slab were determined.

## 4. Conclusion

In this paper design concept of the steel truss web prestressed concrete bridge is introduced. Construction of the substructure will be starting from 1997 and construction of the superstructure will be starting from 1999.

# Connections for Ease of Fabrication and Erection with Cold-Formed Steel Permanent Formwork

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## 1.Steel - RCC Hybrid composite frame. (Figs. 1 & 2)

Buildings for Turbo-Generator of power plant, raw material crushing plant, and steel making plant are examples of heavy industrial structures suitable for this concept. Present example of T.G. Building has Corner angle protectors for R C Column with lacings and temporary shuttering panels, to act as column during construction phase. Pressure of wet concrete is also considered as a major load. Steel soffits of the beams with temporary side shuttering panels and deck slab together, act as floor grid system.

The channel cleat on the column carries the shear from beam($V_d$) and the weld connections between the cleat and cloumn carry force ($T_s$) from beam soffit based on 50% of the design load of column, assuming mobilisation or weld forces after frictional bond and bearing failures of the embedded soffit. $T_s = V_d\ L_c/a$ (Where $L_c$ is the shear span of the beam and 'a' is the lever arm). The force in the weld ($P_w$) = $2d_{pw}\ t\ F_{xx}$, where t is thickness of angle cleat. $F_{xx}$ is weld strength. Force ($T_s$-$P_w$), is resisted by soffit anchorage into column. Anchorage frictional resistance ($R_f$) with coefficient of friction ($\mu = 0.57$), and for normal forces on the contact area ($A_c$) due to confinement pressure $P_c$ of 0.722 N/Sqm, is given by $R_f = \mu\ P_c\ A_c$. Resistence due to vertical achorage($R_b$) by bearing, $R_b = \sigma_b\ A_b$, where $\sigma_b$ is ultimate bearing stress of concrete (1N/Sqmm). $A_b$ is bearing area.

## 2. Frame with light guage steel-concrete composite(Figs. 2 to 5)

Multistoreyed residential and office Complex, Commercial and High-rise Parking Complex, School & Library buildings etc. are examples of this concept. For columns, loss of column flange area due to notch (for continuity of RCC between beam & column) is compensated by extra bar reinforcement, to prevent buckling of this zone. Transverse stirrup is replaced by diaphragm which also acts as stiffener to column envelop steel. Welding between the cleat and column is found to be critical, as one of the specimens failed in this area.

Contribution of tiffness of column element to joint-stiffness is a vital parameter for Euro-Code 3 based classification of beam-column joints. Continuity aspects like improved rotation capacity and ductility make these connections amenable to semi-rigid and rigid beam-column joints. Substitution reinforcement to permanent shuttering sectional area, provides continuty in beam-to-beam connections. Ease of handling, 30% increase in strength-to-weight ratio, simple connections and speedier construction are the merits of above two systems.

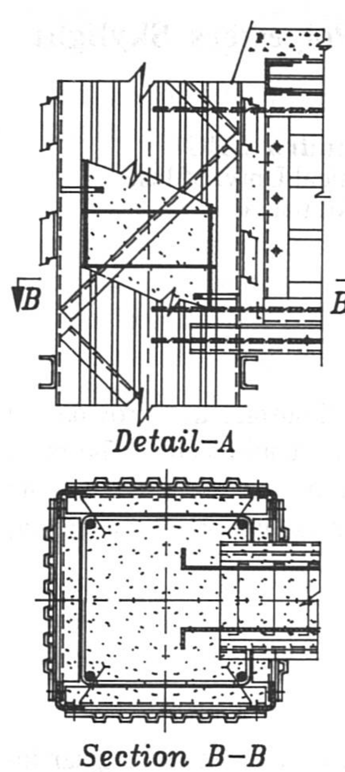


Fig.1 Beam-Column Joint

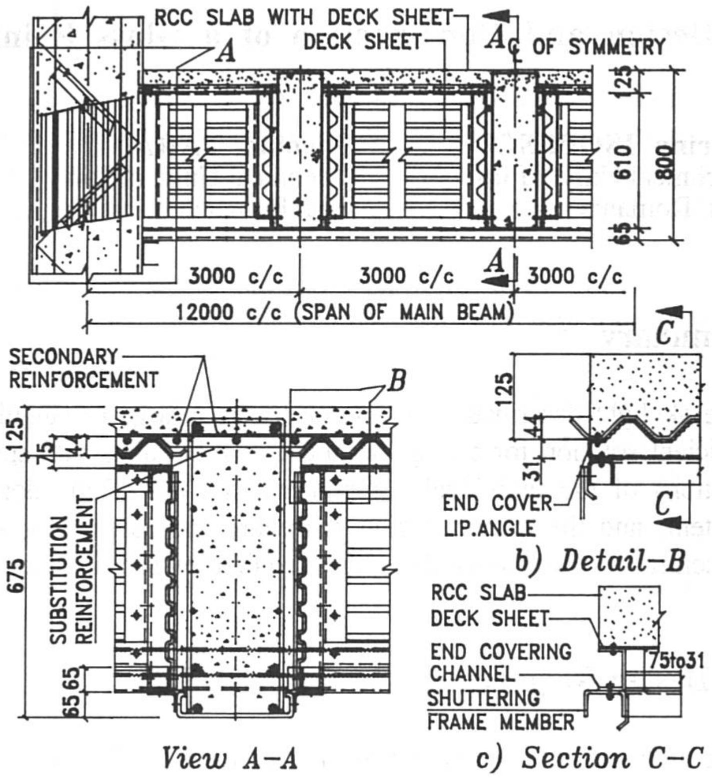


Fig.2 Composite Slab-Beam-Column system

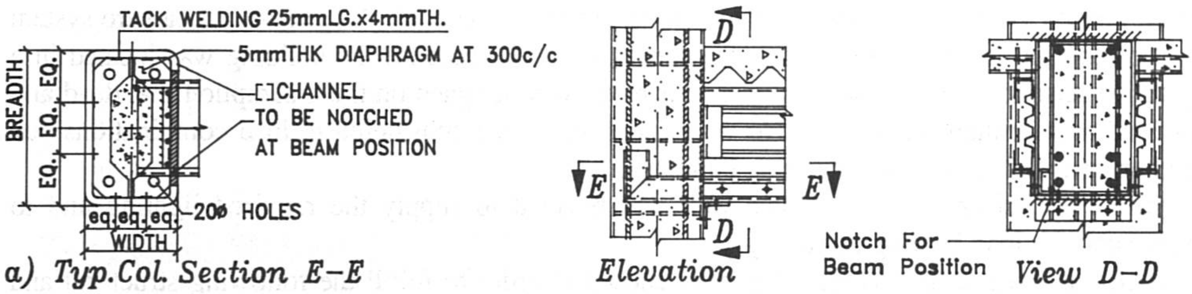


Fig.3 Beam-Column Connection With Concrete Filled Tubular Column

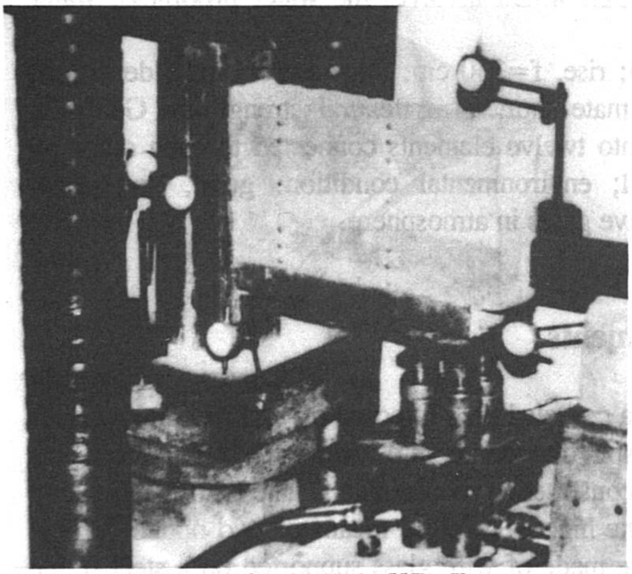

Fig.4 Test Set UP For Beam-Column Joint Connection

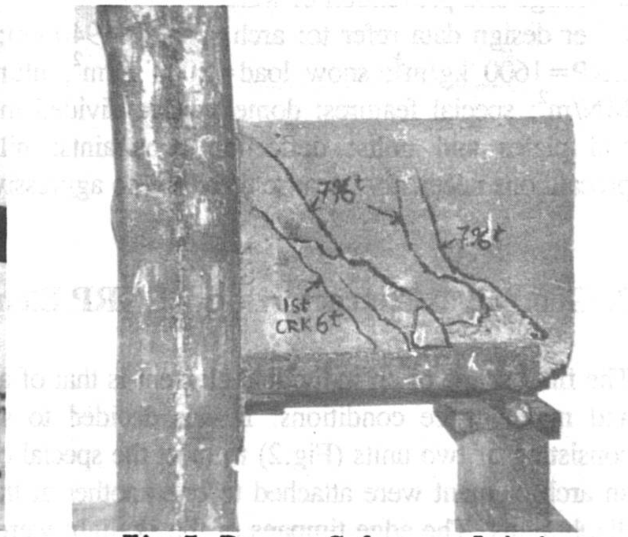


Fig.5 Beam Column Joint Specimen After Test To Failure

# Design and Construction of a Glass Reinforced Polyesters Skylight

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## Summary

The skylight described in the paper covers a large rectangular horizontal surface and provides an efficient solution for a large administrative building with special lighting requirements. The cross sections of the individual components, the use of the double curvature, the proposed jointing system, and the manufacturing procedure and assembling satisfy the most important functional, structural and architectural qualities required by such an enclosure system.

## 1. Design Requirements and Constraints

The architectural constrains and the structural build up of the edifice have led to a rectangular in-plane surface (9.40 m x 10.50 m) to be covered by the lighting aperture.
The skylight has been contrived as a cylindrical dome made of glass reinforced polyesters (GRP), (Fig.1), chosen to perform as structural as well as enclosure material. A light weight closure system was also required in order to minimize the earthquake load, because the building was located in a very active seismic area. The skylight elements have been designed on the assumption that dead and snow load act simultaneously on it. No wind loads have been considered in load combinations due to sheltering effects of the neighbouring buildings.
An appropriate thickness of the dome wall was selected to supply the required lighting and to preserve enough load bearing capability.
The particular cross section (Fig.3) has been chosen in order to fulfill the following structural and functional requirements: strength and stiffness during handling transport and assembling; in service loadbearing capability and rigidity; covering function which involves the water proofness, water discharge and prevention of water seepage.
Other design data refer to: arch span, L=940 cm; rise, f=330 cm; thickness=3 mm; density of GRP=1600 kg/m$^3$; snow load=1000 N/m$^2$; ultimate short term flexural strength of GRP=25 MN/m$^2$; special features: dome surface divided into twelve elements connected to each other by end plates and bolts; deflection constraints: nil; environmental condition: good, no special precautions necessary because there are no aggressive gases in atmosphere.

## 2. Geometric Characteristics of GRP Elements

The final shape of an individual element is that of a corbeled arch imposed by architectural, service and maintenance conditions. It was decided to split the dome into twelve arch type elements consisting of two units (Fig.2) to meet the special construction requirements. The two units making an arch element were attached to one another at the highest level by means of two bolted vertical diaphragms. The edge timpans of the skylight were made of wire glass supported on a steel frame that also provides wind bracing. All dome elements and timpans rest on a reinforced concrete corbel.

## 3. Manufacture and Erection of GRP Elements

An open mould method, namely the hand lay-up technique, was used to take full advantage of the fact that the polyester resin does not need special conditions like heat or pressure for complete polymerisation to occur. The mould was made of timber slats to achieve the double curvature pattern of the element. A release agent was applied to the mould to prevent bonding between GRP element and the timber form.

The reinforcement in the form of woven fabric and strand mat had been precut to the correct size. A gel coat necessary to protect the fibres on the exposed surface of the composite was sprayed on the mould surface. After the gel coat became tacky and firm a first layer of resin was brushed over and the first layer of glass reinforcement was placed in position and consolidated with rollers. The required layers of resin and reinforcement have been applied until the designed thickness of the wall element was obtained. Curing of the GRP elements took place in a construction workshop under a temperature range, 17°...20°C.

Each arch element, made of two parts, was assembled with bolts then erected and laid up in the final position. A light weight steel scaffolding was used to offer a temporary support to the first GRP element while being set up on the first one and this sequence was used until the skylight was completed.

The final joints between the GRP elements and the concrete corbel were made of mild steel bolts and sealed with a plastic compound. Then, the lateral steel formed timpans filled with coloured wire glass were erected. Eventually the border elements and the timpans were jointed.

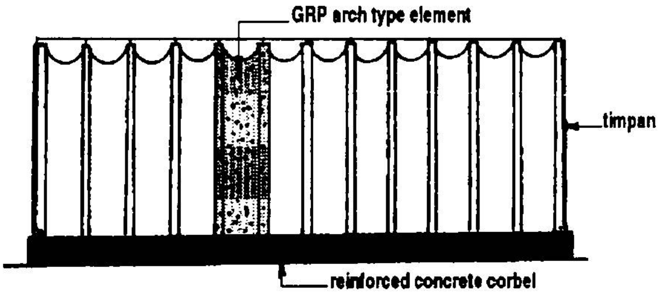
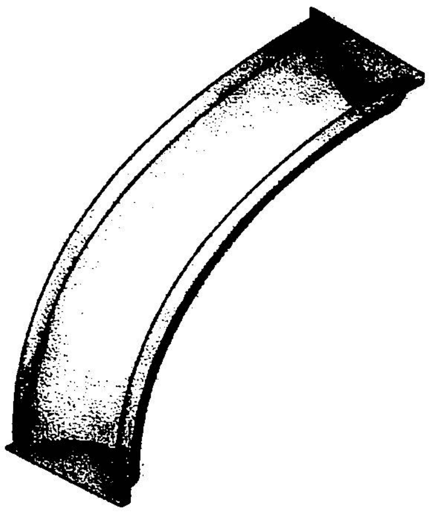



*Fig.1 GRP skylight*



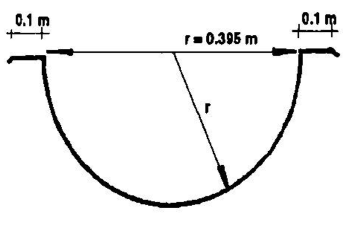



*Fig.3 The cross section of unit*

*Fig.2 The unit of arch type element*

# Large-Span Architecture with Hyperbolic Composite Thin-Shell Structure

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**Summary** The large-span elemental erecting architecture with composite hyperbolic thin-shell structure is a scientific and technological result of structure, architecture, construction and material. The application, calculation principle and construction craft of this architecture form are introduced in this paper.

**Key words: hyperbolic thin-shell, composite structure, binder**

## 1. Introduction

The large-span architecture has been widely applied in the fields such as large scale gymnasium, airport, ware house, exhibition etc.. Thin-shell structure, cable structure and net-frame structure have been widely used. Of course, the construction of those kind of structure is difficult.

In 70's decade of this century, the structural principle, originated in the Chinese pre-fabricated hyperbolic arch bridge, has been successfully applied in the architecture design in China. The authors of this paper have participated in the design and construction of the assembly hall of the Chinese Petroleum University, which is one of the typical structure of these kinds. By elemental pre-fabricating and erecting method, 40m span hyperbolic thin-shell roof with bearing and maintenance functions are conveniently achieved. The compressive shell of the roof is composed by wire mesh concrete hyperbolic curved shell plate, and its low chord drawing member is composed by welding channel steel.

## 2. Structure Scheme

There are 2 types in this architecture form, one is the shell-plate structure scheme and the other is the rib-plate scheme. In the shell-plate structural scheme, the elemental hyperbolic thin-shell is divided into 5, 7, or 9 blocks along the span direction and assembled one by one on independent support frame. (Fig. 2). In the rib-plate scheme, the elemental hyperbolic thin-shell is divided into arch ribs and cross curved plates. The arch rib is parallel to the span, while the cross curved plate is perpendicular to it. The arch rib is pre-fabricated by reinforced concrete or welded with shaped steel, and divided into 2 or 3 segments for erection. The finished rib should be fixed firstly by tension rod, and then the cross curved plate are assembled between two ribs. The cross curved plate is made of wire mesh concrete, metal or glass fiber reinforced plastic(GFRP). The span of cross curved plate adopts the distance between two adjacent ribs or takes 5m, 6m, 9m, etc..(Fig. 3)

## 3. Key Point

An important characteristic in our study is that it provides a supplementary scheme for the natural lighting of the large-span architecture. Based on the demand for natural lighting, some of the shell part in the hyperbolic thin-shell elements are replaced by GFRP. Of course the by-product is the abundant color style on the structure. GFRP is a kind of widely used new construction material which can be easily shaped and colored, which has high strength, and which is light-penetrated. That is why we use this material.

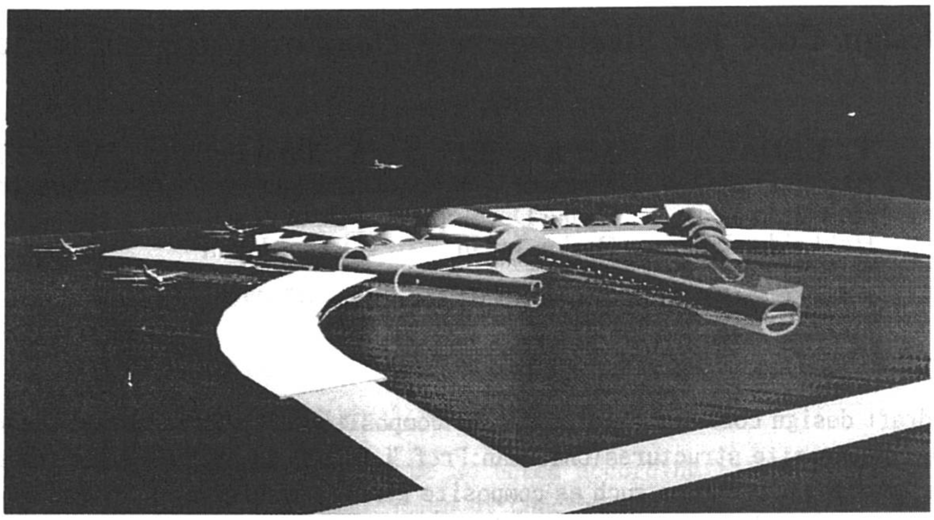


FIG.1 AN AIRPORT DESIGN

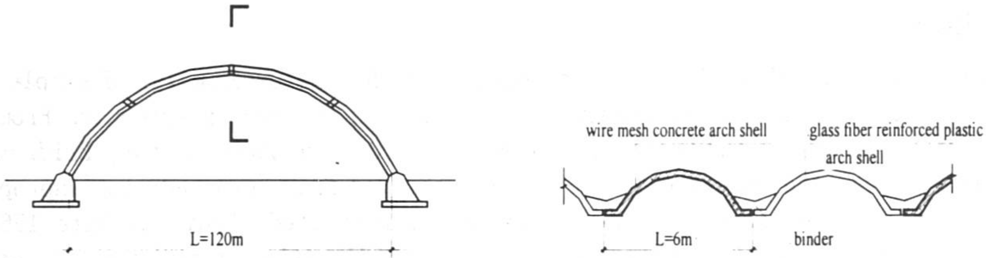
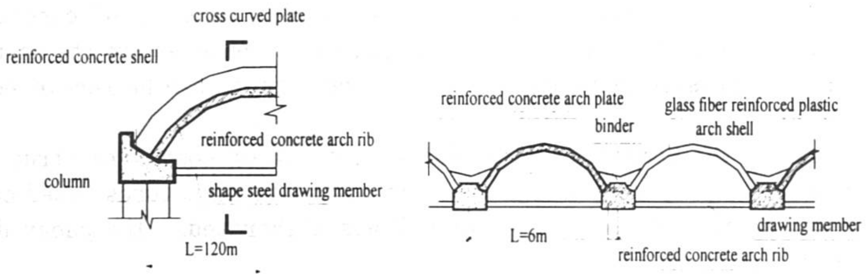



FIG.2. SHELL-PLATE SCHEME




FID.3. RIB-PLATE SCHEME

What should be pointed out is that, on designing this kind of architecture, careful consideration should be taken on the aspects of architecture scheme, structure scheme, construction scheme as well as material features so as to make ultimate use of its features and ensure its safety and utility. For example, in the construction of rib-plate structural scheme, the arch rib actually acts as the temporary support frame for the erection of cross curved plate. But after the completion of the whole element and the integration construction, the rib and the plate become an integrated bearing element. As we know, the elasticity and the expansion coefficient of GFRP can be changed by adjusting the ratio and type of glass wire, raisin and additives. This feature provides the theoretic basis and practical possibility for the design of composite element which is made by this material and other materials.