Prefabrication and Prestressing of Concrete Slabs in Composite Bridges

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SUMMARY

This paper describes the different solutions developed or proposed in France to precast and prestress slabs in composite bridges, aiming at reducing tensile zones and limiting cracks under permanent loads. Slabs have not been prestressed in the practical applications due to the economical competition with reinforced slabs cast in situ, since the improvement in quality produced by longitudinal prestressing has not yet been recognized.

1. INTRODUCTION

In most composite bridges, the slab is directly cast in situ on the steel structure, sometimes with a concreting sequence organized to reduce tensile zones. This solution is extremely efficient from an economical point of view but it has some drawbacks, limiting the participation of the slab to composite action and producing cracks in the tensile zones of the slab.

1.1 Shrinkage and thermal effects.

Since the slab is directly concreted on the steel structure, all effects which produce shortening are restrained by the upper members of the steel structure and result in tensile stresses - and cracks - in the concrete slab. Specialists divide concrete shrinkage into three parts :

- thermal shrinkage, which results from the increase in temperature during concrete hardening, produced by cement hydratation, and from the later return of hardened concrete to outside temperature ;
- autogenous shrinkage, which directly results from the effect on concrete volume of cement hydratation ;
- drying shrinkage, which results from evaporation of water in hardened concrete.

These effects have been often underestimated in concrete slabs of composite bridges, since engineers frequently refered to classical evaluations of shrinkage made for prestressed concrete structures in which the first two parts have no, or practically no, effect.

Of course, the unfavorable effects of shrinkage can be limited by a reduction of the cement ratio in concrete (on condition to have a perfectly constant quality of cement) and of the water to cement ratio. A careful curing of fresh concrete - for example with humid blankets - and a late removal of shutters also help limiting shrinkage effects, but these provisions are often forgotten under the pressure of cost. A last provision, adopted in some countries, consists in slightly heating the steel member during concrete hardening.

1.2 Structural effects.

Engineers perfectly know the structural effects of the concreting sequence : tensile zones can be reduced upon supports when concreting in the spans in a first step. But, for simplicity, erection steps are frequently computed with the long term modulus of elasticity of concrete, forgetting the slow development of creep. This can result in an underestimation of tensile stresses during erection, and in an increase in crack opening [6].

1.3 Evolution of requirements

The French Administration - SETRA - considered that previous requirements on concrete slabs were not enough conservative and could not prevent the development of cracks more open than acceptable for a high durability. Michel VIRLOGEUX decided to publish a recommendation which was later written under the direction of Thierry KRETZ [7]. In addition to practical specifications, the longitudinal reinforcement ratio, which was equal to 1 % in the tensile zones, has been increased on more scientific bases, with a minimum value on the whole length of slabs including the zones under supposed compression.

Diameter of reinforced bars (mm.)		14	16	20	25
Minimum ratio (%)		0,80	0,86	0,99	1,10
Extreme S.L.S. stress (MPa)	without transverse prestressing	320	280	240	200
	with transverse prestressing	240	200	180	160

The reinforcement ratio does not reach the high values specified in some other codes but looks acceptable considering the French experience. We can only regret :

- the extremely unfavorable conditions made for transverse prestressing, which do not consider the adopted technology and the effective influence of cracks on the durability of transverse tendons ;
- and the fact that the minimum reinforcement ratio is also to be applied to slabs made from precast elements when a more scientific analysis would lead to smaller values.

2. SLABS MADE FROM PRECAST ELEMENTS

2.1 Principle and structural effects.

The slab can be made from precast elements, whatever is the structural system for the bridge : with two parallel steel I-girders and cross-beams; resulting in a two web box-girder; with multiple parallel steel I-girders and cross-beams ; with two parallel steel I-girders and floor-beams

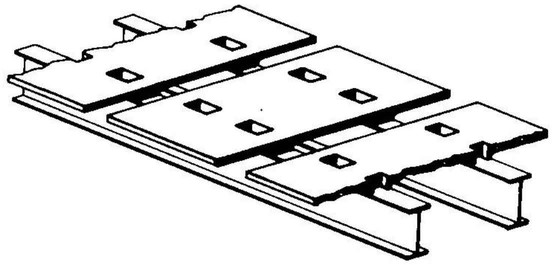

Fig.1 : *Precast slab elements on a steel structure made of two I-shaped beams*

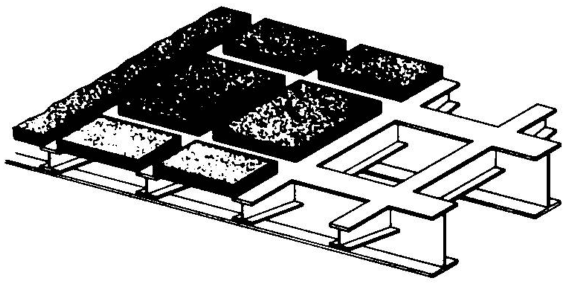

Fig. 2 : *Precast slab elements on a steel structure with floor-beams*

In all cases, tensile stresses produced in the slab by its selfweight can be totally eliminated by a convenient construction sequence (when installing all slab elements before connecting them to the steel structure). And creep has practically no influence on the stress distribution if the sequence is conveniently selected.

2.2 Principle and structural effects.

The use of precast slab elements has the great advantage of eliminating the influence of thermal effects and of early shrinkage (including the largest part of autogenous shrinkage) ; but evidently there is no advantage at all for the second phase concrete, poured in the joints between slab elements or in pockets when connection is made through pockets. The effects of shrinkage are even increased in the second phase concrete due to the additional restrain produced by the precast slab elements.

This is why the second phase concrete must be selected for a limited shrinkage - through a limited ratio of cement, and through a low water / cement ratio -, and for a high tensile strength. Some specific research must be made in this direction. If successful, it could allow for a limitation of the minimum reinforcement ratio in zones under compression, to take advantage of the elimination of the structural influence of early shrinkage and thermal effects in hardening concrete. Of course second phase concrete must be conveniently cured and protected as for cast in situ slabs.

2.3 Watertightness.

Special provisions must be taken to avoid any concrete leakage between the steel structure and the precast elements, specially at the wet joints between elements or at the pockets. These provisions must also produce a perfect corrosion protection of the steel structure where the slab elements just rest on it. These results could be provided by small plastic joints compressed between steel and concrete by the weight of the precast elements, and by an injection of some adapted product between the steel members and the concrete slab, after closing joints and filling pockets.

2.4 Match-cast elements

Some engineers considered the use of precast slab elements assembled with dry joints and prestressing tendons. Prestressing tendons are necessary since no reinforcement bar can pass through the joints. Clearly, the slab elements are to be match-cast and glued ; surprisingly, this was not the case for the single application of this idea on the A8 motorway [3], with pockets for the connection between slab and steel structure.

But the most interesting question concerns the use of prestressing.

3. USE OF PRESTRESSING IN SLABS OF COMPOSITE BRIDGES

3.1 Connection before or after tensioning tendons.

This example of slabs made from match-cast slab segments clearly evidences the main problem : if the connection between the slab and the steel structure is made before tensioning the longitudinal tendons, a large part of prestressing forces is lost immediately in the steel structure and some more later with creep effects ; thus it appears much more interesting to tension longitudinal tendons before connecting the slab to the steel structure ; all prestressing forces will pass in concrete with a limited transfer, later, with the development of creep. This is why we shall not consider at all, in this paper, solutions with longitudinal prestressing tendons tensioned after connection between concrete and steel (for example for cast in situ slabs), but only the case of precast slabs prestressed before their connection to the steel structure.

3.2 Advantages of longitudinal prestressing.

Longitudinal prestressing - when introduced before the connection between concrete and steel - has two advantages :

- first, the slab can have a composite action on the whole length of the bridge, at least under permanent loads (with equipments) and frequent loads (frequent live loads) ;
- and, consequently, the structure durability is increased by eliminating cracks under permanent loads.

In addition, prestressing slabs in composite bridges produces distributions of stresses in concrete which are similar to those in classical prestressed concrete bridges. We don't think, here, in terms of code but of philosophy. In prestressed concrete structures, we try to balance permanent loads with prestressing effects to reduce creep-induced deflection and to increase durability. We accept no tensile stresses under permanent loads (but accept them under extreme SLS loads, in the widely accepted theory of partial prestressing), nor main cracks.

But solutions have to be developed to introduce longitudinal prestressing in slabs of composite bridges without increasing the cost or complicating erection. This paper aims at describing the ideas proposed so far, even if some of them have been applied without longitudinal tendons.

3.3 The Swiss experience.

More than twenty years ago, Swiss engineers developed the idea of a prestressed concrete slab incrementally launched on the steel structure [1,2]. The slab, precast and prestressed by segments 15 to 25 meters long, is equipped with pockets for a later connection. When the slab has been launched (being prestressed), Nelson studs are welded in the pockets and pocked are filled with concrete to produce the connection.

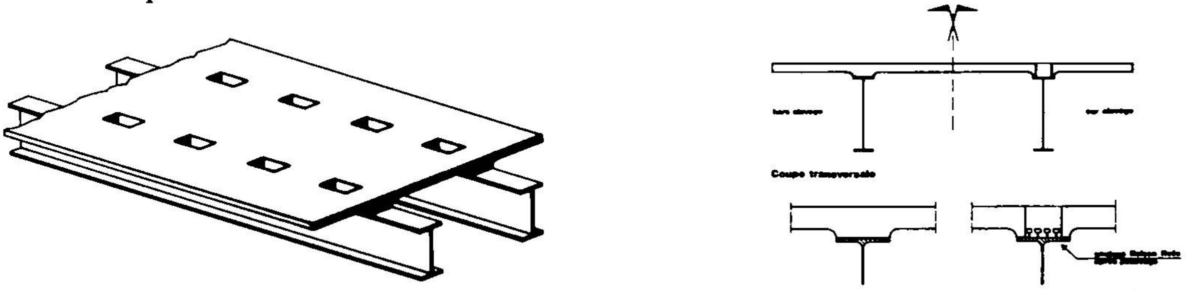

Fig. 3 : *A precast concrete slab launched on the steel structure with pocket, to later weld Nelson studs.*

This solution has been abandoned after an accident which was by no way a real drawback of the idea.

3.4 Ideas derived from the Swiss experience

Jean Claude FOUCRIAT proposed, for the bridge of the A75 motorway over the river Truyère, at Garabit, a solution with a slab stiffened by two longitudinal ribs above the members of the steel structure. This slab was to be launched on the steel structure on a series of concrete blocks extending the concrete ribs ; a steel plate was at the basis of each block, connected to it through Nelson studs. After launching, the steel plates connected to the concrete slab were to be welded to the steel structure below, completing the desired connection. But this solution left a series of holes between the concrete slab and the steel structure below (Fig. 4, [3,4]).

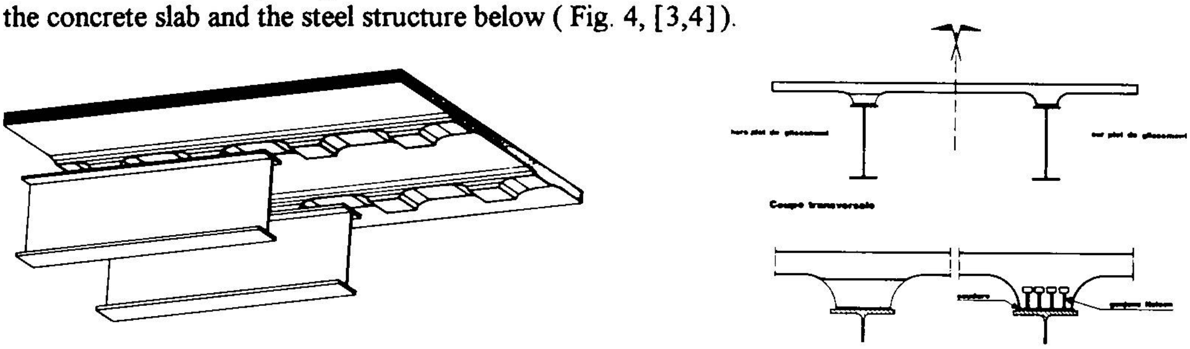

Fig. 4 : *The solution proposed for the bridge over the river Truyère at Garabit.*

Spie Batignolles and Richard Ducros improved the solution in their offer for the erection of this bridge (finally built in prestressed concrete), with continuous ribs each of them resting on a continuous steel plate connected to the concrete slab with Nelson studs ; after launching, the steel plate connected to the concrete slab was to be welded to the steel structure below to complete connection (Fig. 5 [3,4]).

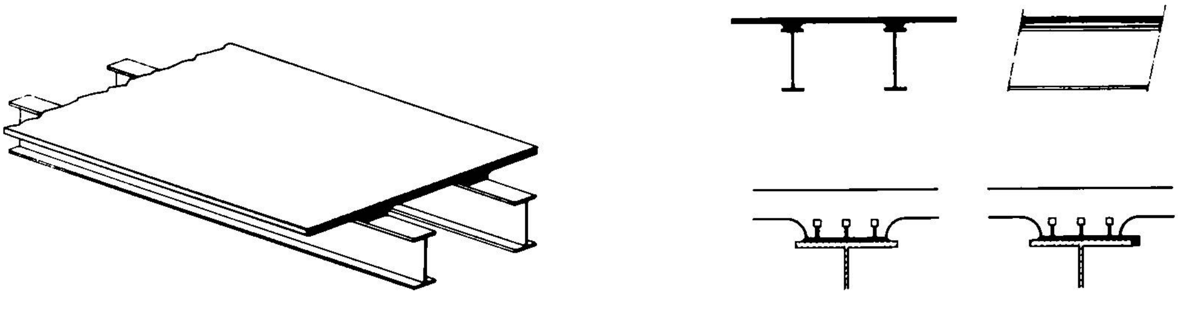

Fig. 5 : *A precast concrete slab precast on two steel plates, with a later welding to the steel structure below.*

This idea has been proposed some years later by RAZEL and Michel PLACIDI, for the Douy Viaduc on the Colioures by-pass. But this solution has been rejected by Joel RAOUL and Thierry KRETZ considering the geometrical tolerances which could prevent welding the two plates together ; in addition, they considered that transverse bending moments could not be transfered from the concrete slab to the steel I-girder through the two corner welds connecting the two superposed plates. Michel VIRLOGEUX and Michel PLACIDI then separately proposed very similar solutions, leaving a channel in the concrete slab above the members of the steel structure already equiped with connectors, Nelson studs or other (Fig.6 [4]).

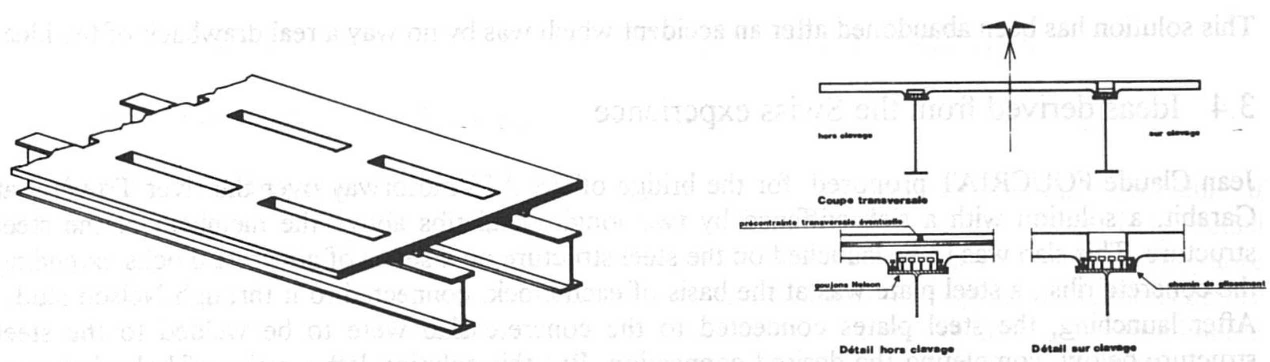

Fig. 6 : *A precast concrete slab precast with a channel over the Neson studs and pockets for reinforcing and conreting.*

In the Michel Placidi's design, the concrete slab elements are independent, transversally, only connected by reinforcement bars rigid enough to control transverse deflections (Fig.7)

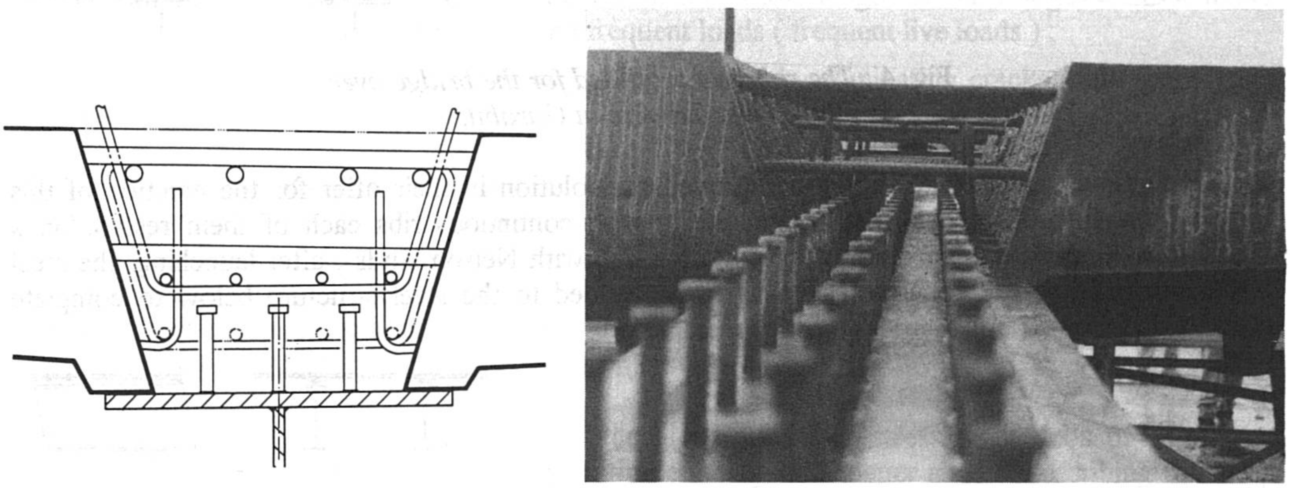

Fig.7 : *A precast concrete slab in independent elements, transversally connected by reinforcement bars.*

RAZEL and Michel PLACIDI built that way the access spans to the mobile bridge over the Tancarville Canal, on the A29 motorway.

But once more Joel RAOUL pointed at drawbacks, evidencing that geometrical tolerances could produce unacceptable effects in the welds between the upper member and the web of the steel structure, when all the load of the concrete slab passes at only one of the two edges of the upper member. Michel VIRLOGEUX concluded that the concrete slab has to be supported during launching by two vertical plates, one on top of each of the two webs of the steel structure below.

This idea was improved and developed by Michel PLACIDI (Fig. 8), and five bridges have been built that way by RAZEL these last years : the viaduct over the National Highway 6 and the PLM railway lines at Varennes-les-Mâcon [8] ; the bridge over the river Fier, at Annecy ; the bridge over the river Orne, at Caen ; the bridge over the river Allier at Brioude and the bridge over the railway lines at Lisieux ; a sixth one is being built, also at Lisieux, over the river Orbiquet.

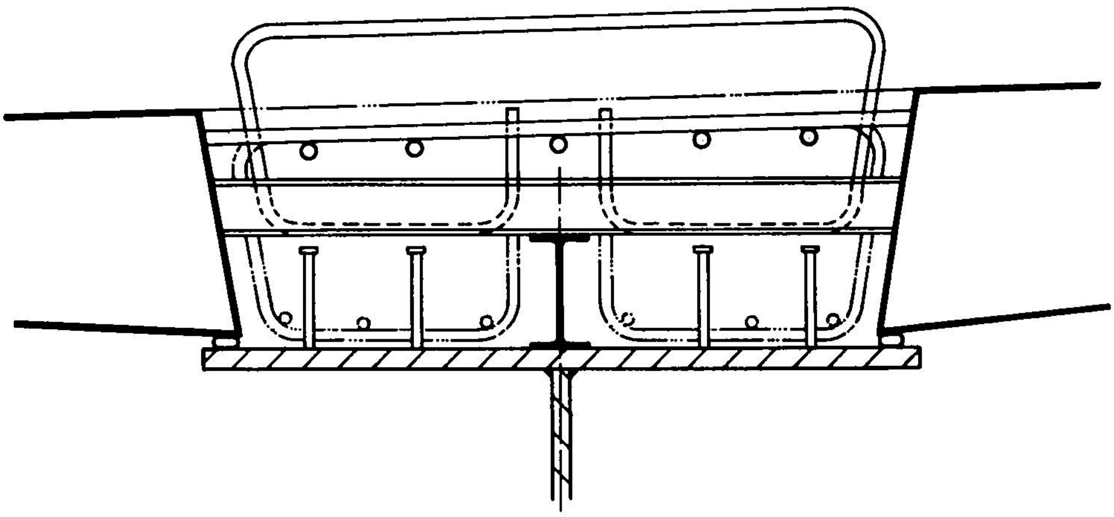

Fig. 8 : *In this solution, independent elements of the precast slab, connected by steel beams, are supported by vertical plates.*

Unfortunately, none of these bridges received longitudinal prestressing, though it would have been very easy to introduce some : the owners were not convinced of the interest of longitudinally prestressing the slab to increase quality and durability, and did not accept the very small increase in cost that it would have required.

3.5 Transverse prestressing

All the bridges built by RAZEL, except the access spans to the mobile bridge over the Tancarville Canal, received transverse prestressing. For the Varennes-les-Mâcon, Annecy, Brioude and Lisieux bridges, this prestressing is made of a series of pairs of individually protected monostrands of 15 millimetres, at about 0,80 metre intervals ; the corresponding cost is balanced by the reduction in the reinforcement ratio, from about 250 to 180 - 190 kg/m^3. For the bridge over the river Orne, which is very wide, the slab is transversely ribbed at 1,50 metre intervals ; and each transverse rib is prestressed by one tendon made of four strands of 15 millimetres, injected with cement grout in a classical duct.

We can only regret - in addition to the lack of longitudinal prestressing - that the new French specifications led to an increase in the longitudinal reinforcement of the last four bridges (over the river Orne, at Brioude, and the two bridges at Lisieux) without really scientific bases.

4. CONCLUSION

As a conclusion, we strongly believe in the improvements which could be produced by longitudinally prestressing the concrete slabs of composite bridges before their connection to the steel structure, as regards quality and durability. Evidently, solutions must be developed to introduce prestressing forces efficiently at a low cost, with simple techniques and a limited equipment. After many hesitations between different solutions, it appears that one is being efficiently developed by RAZEL to launch a prefabricated concrete slab on the steel structure of a composite bridge, allowing for an easy and economical introduction of longitudinal tendons in the slab before its connection to steel. Prestressing the slab would probably cost a bit more than just reinforcing it, but we consider this cost limited as compared to the improvement in the bridge quality and durability.

Clearly, this will by no way reduce the economical competitiveness of composite solutions : in the present conditions, and for spans between 40 and 80 to 100 metres, composite bridges are cheaper than prestressed concrete bridges by 5 to 10 %, except when the size of the structure to build is so large that the cost of concrete can be seriously reduced, or when local conditions forbid the erection techniques adapted to the construction of a composite structure.

In such a situation, a possible increase in quality is an advantage for composite structures which will appear to owners, more and more concerned with low maintenance costs.

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Composite Floors of the Buildings

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Summary

Modular product systematics can be applied in modern prefabrication technology. A module is defined to be a partly independent assembly of components. One module is the floor, which is designed to fulfill the mechanical and physical requirements, but also to allow an easy assembly and rapid changes of the partition walls and installations. Different types of floors are classified and examples of composite structures are presented. The main specialities in the design are listed.

1. The principles of modular building system

The building system is an organised whole consisting of its parts, in which the relations between the parts are defined by rules [1], [2]. Modulation involves division of the whole into sub-entities, which to a significant extent are compatible and independent. The compatibility makes it possible to use interchangeable products and designs that can be joined together according to connection rules to form a functional whole of the building.The floor is a typical module owing a character.

2. Performance requirements of floors

The common main requirements of floors can be classified in the following ways:

1. Mechanical requirements, including
   - static load bearing capacity,
   - serviceability behaviour: deflection limits, cracking limits and damping of dynamic vibrations.
2. Physical requirements, including
   - air tightness
   - airborne sound insulation
   - impact sound insulation
   - moisture tightness (in wet parts of the floor)
   - thermal insulation between cold and warm spaces
   - fire resistance and fire insulation.
3. Flexible compatibility with connecting structures and installations
   - partitions
   - piping,
   - wiring,
   - heating and ventilating installations.

In addition to the performance requirements during service life, the entire life cycle requirements include the buildability, changeability of spaces during the use and easy demolition, reuse and wasting.

3. Advantages of prefabricated composite floors

The multiple requirements can be preferably fulfilled with composite structures, because for alternative designs they own more performance parameters than monolithic structures. Usually the composite structures have the meaning of mechanical composite performance under static and dynamic loadings. Taking into account the multiple requirements listed above, the composite performance has to be widened to include all types of these requirements. The weighting of different types of requirements varies for building types. In apartment buildings the static and dynamic requirements are quite easy to fulfil and physical requirements together with the flexibility towards changes during use of spaces and installations is important. In office, commercial and industrial buildings high loading capacity and easy changes of spaces and installations during the use are dominating properties. In special cases like under seismic conditions, the vertical and horizontal loading capacity and large deformation capacity are important.

The main advantages of composite structures for mechanic properties of floors are:

- the increase of load bearing capacity globally or locally, e. g. around openings, either in new structures or in renovation
- reduced structural depth
- reduced material expenditure
- increased rigidity and thus the decrease of deflections and vibrations.
- improvement of the diaphragm action of the slab field
- the mechanic advantages can be reached through increased loading capacity of the structural members, through continuity or through combination of them.

The main advantages for physical properties are:

- improved water tightness
- improved sound insulation
- improved fire resistance.

The main advantages for flexible compatibility with connecting structures and installations are:

- Possibility of installation spaces for pipes, wires, cables and fastenings.
- Possibility for tight connections with connecting structures.

It must be noticed that the composite structures often have also disadvantages associated with the demountability of the floors at the end of the service life. This is because of the difficulty in separation of the components from each others. This is especially the case when using composites of prefabricated components and in-situ concreting. However, the improved flexibility for changes of installations and connecting partitions is much more important than the demolition phase, because the experience shows that the building will typically be changed several times for different uses before demolition. The entire service life of the buildings in Europe is often several hundreds of years. In the opposite, in Japan the service life of office buildings has been reported to have a mean as low as 25 years.

4. Examples of different types of prefabricated composite floors

4.1 Composites for the improvement of mechanical and physical behaviour

The alternative composite floor slab structures can be classified as follows [3], [4]:

1. Composite concrete floors (Fig. 1.)
   - 1.1 Reinforced or prestressed hollow core slabs
   - 1.2 Reinforced or prestressed solid planks
   - 1.3 Reinforced or prestressed double-T units
   - 1.4 Reinforced Filigran solid planks
   - 1.5 Composite beam-block floors
2. Composite steel-concrete floors [6] (Fig. 2.)
   - 2.1 Composite steel sheet-concrete slabs
   - 2.2 Composite prestressed steel sheet-concrete slabs
   - 2.3 Composite steel truss-concrete slabs
3. Steel profile-board floors [6], [7], [8] (Fig. 2).

Concrete slabs can be connected with supporting beams to also work as composite slab-beam structures. The beams can be made of concrete or steel. Wooden beams are sometimes also used as composite structures with concrete slabs. In order to achieve a good flexibility for future changes to connecting structures and installations, slim beams have reached increasing use.

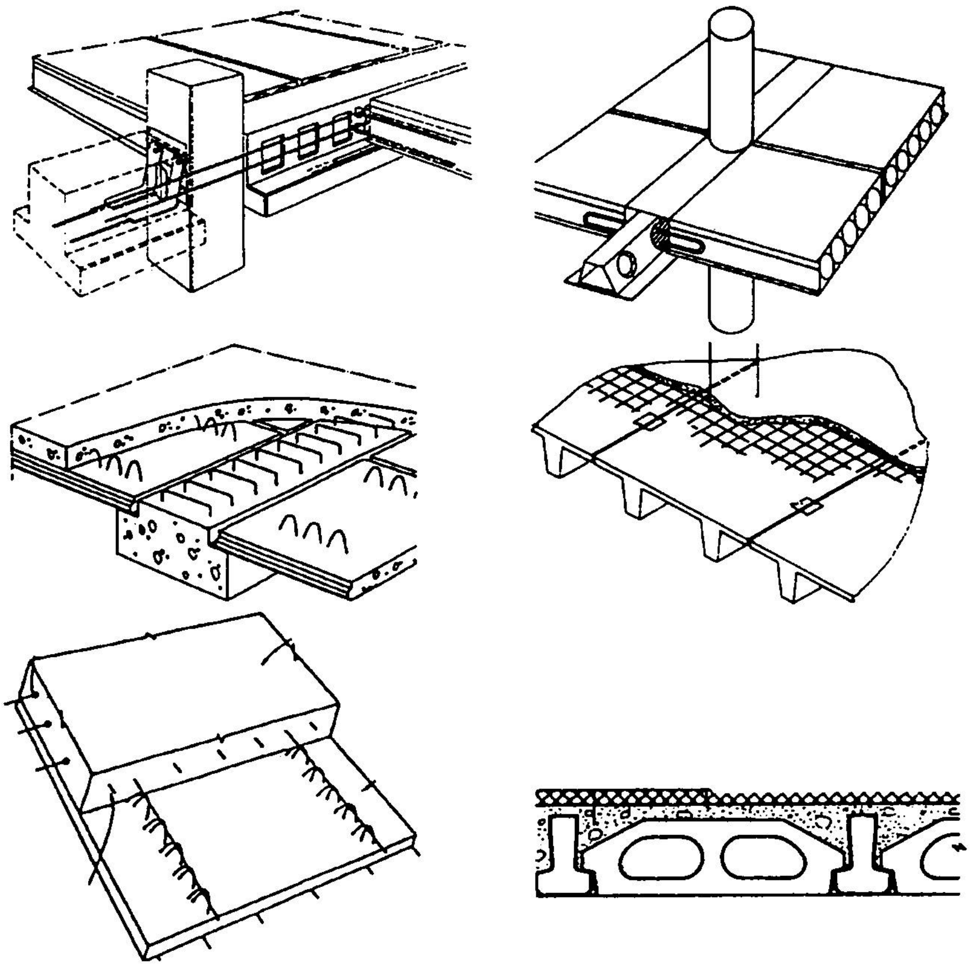

Fig. 1. Composite concrete floors.

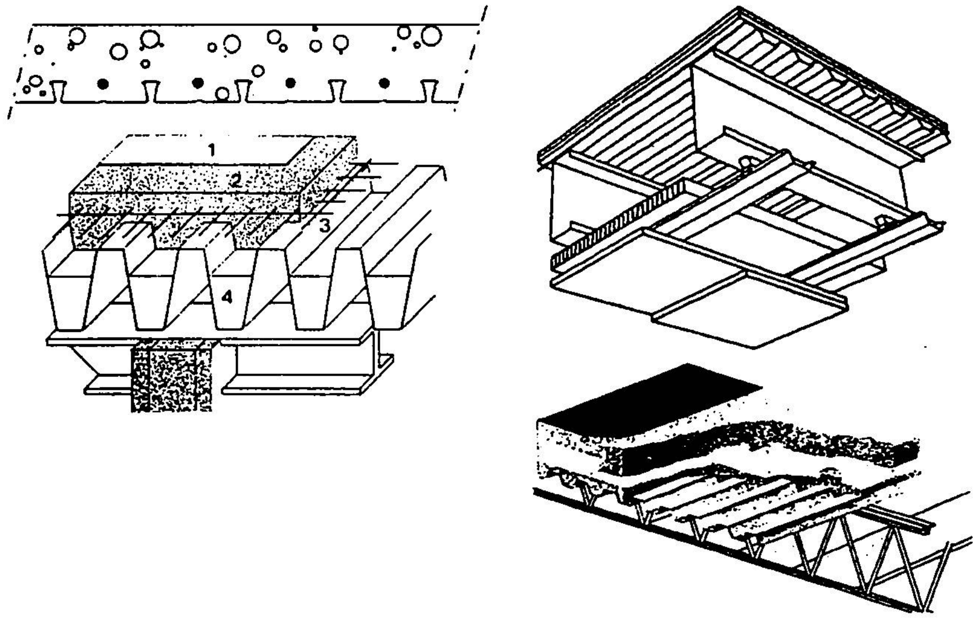

Fig. 2. Composite steel-concrete floors.

4.2 Composites for the improvement of flexible compatibility with connecting structures and installations

The floors are the most important modules for improvement of the installation flexibility for changes during use of the buildings. There are four main principles to solve the compatibility for free distribution of ventilation, electrical wiring, water and sewage piping and information cable networks over the floor area (Fig. 3.) [4]:

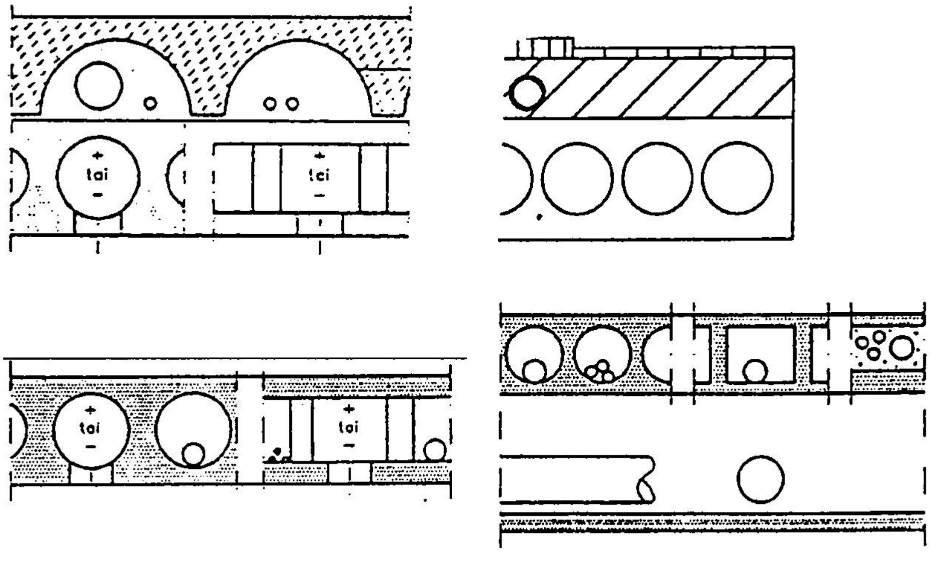

Fig. 3. Installation models in floors.

1. Installations in one- or two-dimensional holes of the bearing slab
2. Installations in the installation floor space above the bearing slab
3. Installations in the soft material above the bearing slab
4. Installations in the ceiling under the slab

The last 3 models listed above can practically serve for the improvement of sound insulation of floors even into very high level. Type 2. is often used in office buildings, where large pipes are needed for ventilation. Type 1. floor can serve the addition purpose of distributing the air for ventilation or combined ventilation and heating without any special pipes. Such kinds of solutions are used for low-energy buildings, where the massive floors can serve for daily storage of heating energy [4].

5. Specialities in designing the composite floors

In order to guarantee the proper behaviour of composite structures, the following phenomenas have to be analysed and solved during the design:

1. Shear stresses at the interfaces of the structural parts.
2. The interface stresses and deformations at different phases of production, as:
   - separated components
   - assemblies of components before possible in-situ concreting
   - final stage without and with external loading.

In addition to the load induced stresses and strains, time dependent visco-plastic stresses and deformations at different times are also important in calculating the interface stresses and deformations.

When using slim beams-slab composites it is important to notice the two-dimensional shear at the interfaces between slab and beam and inside the slab. The last mentioned two-dimensional shear is especially important when using hollow-core slabs with thin webs [5].

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Bending Behaviour of Sandwich Member Using Steel Shell with Joint

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Summary

The tunnel structure constructed by MMST method consists of steel and concrete sandwich members reinforced by double steel shells with joints, and RC members to connect the sandwich members. This paper proposes a friction type joint which uses addition connecting plates to connect the steel shells. Tension test of main girders connected by the proposed joints, and bending test of sandwich member with and without joint have been performed, and the applicability of ordinary RC theory is verified.

1. Introduction

Recently, as one of new shield tunnel construction methods to effectively use the limited underground space, the application of MMST method has been investigated. The MMST method is an effective shield tunnel construction method to construct shield of large section by combing many shields of small sections. The method is now being experimentally applied in the construction of ventilation tunnel of Kawasaki Crossing Highway by Capital Highway Public Corporation.

The tunnel structure constructed by MMST method consists of steel and concrete sandwich members (hereafter called sandwich member) reinforced by dOuble steel shells with joints, and RC members to connect the sandwich members.

The steel shell of sandwich member consists of a skin plate fully welded to main girders in longitudinal direction of member and to the vertical ribs in transverse direction of member, and the joints to connect steel shells (See Fig. 1). Therefore, it is necessary to study the performance of sandwich member reinforced by steel shells with joints.

In this paper, friction type connecting joint is proposed. The friction type joint is such a joint that addition connecting plates are arranged on the two sides of main girder (the addition connecting plate is attached to the main girder by high tension bolts), which transfer the tension forces of skin plate and main girder. The outline of the proposed joint is shown in Fig. 2.

In order to understand the performance of sandwich member reinforced by steel shells with friction type joints, and to study the applicability of RC theory in the case, the effects of the friction at joints and local stresses are first studied, and then the behavior of sandwich member in the cases with and without joints are compared.

In this study, tension test of main girders connected by addition connecting plates was first performed, and then bending tests of sandwich member reinforced by steel shells with and without joint were conducted. Based on the test results, bending behavior of sandwich type specimen reinforced by steel shells with friction type joint which uses addition connecting plates was investigated.

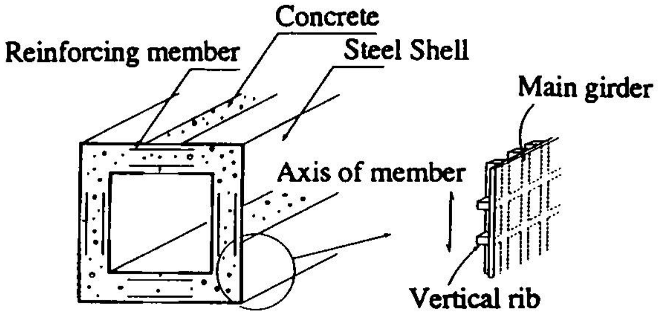

Fig. 1 Outline of steel shell for sandwich member

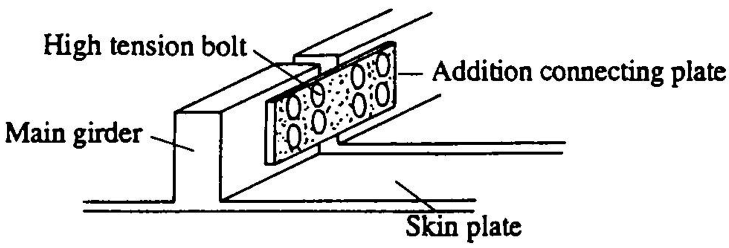

Fig. 2 Outline of friction type joint

2. Tension test of steel shell

2.1 Specimen and test method

The outline of specimen and arrangement of measurement points are shown in Fig. 3. The middle main girder of steel shell (175mm wide and 13mm thick) was put in between two addition connecting plates (113mm wide and 13mm thick), and 16 bolts (M16, F10T)) were used to fix the addition connecting plates to the main girder.

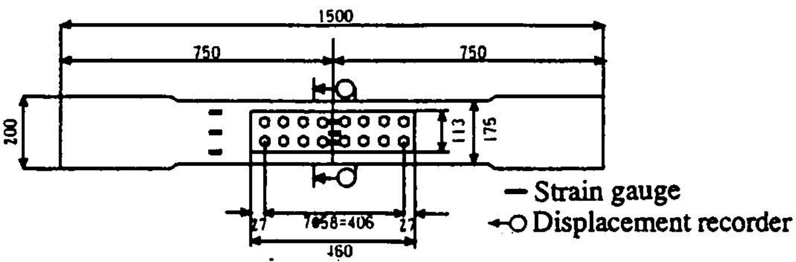

Fig. 3 Outline of specimen and arrangement of measurement points

The size of specimen was 1/2 of the size of real structure considering the minimum thickness of steel plate, and all plates used were plain plates corresponding to SS400. The material properties are indicated in table 1.

In the test, the joint part was set in the middle of tensioned region, and the two ends of main girder were fixed to the tension test machine.

Thickness (mm)	Yield strength (N/mm^2)	Tensile strength (N/mm^2)	Usage
3.2	285	439	Skin plate
4.5	310	434	Vertical rib
6.0	276	433	Main girder
13.0	266	428	Main girder & Addition plate

Table 1 Material properties of steel

2.2 Test results and discussions

Figure 4 represents the relationship between load and displacements of whole specimen and joint. From the figure, it can be seen that joint part yields when load reaches 600kN, while the maximum slide resistant force is 735kN when friction coefficient is set as 0.4. The yield loads for each part of specimen were as follows: 474kN for bolt hole part of main girder, 519kN for bolt hole part of addition connecting plate, and 605kN for main girder. Therefore, the yield load of joint part was almost the same as that of main girder. In addition, it was confirmed that the maximum load during the test was 828kN, which was larger than the tensile strength (753kN) of bolt hole part of main girder.

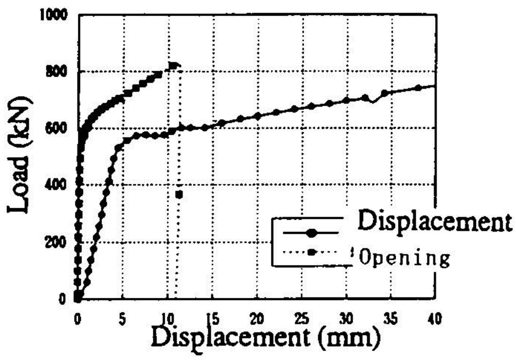

Fig. 4 Relationship between load and displacement

Figure 5 indicates the relationship between load and strain of addition connecting plate. It can be observed that the strain increases sharply after load exceeds 400kN. This may result from the uneven distribution of the force transfer ability on the addition connecting plate because of the existence of bolt holes. The maximum load was 828kN. Collapse mode was the failure of the main girder part at bolt hole. During the test, no clear slide was observed until the failure of main girder.

From the above results, it was confirmed that the local stress and friction affected the behavior of friction type joint under tension load.

3. Bending test of sandwich member

3.1 Specimen

Two specimens were prepared. One (HB1) was a model without joint, and the other (HB2) was a model with friction type joint. The size of the section of addition connecting plate was determined such that the yield load be the same as the sum of yield load of main girder and yield load of skin

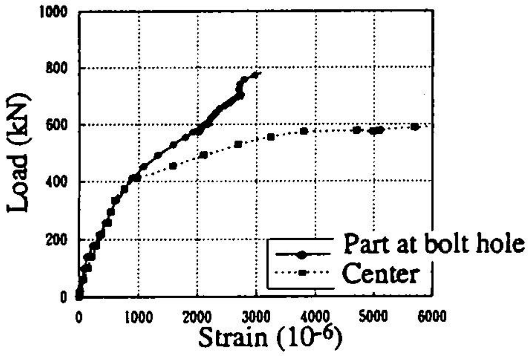

Fig. 5 Relationship between load and strain of addition connecting plate

plate (taking the effects of bolt hole into account). Shot blasting treatment was conducted at the interface between main girder and addition connecting plate, and axial force of 115kN was applied to each bolt under torque management. In total, 16 bolts (F10T, M16) were used.

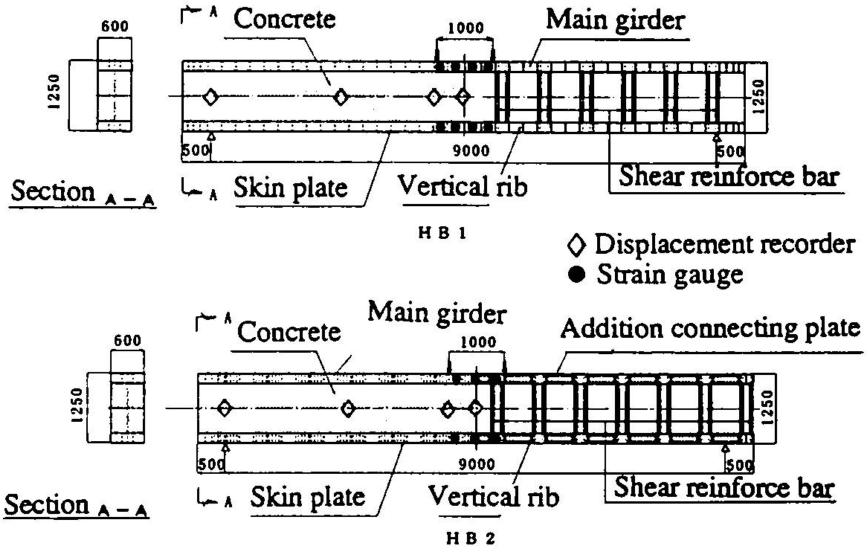

Fig. 6 Outline of specimen and arrangement of measurement points

The outline and arrangement of measurement points are shown in Fig. 6. The size of specimen and the material were the same as those used in tension test of steel shell (See Table 1).
The maximum size of coarse aggregate was set as 10mm taking the size scale of specimen (1/2 of real size) into account. In order to make casting effects the same in vertical direction, the specimen was set with side face down to the ground, and under this condition the concrete was casted. The compressive strength of the concrete during tests were 29N/mm^2 for HB1, and 25N/mm^2 for HB2.

3.2 Test method

Two point loading test method was adopted, and the length of same bending moment was set as 1000mm. Load was applied using jack with capacity of 5000kN. The load was first increased to the level that the stress reached the predicted allowable stress of steel (602kN), and next unload was conducted. And then the load was again applied until the specimen failed.

3.3 Test results and discussions

(1) The case using steel shells without joint

1) Test results

Figure 7 indicates the relationship between load and displacement of sandwich member reinforced by steel shells without joint. The distribution of cracks when specimen failed is shown in Fig. 8.

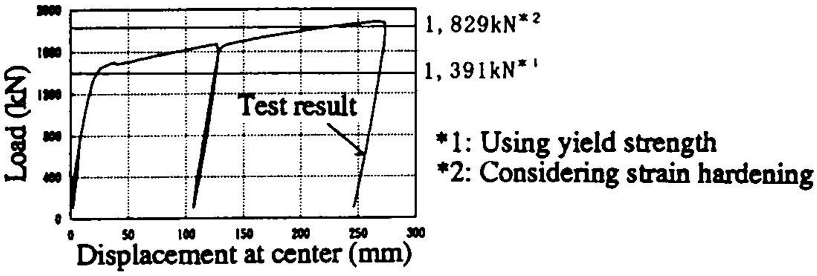

Fig. 7 Relationship between load and displacement (HB1)

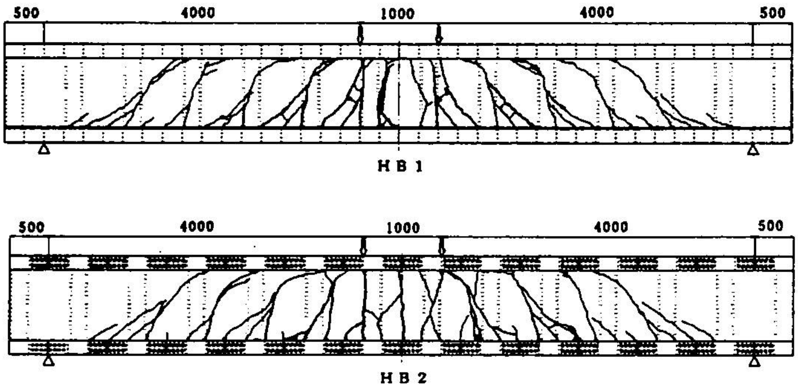

Fig. 8 Distribution of cracks (ultimate state)

The cracks were first observed at vertical rib in the middle of the region of equal bending moment when load reached 312kN, and then were gradually found at other ribs towards the loading points. The skin plate on the tensile side yielded when load touched 1020kN. When load reached 1420kN, the skin plate on the compressive side also yielded. After this stage, with 100kN increment at each step, the cracks in the shear span became wide. The maximum load of 1891kN was observed when displacement reached 268mm, and the skin plate on the compressive side buckled together with the sudden decrement of load. This is because the constrained force applied to the concrete by the steel shell on the compressive side was lower than the level that could cause buckling failure.

2) Comparison with predicted results

Figure 7 shows the computed result of 1391kN, which was calculated through resistant moment using the yield strength of steel by RC theory. As the strain of steel shell on the tensile side reached the strain hardening region at the final stage, the calculation considering the increment of resistance was also performed, and the result was 1829kN, which is also shown in Fig. 7. In the calculation, steel shell was treated as a single steel bar positioned at its geometry center. These two computed results were smaller than the test results, which are 1420kN and 1891kN, respectively. Therefore, the calculation using RC theory provided results on safe side.

(2) The case using steel shells with joint

1) Test results

Figure 9 represents the relationship between load and displacement of sandwich member reinforced by steel shells with joint. The distribution of cracks when specimen failed is shown in Fig. 8.

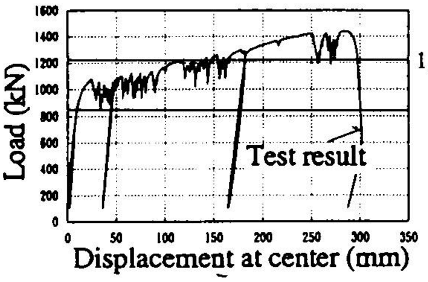

Fig. 9 Relationship between load and displacement (HB2)

The cracks first occurred at the vertical rib in the middle of the region of equal bending moment when load was increased to 292kN. And then, the cracks were gradually found not only at the ribs but also at the position of joint towards the loading points. When the load reached 990kN (the stage just before the skin plate and addition connecting plate on the tensile side yielded), the relative displacement of bolt or concrete were found, and temporary load decrement occurred. After this stage, similar trend was observed, but the load increased.

The maximum load of 1445kN was reached when displacement was 284mm, and failure was found at position of the closest bolt to the joint part together with sudden decrement of load. The failure pattern was different from that of the case with only steel shells. In the test of steel shell, the addition connecting plate first yielded, and then the relative displacement of bolt was observed, and finally the failure of main girder occurred. The reasons for the difference in failure pattern are as follows. In the case with sandwich member, after the crack occurred at the position of joint, only the addition connection plates bear the load, and local stress occurred due to both tensile force and bending moment applied to the addition connecting plate. In the case with only steel shells, eccentric load acted on the specimen because there was no skin plate. The skin plate on the compressive side deformed out of the original plane in several places, although no yield was observed.

2) Comparison with predicted results

Figure 9 shows the computed results of 847kN, which was calculated through resistant bending moment using the yield strength of addition connecting plate based on RC theory. The computation using the tensile strength of addition connecting plate was also performed. The computed result of 1216kN is also shown in Fig. 9. Computed yield load and maximum load were smaller than the test results, and the prediction provided results on safe side. In the case of steel shell without joint, the prediction through resistant bending moment using yield strength of steel by RC theory gives yield load on safe side. In the case of steel shell with joint, by taking the effects of bolt holes into account, the prediction through resistant bending moment using yield strength of steel based on RC theory can also provide results on safe side.

4. Conclusions

On the performance of sandwich member reinforced by double steel shell with joint, following conclusions are drawn.

(1) It is clarified that, in the case using friction type joint by addition connecting plate, the tensile behavior are affected by local stress and friction; and (2) it is found that, by taking the characteristics of joint into account, ordinary RC theory can predict the bending behavior of sandwich member with friction type joint.

Reference

(1) Sakurai, J., Hasegawa, K. and Hirabayashi, Y.: On the application of MMST method, 51th Annual Conference of JSCE, pp.224-225, Sept. 1996.

Composite Steel and Concrete Pier Using Durable Precast Form

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Summary

This paper reports on the development and practical application of a new construction method for bridge piers in which H-shaped steel columns with ribbed flanges and durable precast forms with stainless steel fibers are combined.

It was also demonstrated in the practical application that the implementation of the method shortened the construction period by 66% and required 40% less workers, as compared with the conventional method.

1. Introduction

The devastating earthquake that occurred in the Kobe region of Japan in January 1995 caused various forms of damage to reinforced concrete bridge piers. As a country with frequent earthquakes, Japan was awoken again to the fact that it was absolutely crucial to improve the aseismicity of bridge piers.

Meanwhile, highways have increasingly been planned in mountainous regions where the bridge piers are required to have a considerable height. For such highway projects, there has been a demand for the development of a rational and speedy construction method which also saves labour .

The method described in this paper was developed to provide both a new structural configuration of and a new erection system for bridge piers ,with the aim of improving seismic performance and durability, saving labor and speeding up the erection work. The following report gives the outline of this method, describing its development and practical implementation.

2. Outline of the Method and its Features

2.1 Outline of the Method

The method introduces composite reinforced concrete bridge piers which combine durable precast forms and H-shaped steel columns with ribbed flanges instead of the conventional main longitudinal reinforcement (Fig. 1).

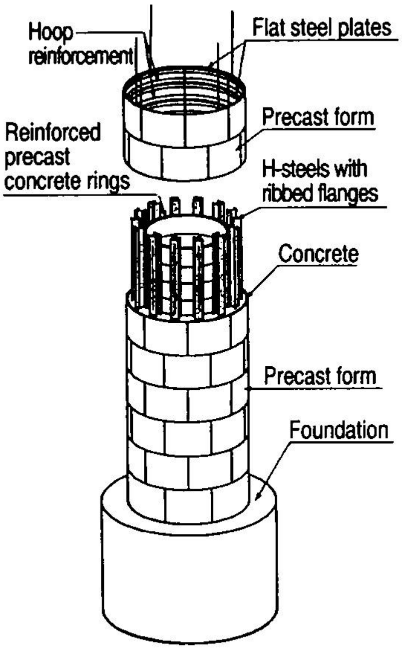


Fig. 1 Steel-concrete composite pier

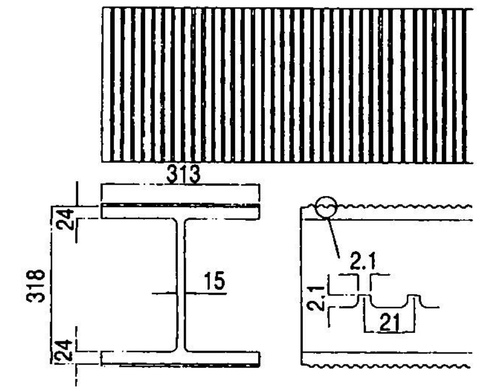


Fig. 2 H-steel with ribbed flanges

2.2 Materials

The H-shaped steel columns with ribbed flanges are H-shaped steel members that have small ribs at a right angle to their flange faces (Fig. 2). These ribs provide the section with a higher concrete bond strength than conventional H-shaped steel members.

The durable precast form is made of mortar with a compressive strength of 700 kgf/cm^2 that contains stainless steel fibers. It works as a form when placing concrete, and functions as part of the structure by acting together with the concrete once the concrete achieves working strength.

2.3 Features of the Method

This method has the following features:

(1) Superior seismic performance is provided by the composite steel and concrete pier which consists of the durable precast forms and H-shaped steel columns with ribbed flanges having enhanced bond strength and higher rigidity than conventional reinforcement bars. This feature is described in detail in 3.2.

(2) Because the precast forms used in the method have stainless steel fibers on their surfaces, the effects which limit both the crack dispersion and the widening of cracks are equivalent to or better than those experienced in reinforced concrete structures. These effects are described in detail in 3.1.

(3) Because the surfaces of the structure are protected by the durable precast forms, the structure becomes highly resistant against salt attack, frost damage and carbonation with much improved durability.

3. Outline of Tests

Having a new structural configuration and erection system, the structural performance of the method had to be confirmed by a variety of tests. The following sections outline the bending tests carried out on beams and the horizontal loading tests using scale models of bridge piers.

3.1 Bending test on beams

3.1.1 Test objective and preparation

The object of this test was to study the structural performance and deformability of the proposed composite steel and concrete pier when it functions as a flexural member. To fulfill this aim, the loading behavior of a composite steel and concrete beam specimen with the durable forms (hereinafter called *SC+PCa* Specimen) was compared with those of a specimen without the durable forms (hereinafter called *SC* Specimen) and a reinforced concrete beam specimen (hereinafter called *RC* Specimen).

3.1.2 Test results

It was found from the test that the width of cracks in the *SC+PCa* Specimen was 50% to 80% of that in the *SC* Specimen. This is because the precast form installed at the extreme tension fibre contributed to the effective restriction of the widening of the cracks.

Furthermore, it was confirmed that the *SC+PCa* Specimen had a higher resisting force at the ultimate state and the best deformability, because of the precast form installed at the extreme compressive fibre.

3.2 Horizontal loading test using scale models

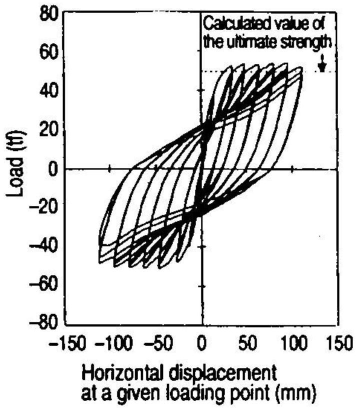
3.2.1 Test objective and preparation

The object of this test was to study the seismic performance (ductility) of the proposed composite steel and concrete bridge pier. It was confirmed by carrying out horizontal loading tests on scale models with alternating positive and negative loads.

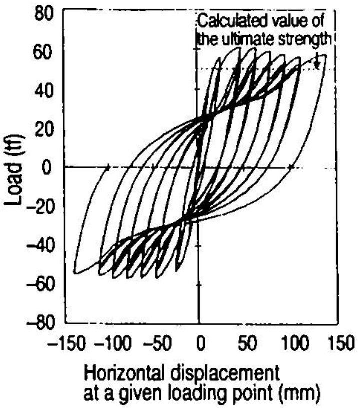
The specimens were 1:5 scale models of the real bridge pier. Two specimens were used: an *SC+PCa* Specimen with the H-shaped steel columns with ribbed flanges and an *RC* Specimen. In consideration of the different types of steel used in each specimen, the amount of the steel was calculated so that the both specimens had an equal bending strength. An equal amount of hoop reinforcement was installed in each specimen. In the *SC+PCa* Specimen, the hoop reinforcement was installed so as not to be in direct contact with the H-shaped steel columns.

3.2.2 Test results

Fig. 3 shows the load-displacement curves of each specimen. Both specimens maintained their strength which was greater than the yield load of the main longitudinal reinforcement until the displacement value of $7\delta y$. In particular, the *SC+PCa* Specimen retained about 57tf of strength which was close to the peak strength and greater than the yield load under the displacement value of $9\delta y$ which was the limit value of the loading instrument.



(a) *RC* Specimen



(b) *SC+PCa* Specimen

Fig. 3 Load–displacement curves

This test confirmed that the composite steel and concrete pier had the following structural performances:

(1) The hoop reinforcement has enough restriction effects on the H-steel columns with ribbed flanges without being in direct contact with them.

(2) The proposed composite structure has better deformability and higher seismic performance ductility than the reinforced concrete structure.

4. Construction Method

So far, three highway bridge projects have adopted the new composite steel and concrete structure. In this chapter, the construction of actual highway bridge piers with a hollow circular section exemplifies the construction method of this new structure. Fig. 4 is the schematic drawing of the bridge, and Fig. 5 is the structural drawing of P_2

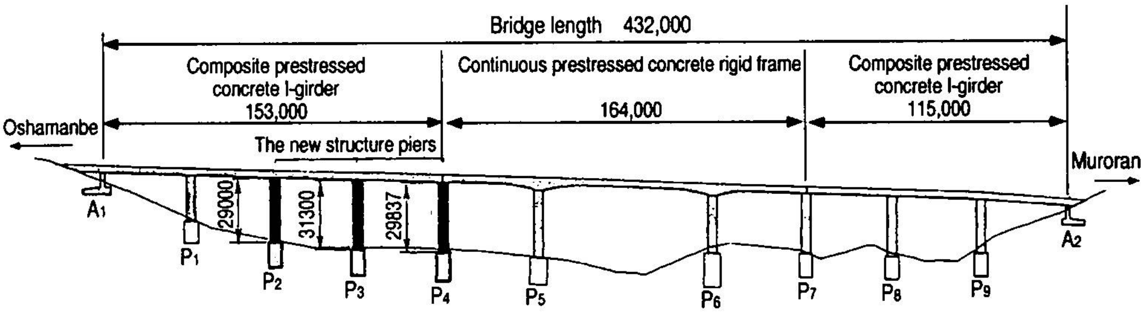


Fig. 4 Schematic drawing of the bridge

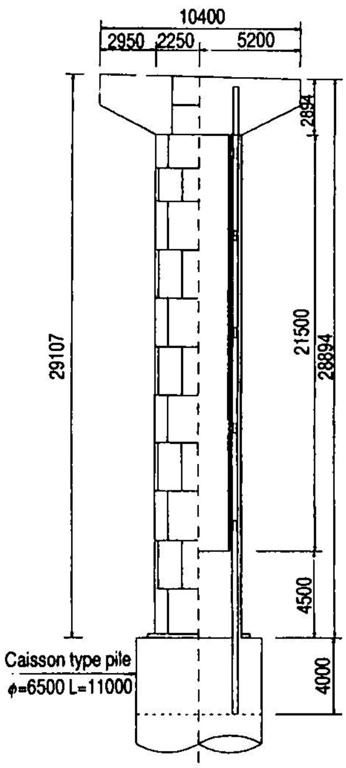
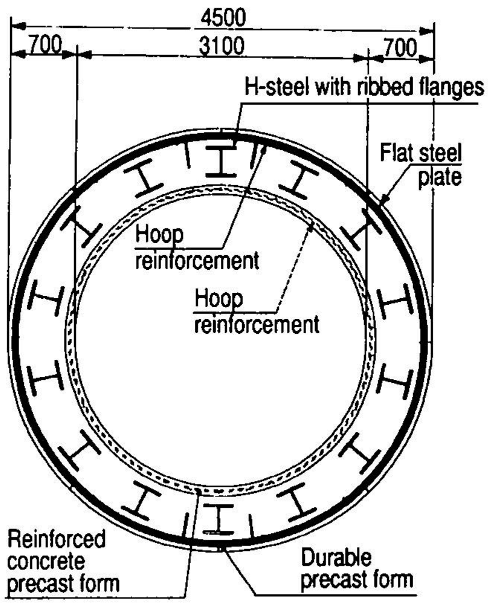


Fig. 5a Structural drawing of the pier 2



H-steel with ribbed flanges	H-318x313x15x24
Durable precast form	t= 50 mm thick
Reinforced concrete precast form	t=100 mm thick
Outer hoop reinforcement	D19@150
Inner hoop reinforcement	D19@250

Fig. 5b Standard cross section

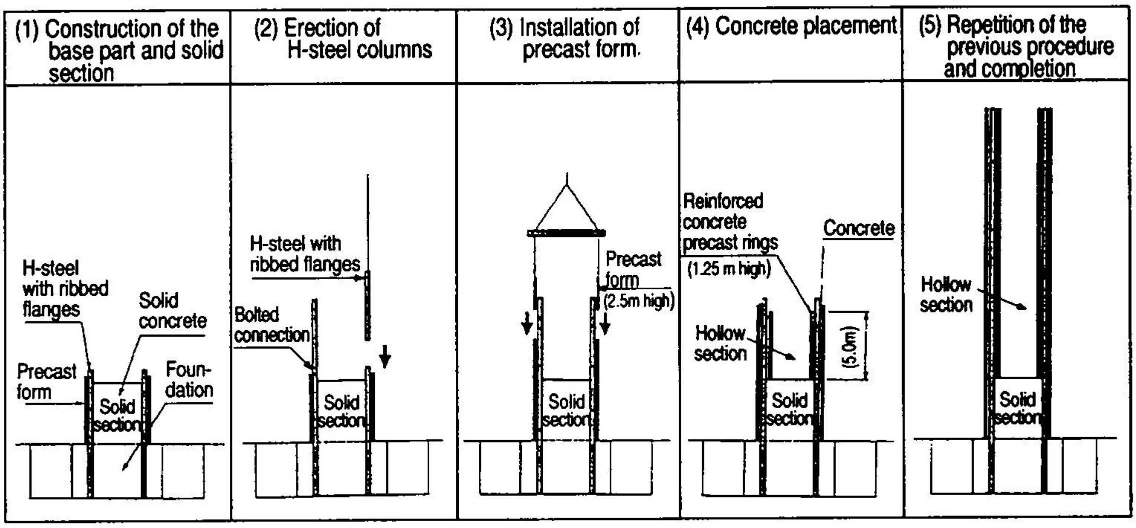


Fig. 6 Construction procedure

4.1 Construction procedure

Fig. 6 gives the conceptual drawings of the construction procedure.

(1) H-steel columns with ribbed flanges were positioned inside a caisson type pile to become the bottom section columns into which concrete was placed. The base part was then completed.

(2) The H-steel columns (5 m long) for the next section were erected, and then connected to the bottom section columns by high tension bolts.

(3) Precast forms were installed after being fabricated in the assembly yard on the ground (Photograph 1). The joints were bonded with resin mortar and sealing rubber which enabled both the transmission of compressive forces and sealing against water penetration under a tensile load.

(4) Cylindrical reinforced concrete precast rings were installed to become the inner form, and concrete was placed into the annulus.

(5) The procedure from (2) to (4) was repeated. After the pier shaft was finished, the pier head was erected and the construction was completed (Photograph 2).

4.2 Erection cycle

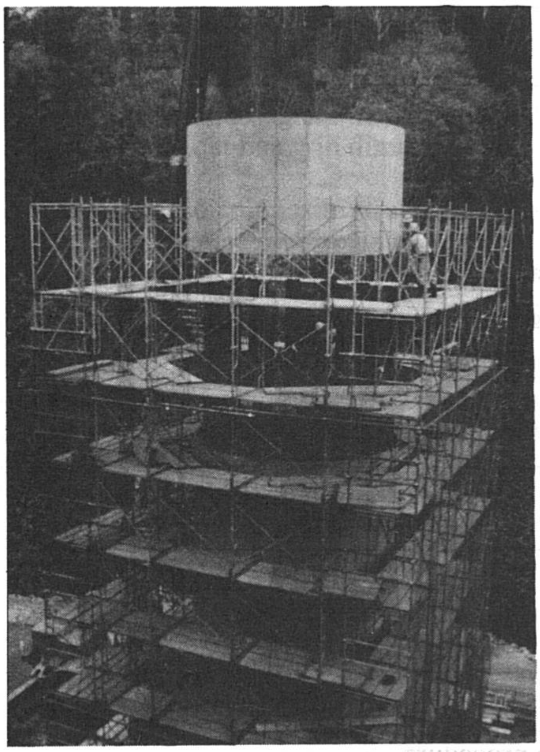
Table 1 shows the erection cycle of a unit lift (5 m), which was adopted when constructing P2, 3 and 4 piers in the project.

4.3 Results

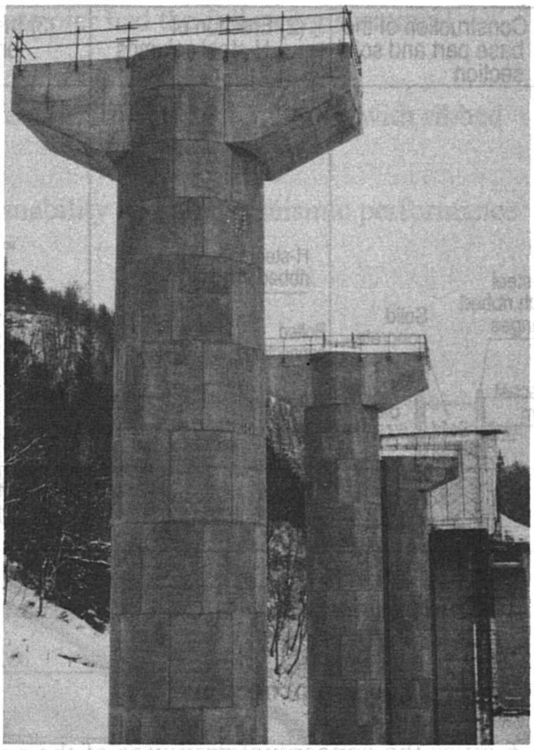
The following results were obtained by comparing the piers constructed in the new method with P8 pier erected in the conventional method:

(1) The construction of piers was 2.8 times faster in the new method than in the conventional method when comparing the average construction periods.

(2) Labor was reduced by about 40% as compared with the conventional construction method.



Photograph 1 Installation of precast form



Photograph 2 Completed bridge pier

Table 1 Construction cycle of the hollow part of the bridge pier (for one erection cycle of 5m)

		Day one	Day two	Day three	Day four
1	Fabrication of scaffolding				
2	Erection of H-steel columns				
3	Erection of precast forms				
4	Erection of inner reinforced concrete rings				
5	Concrete placement				
6	Clearing of laitance				

5. Future Objectives

In respect of the initial objectives, the shortening of the construction period by 66% has been achieved. With regard to the saving of labour, the following improvements can be made:

(1) To reduce the number of bolted connections by extending the unit of the H-steel columns with ribbed flanges to over 10 m.

(2) To rationalize the fabrication work of the precast forms, which occupies 33% of the total labour requirement to achieve a higher efficiency.

(3) To reduce the number of workers and time needed for scaffolding works as much as possible by systematizing the falsework, such as by employing travelling stages.

Truss Composite Bridges

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Summary

Out of the variety of all possible sorts of bridge construction in composite design the truss composite bridges are viewed more closely with regard to their constructive details. The profitabiliy of such constructions in fabrication and maintenance is pointed out. By means of projects which are already carried out it is gone in the special qualitiy of such bridges. Different erection methods, in special the erection of Innbridge Königswart, are presented.

1. General

If the two materials steel and concrete are combined in a functional way, composite main structures generally can be designated as an advanced method of construction. Economic constructions with low maintenance costs too result essentially from using the material concrete for compression stressed supporting elements and the material steel for shear and tensile stressed elements of a construction.

Out of the large spectrum of all possible bridge constructions I would like to pick out the truss composite bridges described in the following. The special way of construction where the steel top chord is surrounded by concrete will be explained particularly. For the first time this principle was applied in Germany as an advanced solution in steel railway bridges construction: The bridge Nesenbach Viaduct paved the way for many following projects in this type of construction as for example the Isarbridge in Großhesselohe, the Fuldatal bridge Kragenhof in Kassel and many other bridges for railway- and road traffic. Some of these kinds of bridges are mentioned in this paper, regarding only those, with one or more special qualities as described below:

2. Special Qualities of Truss Composite Bridges:

2.1 Fully welded construction with top chord surrounded by concrete.

On the basis of designing reasons the welded construction takes priority because of the plane surfaces resulting from welded constructions (on the contrary to screwed connections). The omission of joint plates causes not only a contribution to an easy way of production but also advantages in maintenance. Figure 1 shows the lower chord with the transition to the diagonal members. The plain transition in the node area is well to be seen.

At the Nesenbach - Viaduct (1) the top chord was embedded in concrete for the first time. Figure 2 shows the principal situation that the diagonal connections are also in the concrete girder, which results from a reinforcement in the area of the steel top chord. By total embedding the steel girder no weakening of the cross section takes place. Therefore always open sections like U- and H-sections are used for steel chords.

The transfer of the shear forces happens continously by a spreaded three sided arrangement of the stud shear connectors. In order to avoid any sudden concentrated force transfer, the connection of the diagonals to the top chord embedded in concrete is done with fillets analogous to the lower chord. Figure 2 shows how the transition between steel and concrete is demarcated by a horizontal metal sheet. This simplifies the maintenance for this detail. Corrosion damages are avoided considering a possible cross bending of the diagonals at this joint.

Only secondary tensions in the steel girders are caused by a nearly total accordance of the system lines of steel top chord and the concrete system of the top chord locally stressed by bending und longitudinal forces. This important aspect leads to exceptional economic steel masses.
Besides the mentioned technical details truss composite bridges have some important advantages compared to other types of construction: Apart from the economic advantage in erection and maintenance the aspect of environment has to be mentioned. Because of the low sound emission and the attractive design of the main structure the influence of such constructions on the environment is minimized.

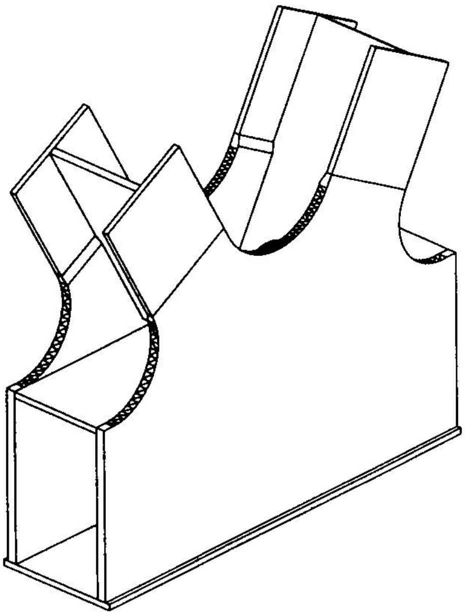

figure 1
(lower chord with the transition to the diagonal members)

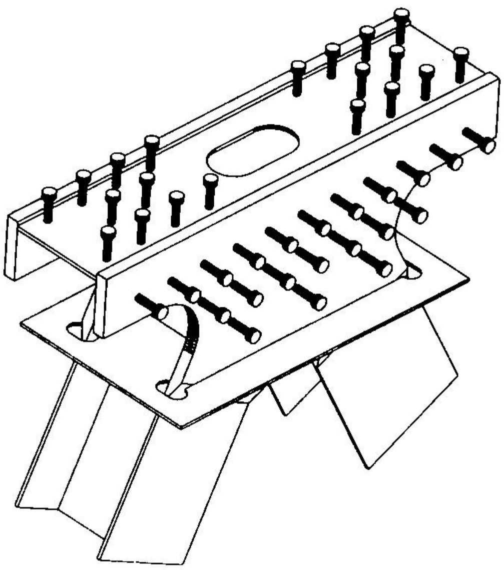

figure 2
(the principal situation that the diagonal connections are also in the concrete girder)

2.2 Sound emission and characteristic vibration

By means of ballast and the underlying concrete slab, thickness 40 - 60 cm, the bridges have a far bigger mass concentration under the rails in comparison to orthotropic steel slabs. Exactly this mass concentration in the sector of stimulating forces (wheel-rail-system) causes a convenient transfer of vibration energy to the remaining construction. The ballast und the concrete slab affect the resonance attidude in a positive way by means of their high damping property. Additional big slab fields, which maybe could respond inconveniently to the vibration transfer, are not caused by the truss girder construction. A convenient vibration attidude relating to the sound emission is caused by continous interruption of the stereoscopic fields and by the comparatively stiff nodes of the truss composite construction.

Such bridges can justly be named „whisper bridges" as the sound emission on the bridge is not bigger than between stations.

2.3 Visual pollution controll

On the contrary to solid girder constructions the truss girder construction does not form such a strong optical barrier by means of a higher transparency. Demonstrated in many examples such bridges fit harmoniously in our landscape. The method of truss composite construction fulfills the demands of clients for utmost design preservation when replacing an old steel bridge by a new truss girder.

3. Selected bridges with different special qualities.

3.1 Innbridge Königswart.

In 1874 a truss composite bridge was erected in Königswart over the river Inn with the participation of the famous Bavarian bridge engineer Dr. Heinrich Gerber. The building owner, the Deutsche Bahn AG, decided for a new construction including the revamping of the piers. For the new construction a continous beam system built up as a truss deck bridge in the way of steel composite construction was chosen. This way of design pays attention to the visual pollution control because the truss fits harmoniously in the wonderful landscape. The span of the three fielded continous beam is 89,7m, 69m and 89,7m with a total length of 248,4 m (figure 3) between the abutment axes. The steel construction of the jambless strut frame has a constant hight of 6630mm (figure 4).

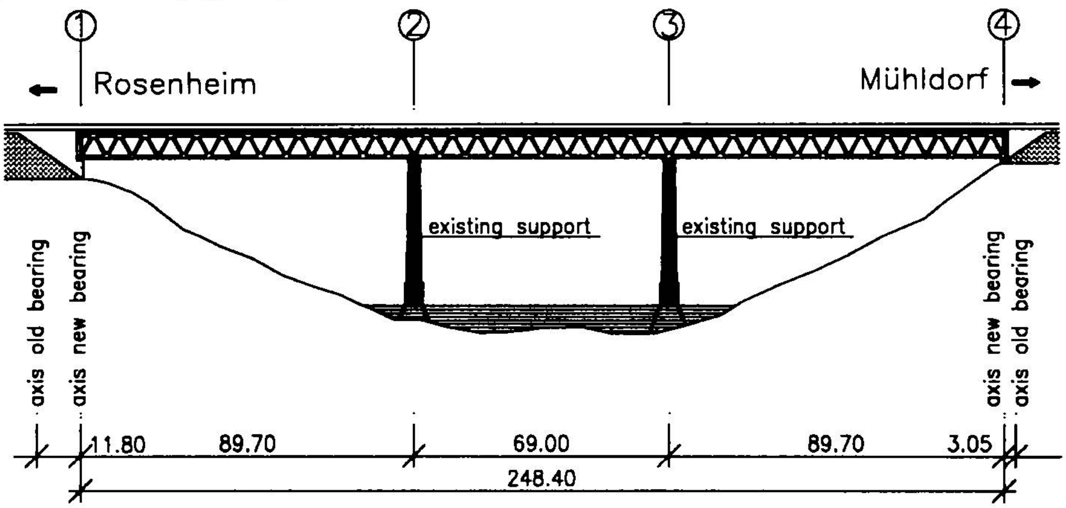

figure 3 (longitudinal view of Innbridge Königswart)

The system hight of the complete main structure is aprox. 6,3m. Therefore the relation of span to systemhight is calculated with aprox. 14 or 11 respectively. The center to center distance of the one truss girder structure is 4m. The steel mass is aprox. 810 to, that is equivalent to aprox. 3,3 to/m. At this bridge the top chord passes outside the concretee slab on the contrary to the bridges described in here. The bridge is also a completely welded construction. Above all in this project the erection plan is very remarkable. This will be discussed in a few words because of the special quality and the given construction situation.

The individual essential phases of erection are shown by means of dias. At the pre erection site behind the abutment in the bridge axis the delivered elements were assembled to large elements sized 90m, 70m and 90m. Afterwards this elements were moved and taken over from the transporter using temporary frames and launching beams. Now the three bridge elements were transported over the existing old bridge, moved to the final position and were welded together to the three field continous girder system. The new main structure was lifted and fixed in the bearing axis over the old bridge with so called „equipment towers" with cross beams making use of hydraulic lower jacks. Afterwards the existing old main structure was removed with transport units running on the top chord of the new main structure. Then the new main structure was lowered to the final position by lower jacks and put on the bearings. The concrete plate was produced in 18 concrete units.

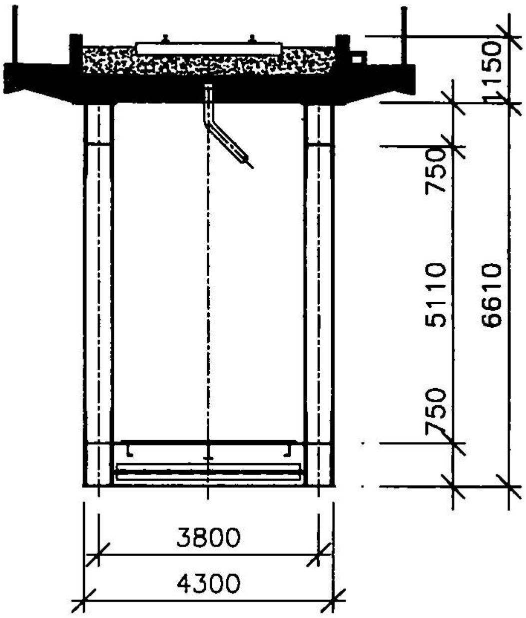

figure 4 (cross section of Innbridge Königswart)

Through this fast method of construction the environment was not much bothered with dust and sound. This interrupted the traffic only short. Through a high mass concentration in the ballast and through the composite slab the sound emission is reduced. Therefore the bridge can be called whisper bridge .

3.2 Railway bridge across the river ATTEL

This bridge, located close to the Königswarter Bridge is mentioned because the design of the composite slab is different to the one described before. The existing old main structure, built in 1873, fails to meet today's requirements and had to be renewed together with the substructure. The new main structure, a truss composite construction, is constructed with a concrete deck (figure 5). The span of the one fielded main structure is 60m. The height of the system is 4,3 m, with a distance of 2,7 between the truss levels. The mass allocation, 2,5to/m, is calculated from a weight of 152to.The cross section of the bridge is a two flanged T beam.

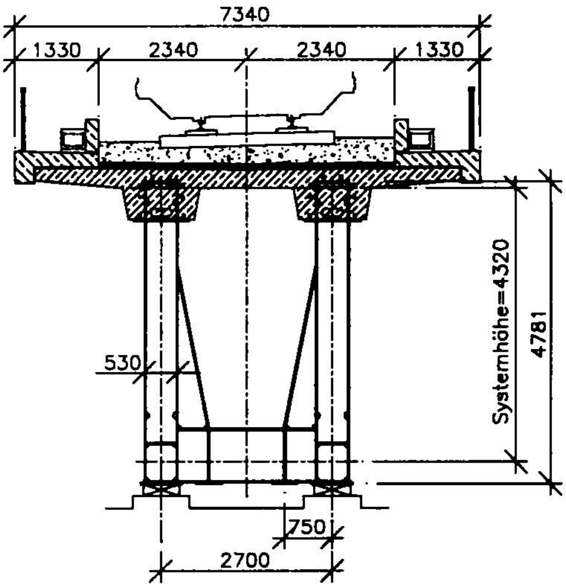

figure 5 (cross section of Attel Bridge)

The flanges are formed out of total welded jambless truss girders. The cross section of the 30cm sized concrete slab is constructed in such a way that the centre of the steel top chord is identical with the concrete slab. Welded open sections were used for top chords and diagonals, whereas the lower chord was constructed as an airproof welded box section.It is remarkable that it was possible to avoid a strenghtening of the joint plates in the initiation sector due to an appropriate selection of the plate thicknesses of the lower chords. This results in a functional and also visual pleasing detail solution because of the plane surfaces. Both in longitudinal-and cross direction the concrete slab is singly reinforced. In the sector of the truss top chords 50 cm high and 120 cm wide beams are planned which surround the steel chords totally.

In the following the method of erection and a rarely used method of dismantling are mentioned.

Parallel to the old main structure the new bridge is erected on cross adjustment ways. The truss walls are supported horizontally to the old wing unit, therefore it was possible to omit the very complicated additional bracing. After completion of concreting the couplings to the old main structure were removed. Following unusual dismantling of the old bridge was selected after finishing the parallel situated new main structure: The old bridge including abutments was removed by use of directed blastings. Afterwards the new bridge was traversed in the final position and put on the bearings. For erection of this bridge the old main structure was used as an important part in the erection concept.

3.3 Taggenbrunner Viaduct

Tenders for a new building were invited including technical and economic aspects, caused by the age of the bridges (built in 1905 and 1911) and the resulting efforts in repair. Attention was payed to the integration of the new main structures into the landscape. The old appearence of the bridge was to change only slightly. Therefore it was decided to leave the position and design of piers, of abutments and of parts built with bricks, to revamp them and to replace only the superstructure. (2)

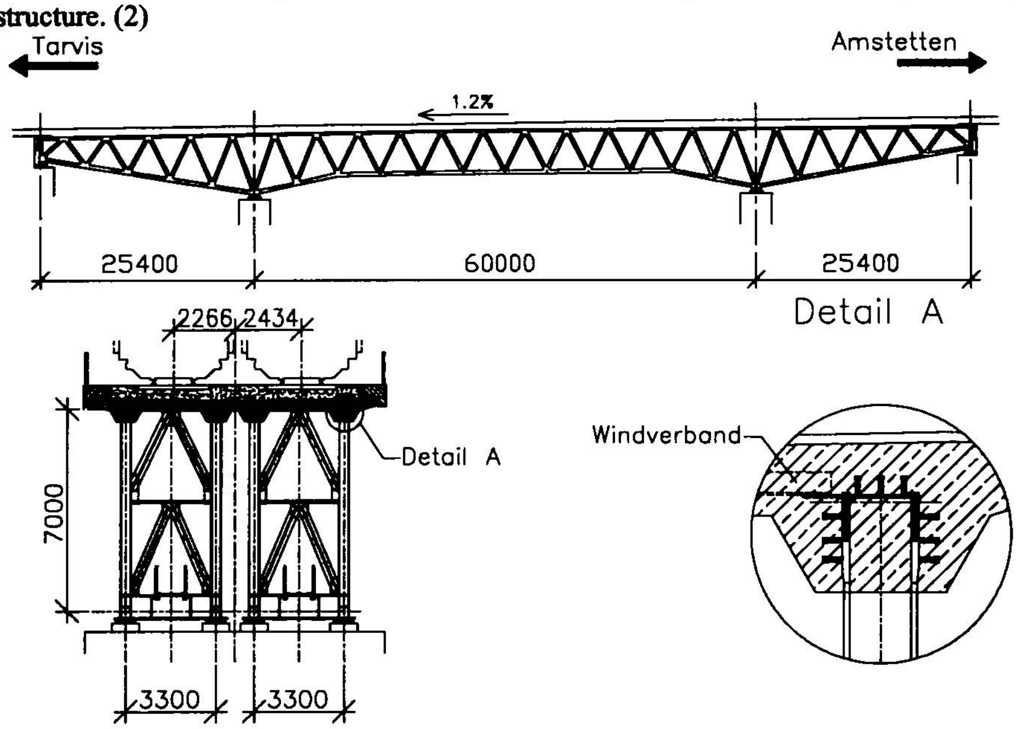

figure 6 (longitudinal and cross section of Taggenbrunner Viaduct, detail of upper chord)

In several intermediate stages in the competitive procurement procedure, in close cooperation with the client, the Austrian Railway company, a solution for the construction was found, which was applied for Austrian railway bridges for the first time. A system of contious beams was designed in the way of truss composite construction. For this double rail track again two single rail main structures were erected with the prescribed spans of 25,4 m, 60, and 25,4m caused by the old piers (figure 6).Therefore the total lenght between the abutments is 110,8m. The truss girders are arranged in a distance of 3,3m. The system height of the composite main structure varies from 5m in the middle of the span to 7m at the supports. The weight of both main structures amounts to 465 to. The mass allocation of the steel structure results in 2,1to/m per rail As a structural special quality I would like to remark, that welded U-sections were used for top and lower chords and rolled sections were used for diagonals. Therefore the members outside the concretee slab are open sections without exception. The site connections of top chord and lower chord are welded. Secondary truss elements were screwed to the main construction. For simplification of the erection, the chords of the diagonals were welded, whereas the webs were screwed. The top chord is completely embedded in the concrete slab and therefore it is placed in the system line of the composite slab. This way of construction practiced in other european countries was used in Austria for railway bridges for the first time. This construction leads to an economical steel mass possible despite of the unfavourable proportion of spans caused by the existing piers.For maintaining the traffic in this line at least on one rail it was necessary to dismantle respectively assemble the main structures one after the other. The new bridge was assembled on the ground parallel to the old bridge, afterwards the new bridge was lifted with jacks and traversed in the final position. After this the formwork was produced for the whole length of the bridge and the composite slab was concreted. After transferring the traffic to the new main structure the second main structure was erected in the same way. By selecting a bridge system with two separate main structures per rail it was possible to reduce the interruption of traffic to a minimum.

References:

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Composite Arch Bridges Developed in China

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Guomin Yan, born 1925, BSc in civil engineering from Tsinghua University in 1949. As bridge designer for more than 40 years, retired in 1990 and turned as professor up to now.

Summary

This paper mainly describes the development of composite arch bridges in China.Three types will be demonstrated. The first is solid arch ribs composed of steel and concrete, second is arch truss ribs with their chord members composed by steel pipes and concrete , and third is reinforced concrete arch ribs or ring supported by composite arch truss structures in the construction and reinforced by the same one after completion. All these arch bridges are very successful, especially for the longest span RC arch bridge in the world.

1. Introduction

Within the years of recent decade, composite arch members used in Chinese highway bridge structures were specially successful. They were designed in different types, such as deck bridge, through bridge, half-through bridge and tied arch bridge, all with rise span ratio from 1/6 to 1/4. Most of the arch members are composed by steel pipes and filling concrete, and some are composed by stiff steel arch truss and the concrete arch rib or ring itself. All these composite arch bridges are mentioned in the following by some typical bridge examples.

2. Bridge with Two Solid Composite Arch Ribs

Bridges of this type are designed with two solid arch ribs composed by steel and concrete. The cross-section of each arch rib is shown by Figure 1, the vertically arranged steel pipes and two steel web plates with concrete filled both into the pipes and the space between two web plates and two pipes to form up as a dumb bell type. Between the two arch ribs some necessary bracings are arranged.

2.1 Examples of Through Bridge (Fig. 1)

The first example is Wang-Cang Bridge located in Sichuan Province. Over its main span is a tied composite arch structure with net span length of 115m and rise/span ratio of 1/6. Each composite arch rib is composed of two ∅800×10 mm steel pipes with center distance of 1200 mm and two steel web plates with thickness of 10 mm .The bridge width is 15 m, in which 7 m for two lanes of roadway , 2×3m for walkways and 2×0.8m for arch ribs. This bridge was opened in 1990.

The second one is Fo-Chen Bridge located in Guangdong Province. Over its main span is also a tied composite arch bridge with net span length of 110.3m and rise/span ratio of 1/5.The cross-section of each arch rib is more heaviver than that of Wang-Cang Bridge due to more wide it is. Each with two ∅1000×14mm steel pipes with center distance of 1500mm and two steel web plates of thickness 12mm .The bridge width is 26m, in which 17m for 4 lanes of roadway, 2×1.0m and 2×3.5m for arch ribs and sideways respectively. It was opened in 1994.

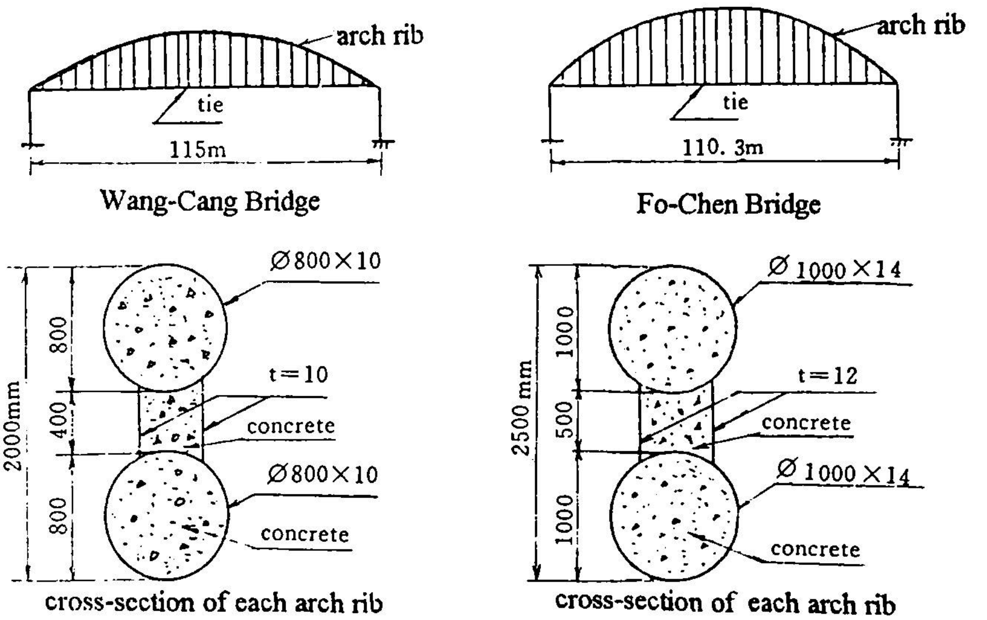

Figure 1. Wang-Cang Bridge and Fo-Chen Bridge

2.2 Examples of Half Through Bridge (Fig. 2)

The first example is Gao-Ming Bridge located in Guangdong Province. It is a two lanes bridge(12m wide only) with net spanlength of 100m and rise/span ratio of 1/4. Each arch rib is composed by two ∅750×10mm steel pipes with center distance 1250mm and two steel web plates with thickness of 10mm .It was opened in 1991.

The second one is Mo-Zi-Wan Bridge located in Chengdu City , the capital of Sichuan Province. This bridge is now under construction with net spanlength of 120m and rise/span ratio of 1/5.5. Its arch rib is designed to use two ∅800×12mm steel pipes with center distance 1200mm and two steel web plates with thickness of 12mm . Its total width is 7.5 m only with 7m for two lanes and without walkways.

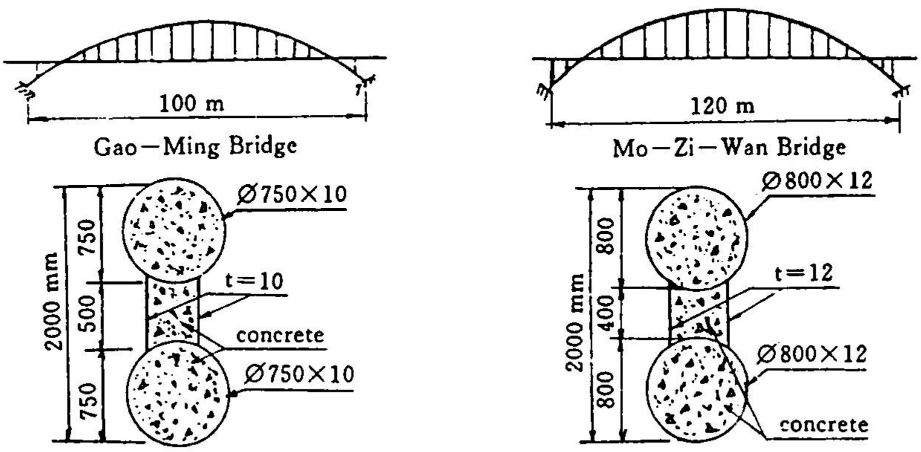

Figure 2. Gao-Ming Bridge and Mo-Zi-Wan Bridge

3. Bridge with Two Composite Arch Truss Ribs

Bridges of this type are designed with two composite arch ribs and some necessary bracings between the ribs.Each composite arch rib is composed of 4 steel pipes arranged at 4 corners of a rectangle or trapezoid and some web members and horizontal members. 4 steel pipes with filled concrete perform the 4 composite chord member of the arch truss rib.

3.1 Examples of Through Bridge (Fig. 3)

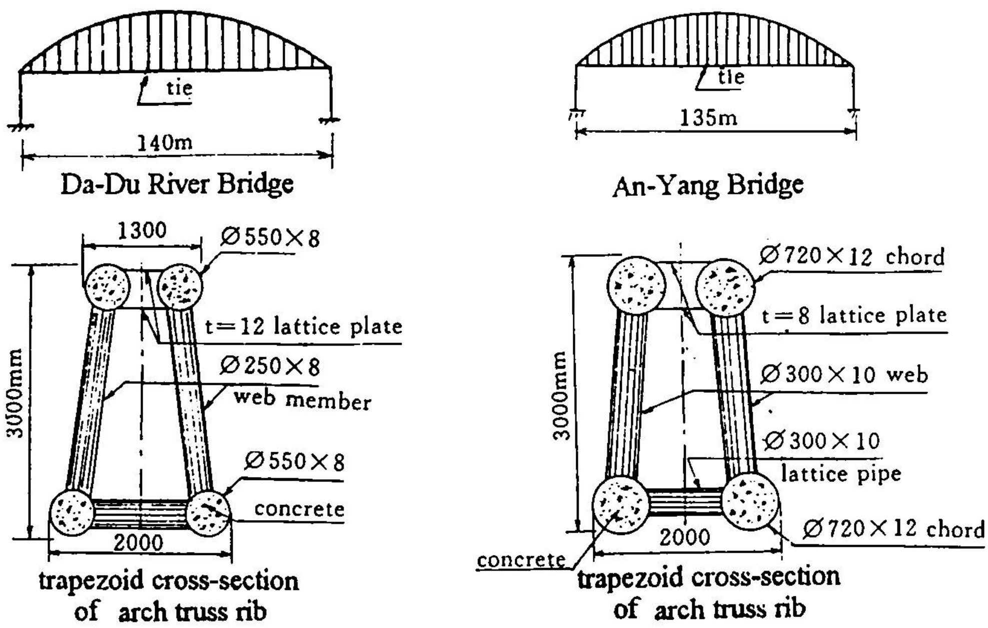

Figure 3. Da-Du River Bridge and An-Yang Bridge

The two examples are Da-Du River Bridge located in Sichuan Province and An-Yang Bridge located in Henan Province with net spanlength 140m and 135m respectively, and both the rise/span ratioes are 1/5 . Each composite arch truss rib is composed of 4 corner steel pipes filled with concrete performed as both upper and lower chord members , their web members are made by hollowed steel pipes, as well as the horizontal members between the two lower chords, two horizontal lattice plates are used to connect the two upper chords.The total width of bridge deck are 13m for 2 lanes and 31m for 4 lanes respectively. Therefore , the composite arch truss rib of later is heavier than the former.The detailed dimensions of their trapezoid cross-section are shown in Figure 3 respectively. Da-Du River Bridge was opened in 1995 and An-Yang Bridge is under construction.

3.2 Example of Half-Through Bridge (Fig. 4)

The San-Shan West Bridge located at Nanhai City in Guangdong Province is an example of this type.It is a half-through tied arch structure with the spanlength arranged as 45+200+45m.The rise/span ratio of the central arch is 1/4.5.Total width of the bridge deck is 28m ,in which 15m for express way,2×3m for trucks and 2×1.5m for pedestrian. Each composite arch truss rib is composed of 4 steel pipes located at 4 corners of a rectangle and both the hollowed steel pipe web membes and double horizontal plate members. Concrete is filled both in 4 steel pipes and the spaces between double horizontal plate upper and lower .Detailed dimensions are shown in Figure 4. This bridge was opened in 1995.

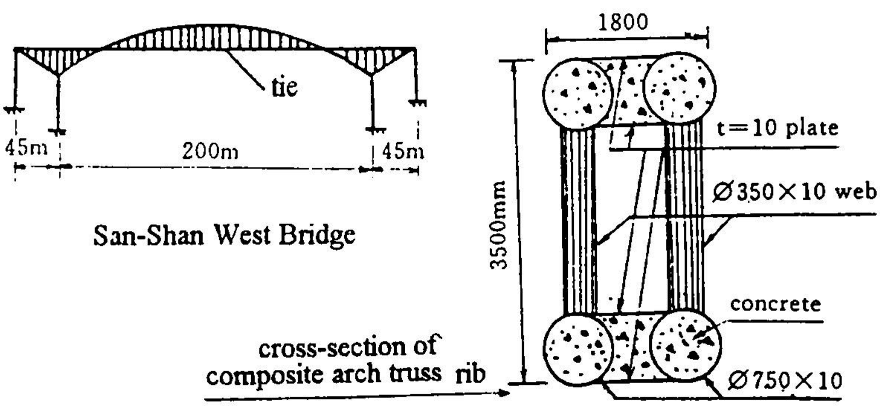

Figure 4. San-Shan West Bridge

4. RC Bridge with Composite Arch Truss Ribs or Ring Acted as Support System in Construction and Stiff Reinforcement after Completion

For this type of bridges,composite arch truss is designed first to support the concrete arch structure in construction stage, and then to be composed by the former and later,and performered a role of reinforcement of the permanent arch structure. The composite arch truss can be both in the types of two ribs and one ring.

4.1 Examples of Half-Through Bridge (Fig. 5)

In 1993, a half-through arch bridge named New Lon-Ao with net spanlength of 117.8m and rise/span ratio of 1/4 was completed and opened to traffic .Its total width 22m is composed of 0.5+9.75+1.5+9.75+0.5m for dual 3-lane of roadway.Two arch ribs each was first erected by a arch truss rib with 4 steel pipes of $\varnothing 300\times 13$mm as its chord members ,and then concrete was filled into pipes to form up a composite arch truss structure, subseqently this structure played a role as support system to take the weight of concrete arch rib with the cross-section in rectangular box type , and finally it was embeded in the concrete arch rib as its stiff reinforcement.

In 1995, another similar bridge named Luo-Guo was opened over the Jinshajiang River in Sichuan Province with a more longer net spanlength of 160m and rise/span ratio of 1/4. Compared with New Lon-Ao Bridge ,the diagonal hangers are changed to vertical, and the depth of arch ribs is increased from 3.0m to 5.4m,and the thickness of flanges and webs is increased from 34 to 37cm and 30 to32cm respectively, also the deck width is decreased from 22m to 15m .The construction method and other structure arrangement have no any change.

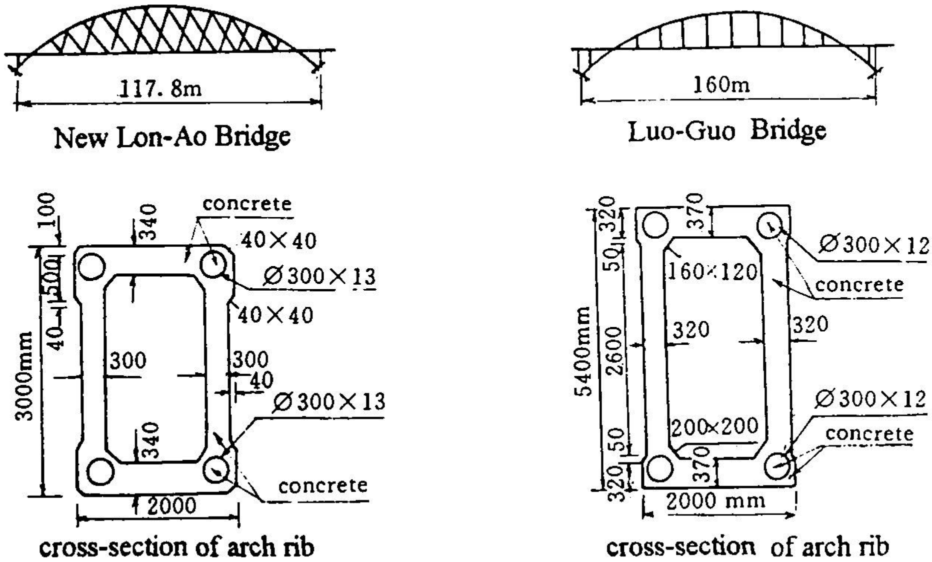

Figure 5. Lon-Ao Bridge and Luo-Guo Bridge

4.2 The World's Largest Span RC Arch Bridge (Fig. 6)

Wanxian Yangtze River Bridge is now under construction and will be soon completed at July of 1997.This bridge with net spanlength of 420m will be the longest one of RC highway arch bridge in the world.It is a deck arch bridge with rise/span ratio of 1/5 and total deck width of 18m , in which 15m for 4 lanes of roadway and 2×1.5m for walkways. The cross-section of the RC arch ring is a 3-cell box with 7m in depth and 16m in width.

The first work was to erect a stiff steel arch truss ring which is composed of 5 plane-truss with spacing of 3.8m from each other. The depth and width of this steel arch truss are 6.45m and 15.2m respectively .10 ∅400mm×16mm seamless steel pipes are used for upper and lower chord members .All web members and bracings are made of shape steels. After the steel arch truss ring was closed at its crown, pumping concrete was then filled into the 10 steel tubular chord members to form up a very stiff composite arch ring structure.

The whole cross-section of the concrete arch ring is divided into7 areas and cast in 7 steps .The stiff composite arch ring was used to support the weight of concrete cast in the first step.The composite arch ring structure composed of the stiff composite arch truss ring and the first area of concrete arch ring was then used to support the weight of concrete cast in second step, and similarly, the concrete cast in No.n step was supported by a new composite arch structure composed of the stiff composite arch truss ring with (n-1) areas of concrete arch ring. The whole concrete arch ring will be thus cast step by step until the whole composite arch truss ring will be embeded in the concrete as stiff reinforcement.

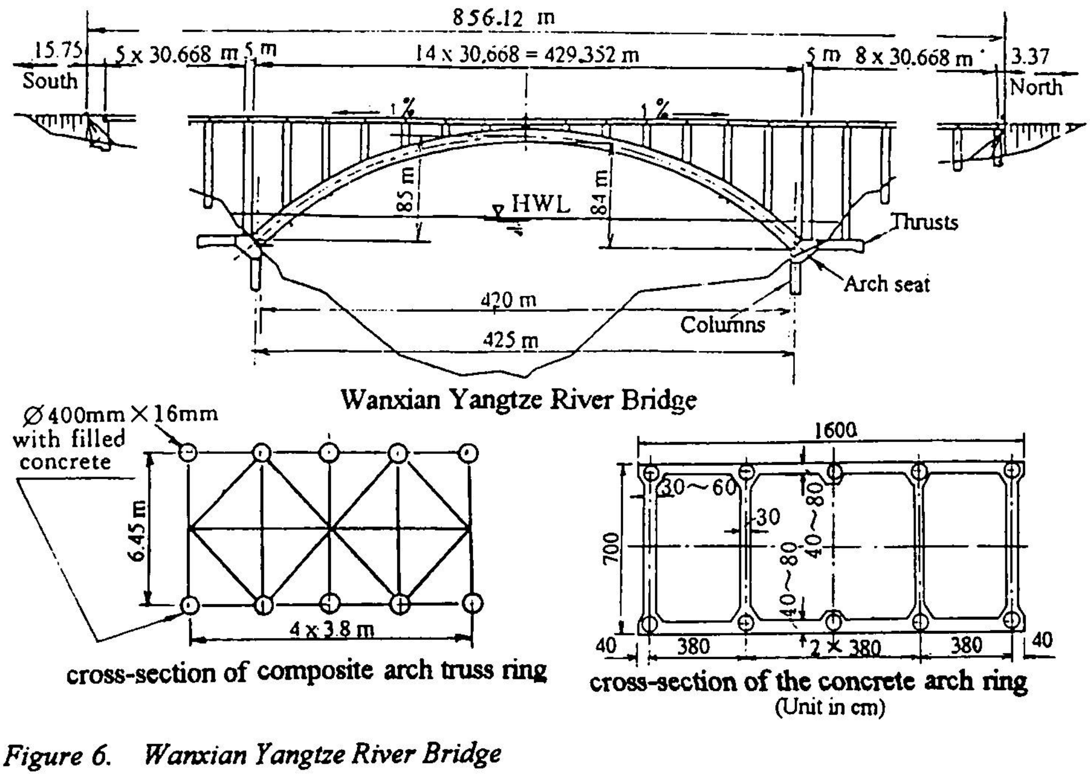

Figure 6. Wanxian Yangtze River Bridge

5. Conclusion

Construction of the Wanxian Yangtze River Bridge was commenced at May 1st of 1994,and it will be hoped to be opened at July, 1997. It may be said that the composite arch bridge developed in China is very interesting and successful.

Steel-Concrete Structures in 24 Storey Bank Building in Bratislava

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Summary

Last trends in construction technology of multistorey buildings. Steel skeleton stiffened by means of concrete elements. Fire protection of steel parts solved by embedding concrete. By a combination of various material-structural elements and technology an optimum could be achieved. At VÚB-Center Building constructed in combined technology with dominance of concrete have been solved by means of steel in the most exposed parts: the inner piers in ground floor cantilevered transverse plate with embedded steel grid, the statically most exposed facade columns.

1.Structural and construction technology of multistorey buildings-last trends

The multistorey buildings for civil and industry purposes could be realized in various structural, material and construction technology. It is to accent the great liaison of all these components. A competion exists among the construction systems, concerning the choise many and various criteria are important, often of conjuctural character and different in the countries. In steel skeleton some development trends could be distinguished:

1. Pure steel skeleton is rarely used, the spatial stability is mostly achieved by means of concrete elements, e.g. by concrete floor stabs, shear walls and cores.
2. The minimalisation of structural parts designed in steel - e.g. steel-sheet and r. concrete floor slab substituted by r. concrete stab, beam elements and/or columns by composite ones
3. The minimalisation of steel parts which have to be protected against corrosion and fire. The most simple and cheap solution is their embedding with concrete.
4. To enable the joining at concrete structures the use at progressive components - in formwork, reinforcing, geometry accuracy, precasting etc.
5. As last trend to win an optimal solution is a mixed structural system using various material-structural parts and/or elements and by following in-situ concrete to achieve a stiff monolitic structure. The following described building is realized in that technology.

2. VÚB - Bank Center building in Bratislava

2.1 Generally about the building

In Bratislava a 24 storey bank building has just been finished, where a combination of steel-concrete structures has been used (Fig. 1). The layout has the ellipse form with main dimensions of 25,2 x 48,0 m with the communication and instalation core 10 x 18 m in the excentrical position (Fig. 2). In the cross direction are the rigid frames in 6 m distance with middle span of 9,6 m, enough flexible in horizontal direction, so that their participation on carrying horizontal loading is negligible, all effects of horizontal loading is carryied by the core with joined shear walls supplemented by torsion forces. The top part of the ellipse is in the ground floor cantilevered 6 m.

For bearing structures a mixture of various material technology has been used:
- in-situ concrete: the core, shear walls and the inner columns
- in precast concrete: the peripheral elliptical parapeth girders, most facade columns
- in composite concrete: the floor structures with prefabricated lower part and in-situ concrete upper layer (slab and cross beams)
- composite steel - reinforced concrete: the inner columns in 1st and 2nd floor, the transverse plate over 2nd floor carrying cantilevered part of the building, the most statically exposed facade columns.

In this paper we pay attention to the composite steel-reinforced concrete structures.

Fig. 1. View on building during erection

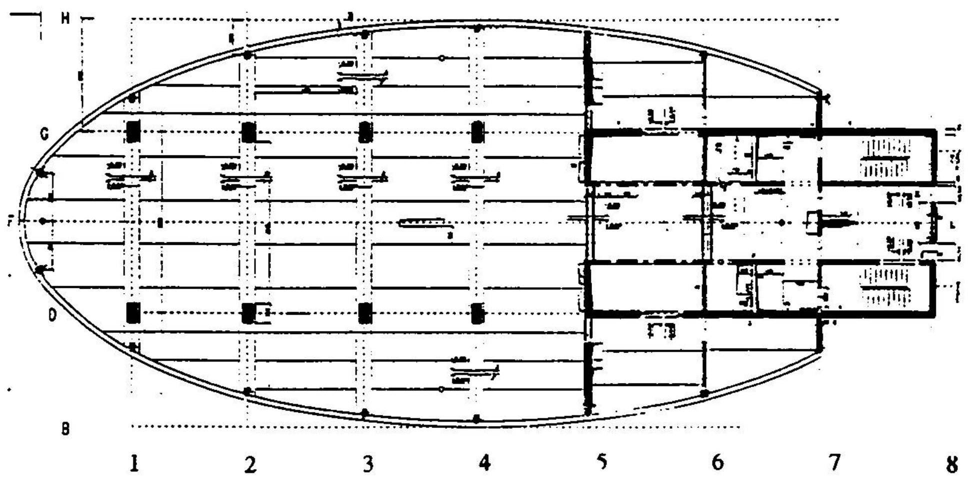


Fig. 2. Layout of typical storey

2.2 Composite inner piers

The inner piers in the ground floor are extreme loaded at axis 1 by an axial force of N_d = 26 800 Kn and in the axis 2 and 3 by N_d = 16 800 kN and demanded small dimensions of 1200/600 or 1000/500 mm respectively. If the need of structural clear seating of steel grid in transverse plate is respected, it is suitable to use for piers the composite steel-concrete form (Fig. 3). The piers have two welded I profiles which carry about 50% of designed normal force. The piers are boarded with a steel plate stiffened by vertical plates, the reinforcing bars are joint to them by welding. On the site were delivered the steel structures of piers included of reinforcing bars completed in two storey length.

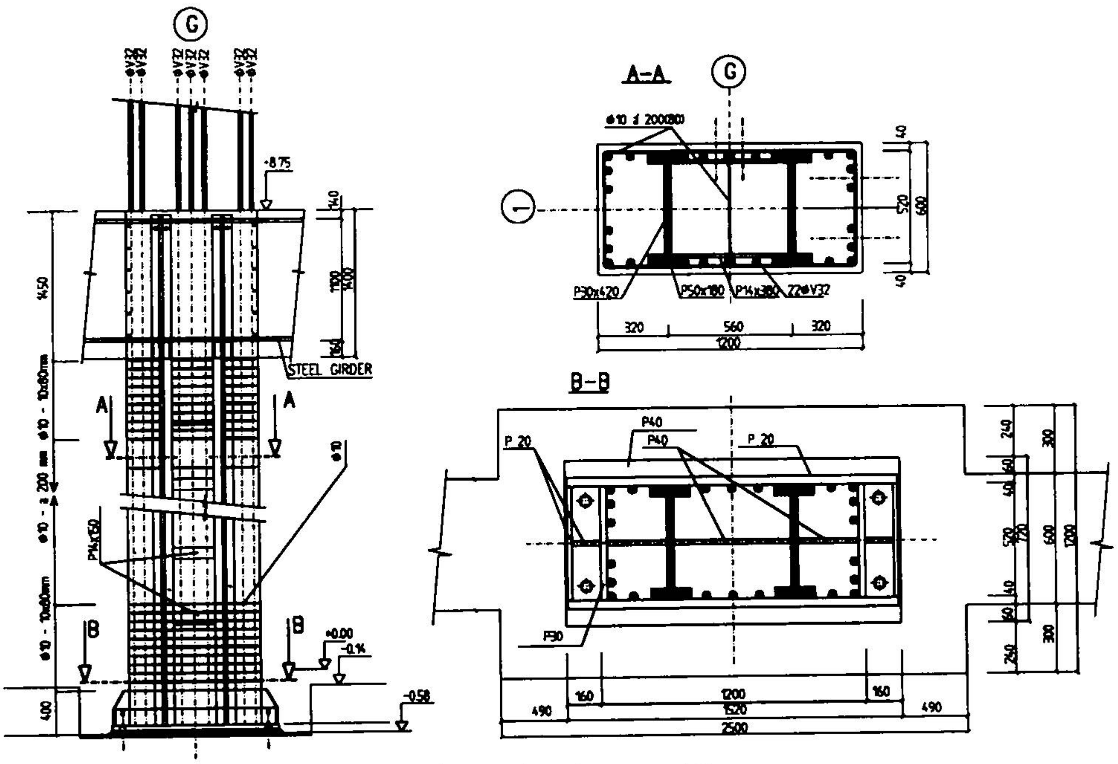


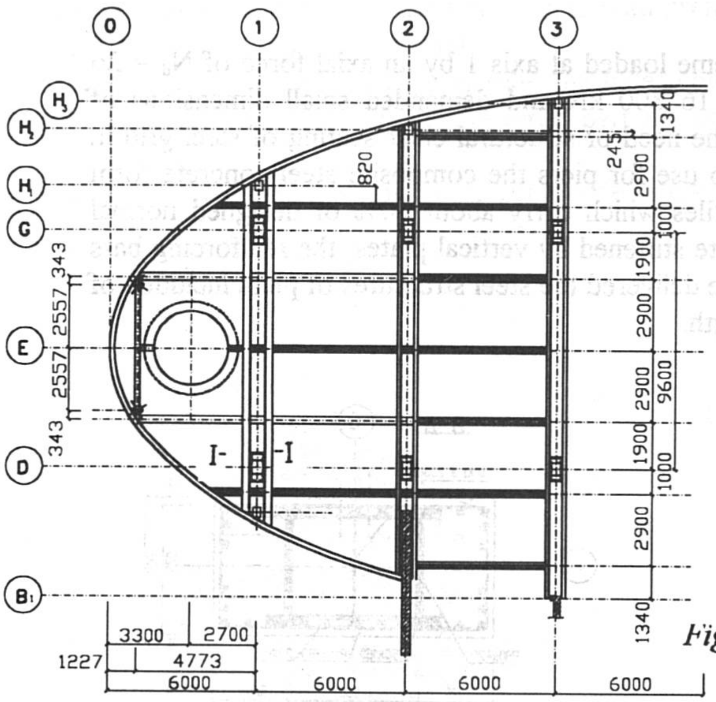
Fig. 3. Composite inner piers in ground floor

2.3 Cantilevered transverse steel-concrete plate over 2[nd] storey

Four facade columns situated in modulus 0 - 1 acting with normal forces about 5000 kN are based on the elliptical transverse plate. The statically exposed transverse plate demand the height of 1,20 ÷ 1,40 m, similar to the upper peripheral girders. The designed facade coating demands a very strong stiffness (the deflection of the top $y_{max} \leq 20$ mm, declination on the periphery tg $\chi \leq 0{,}0015$). The site and construction conditions were not good for prestressed r. concrete, therefore the following solution was elected (Fig. 4):

- steel grid of 1,10m height in two fields with 6 m cantilevered console, elliptical layout, seated on composite piers, built from transversal and longitudinal girders dimensioned on dead load of 1,40 m concrete slab
- precast lower plates of 90mm height suspended on the steel girders dimensioned on the 0,4 m concrete layer
- in-situ concrete with reinforcement bars concreted in two steps.

By this solution is achieved: quick construction tempo (no strong formweek, no technology break), precise geometry.



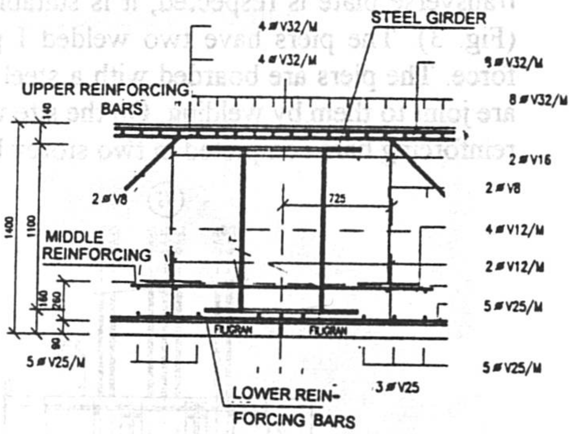


Fig. 4. Cantilevered transverse steel-concrete plate:

a - layout of steel grid, b - cross section

2.4 Precast facade columns

In spite of the normal forces in the columns varying from 1500 kN to 6200 kN the same rectangle size 30.40 m was elected.
The corresponding bearing capacity was achieved at constant total area $A=A_a + A_s + A_b$ by combination of six parameters (index b - concrete, s - steel reinforcement, a - steel profile)
The total compression capacity was determined

$$N_{tot} = N_a + N_s + N_b = (A_a R_a + A_s R_s + 0{,}85\ A_b R_b)\ \delta$$

where A-cross section area, R-design strength, δ-buckling factor (expressing the influence of slenderness and of imperfections and loading excentricity). The minimum compression capacity is achieved at cross section with constructive reinforcement (A_s = 0,006A), the middle one by higher quality of concrete and strong reinforcement and the max compression bearing capacity by addition of inner steel profile (Table 1).
The columns have been constructed as in shop prefabricated element. Most columns are of two storey length, fitted with short console for placing facade elliptical girders and crossing girders. The contact joint is solved by means of steel boarding plate and screws (Fig. 5).

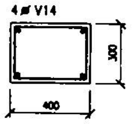
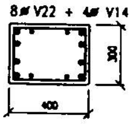
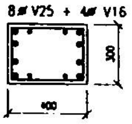
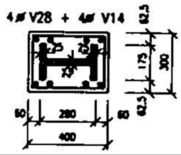
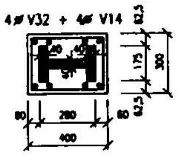
TYPE	CROSS SECTION	AREA [cm^2]	R_i [N/mm^2]	MATERIAL	RESISTANCE N_i [kN]	φ	N_{Rd} [kN]
	4ø V14	A_a	R_a		N_a		
S 150		A_s= 6,2	R_s= 375	O 10 425	N_s= 232,5	0,803	1979
		A_b= 1193,8	R_b= 22	B 40	N_b= 2626,5		
	8ø V22 + 4ø V14	A_a	R_a		N_a		
S 275		A_s= 36,6	R_s= 375	O 10 425	N_s= 1372,5	0,812	2881
		A_b= 1163,4	R_b= 22	B 40	N_b= 2559,5		
	8ø V25 + 4ø V16	A_a	R_a		N_a		
S 350		A_s= 47,3	R_s= 375	O 10 425	N_s= 1773,8	0,815	3642
		A_b= 1152,7	R_b= 27,5	B 50	N_b= 3169,9		
	4ø V28 + 4ø V14	A_a= 145	R_a= 210	O 11 373	N_a= 3045		
S 500		A_s= 24,6	R_s= 375	O 10 425	N_s= 922,5	0,785	5005
		A_b= 1030,4	R_b= 27,5	B 50	N_b= 2833,6		
	4ø V32 + 4ø V14	A_a= 220	R_a= 200	O 11 373	N_a= 4400		
S 610		A_s= 32,2	R_s= 375	O 10 425	N_s= 1207,5	0,785	6141
		A_b= 947,8	R_b= 27,5	B 50	N_b= 2606,4		

Table 1. Facade columns - cross section and compression capacity

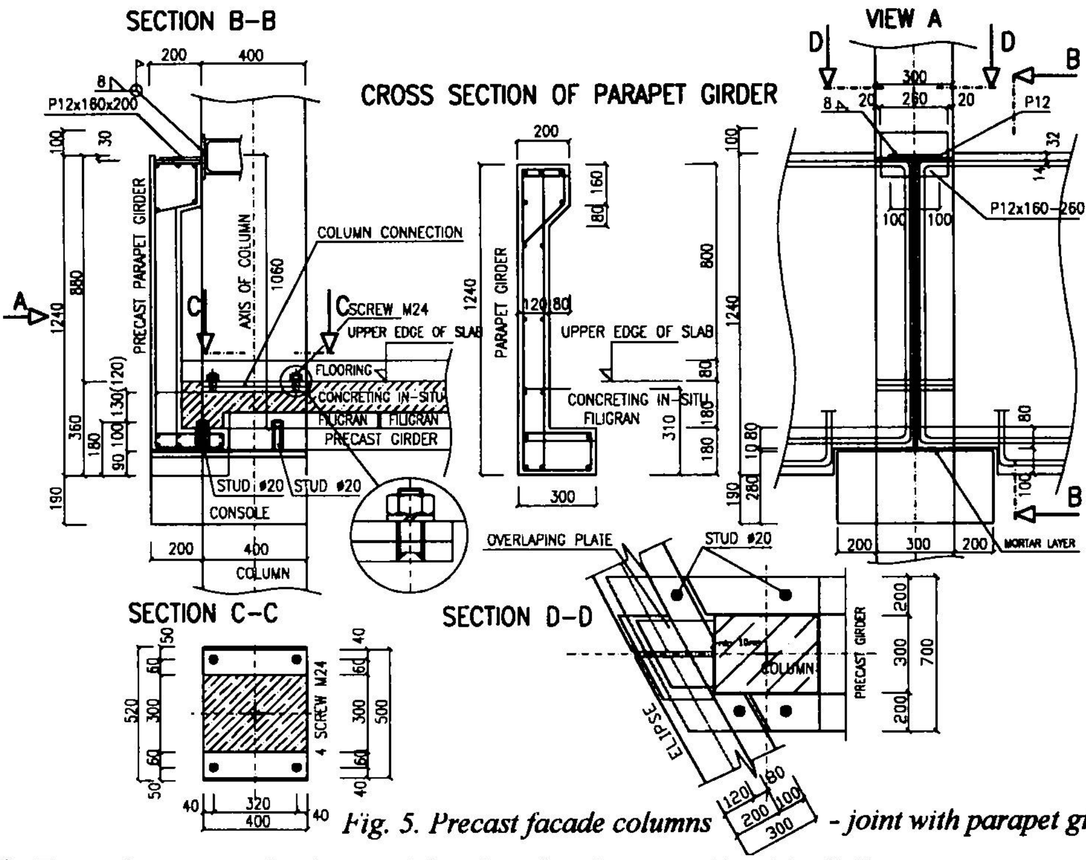


Fig. 5. Precast facade columns - joint with parapet girders

3. Experience results in combined technology realized building

After our experiences, we could summarise:

1. Combined structural system enables new architectural forms and realization technologies.
2. In comparison with complete steel solution: greater stiffness, rigidity and durability, simplified corrosion and fire protection, more structural complementation.
3. In comparaison with complete concrete structure: smaller dimensions of the bearing elements, higher mass of structures which could be prefabricated, more precise geometry, shortening the erection time (namely in comparison with in-situ concrete).
4. New technology, short tradition, not enough knowledge from edication process, structural designer obliged to know perfectly both materials.
5. Higher pretention on the coordination during the planning and realization.
6. The economical effects are in the region of indexes which are difficult to quantity - short time of realization, in the steel proper accuracy, clear static and structural system.

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Long Span Office Construction Using Composite Cellular Beams

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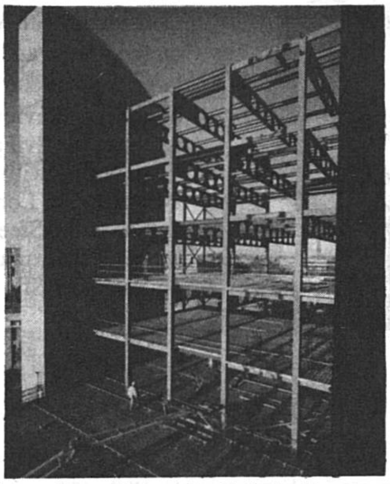
1. Summary

This paper details the development of the cellular beam as a composite floor member from its roots as a castellated section. The paper summarises the aspects of the cellular beam which make it suitable as a composite section before discussing a recent British Steel report (Ref 2) which shows clearly that long span floor construction can be achieved for the same cost as traditional short span methods, using composite cellular beams.

2. Introduction

The development of composite construction in the USA over 25 years ago brought about a revolution in the way modern commercial office buildings are now constructed. By utilising the concrete slab as the compression flange of a steel floor beam, the weight of steel in a typical multi-storey building was reduced by up to 30% when compared to the non-composite equivalent. It was envisaged at that time that long span, column free office space would become the norm, but this was evidently not the case. While composite construction became the most economical method for steel frame construction, the spans remained relatively short, necessitating internal columns.

Cellular Beams were invented and patented by Westok Structural Services Ltd of Wakefield, England in 1987 as a new, flexible form of castellated beam. Their adaptability has led to their use in many structural applications, the most significant of these being long span composite floors. (see photo 1).



Long-Span Composite Cellular Beams

3. Developments in Long Span Floor Construction.

3.1 What defines a long span floor?

A long span floor can typically be defined as an office floor where a client has requested the absence of internal columns, allowing total flexibility for the partition or furniture layout. To achieve this a floor must span from external wall to external wall, or from an internal lift core to external wall, creating spans in the range of 12m-18m.

3.2 Why did long span construction not become the norm?

Due to the work done by the concrete slab of a composite beam, the top flange of the steel beam can be small compared to the bottom tensile flange. Many forms of steel beam have made use of this fact. Stub girders, tapered asymmetric plate girders, lattice girders are commonly designed with a reduced top flange but the cost of production or the weight of steel required make these systems costly.

The cost premiums for long span construction has been evaluated as between 2 and 3% as shown by the British Steel report (Ref 2). Long spans are considered to be desirable but without commercial value. Thus, the extra finance required is seldom provided.

4 Cellular Beam Production Process.

Like their predecessor, the traditional castellated beam, cellular beams are profiled from a hot rolled beam or column section:

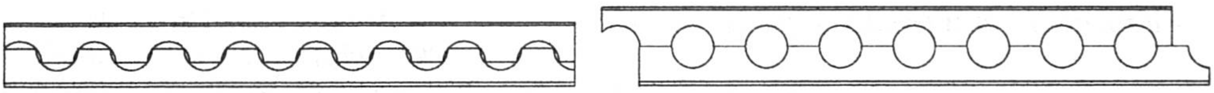


Fig 4.1 Profiling a cellular beam.

After profiling, the two halves are separated (Fig 4.1) and moved relative to each other by half a cell spacing and the beam is then re-welded along the centreline of the cells. The finished depth of the beam is between 1.3 and 1.5 times the depth of the parent section.

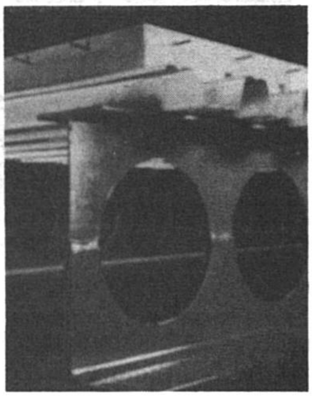
The finished depth is a function of the cell diameter and cell spacing which are varied to suit the structural and geometric requirements of the beam. With exact floor depths being a major consideration for modern commercial office developments, the cellular beam can be designed specifically to fit any given depth of floor. This is achieved using differing combinations of top and bottom section, cell diameter and spacing.

During structural analysis, cells can be varied to adjust the amount of steel around a cell. Thus, if a beam is found fail in shear, the void diameter can be reduced to leave a greater area of web above and below the cell. Likewise, if a webpost is shown to fail in buckling, the cell diameter can be reduced or the cell spacing increased. A good cellular beam design optimises both its structural and geometric requirements.

4.1 Production Advantages for Long Span Cellular Beams.

The two main advantages of the production method for composite floor construction are:

1. The opportunity to mix sections to form asymmetric beams (photo 2).



The Asymmetric Cellular Beam

2. The ability to curve the flexible half beams to produce pre-cambers (Fig. 4.2).

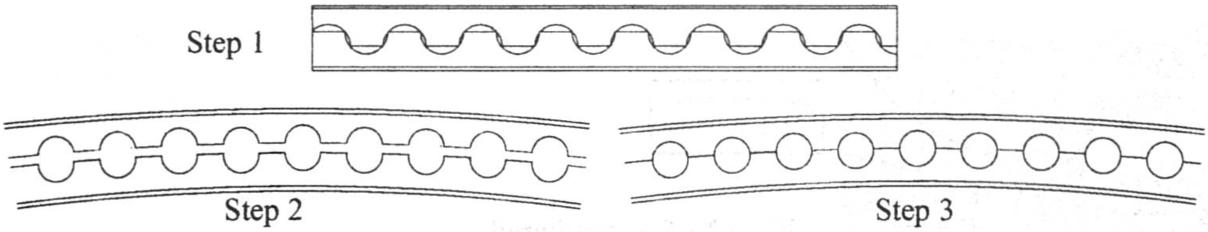


Fig 4.2 - Pre-cambering the cellular beam

As a consequence of the production method, voids are provided for the integration of building services. This allows the floor depth to equal those associated with short span floors (fig. 4.3) maintaining the cost of external cladding and finishes.

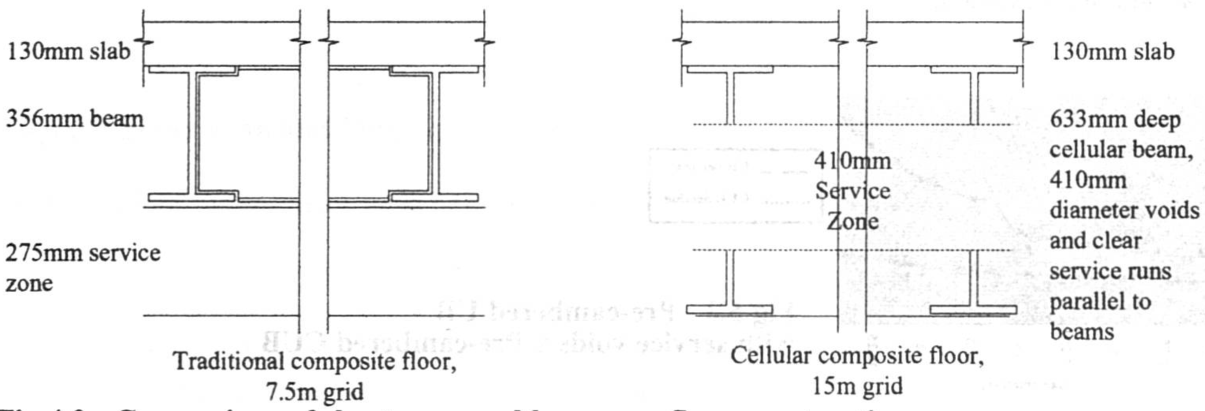


Fig 4.3 - Comparison of short span and long span floor construction.

5 Cost Analysis of Long Span Composite Cellular Beams.

To begin the cost analysis of long span versus short span, a comparison can be drawn between two long span floor options. A Unit cost/tonne can be established for plain universal beams (UB) and their equivalent cellular beam (CUB):
Unit cost of UB = total weight x cost per tonne of steel
Unit cost of CUB = total weight x[cost per tonne of (steel + cellular beam production)]

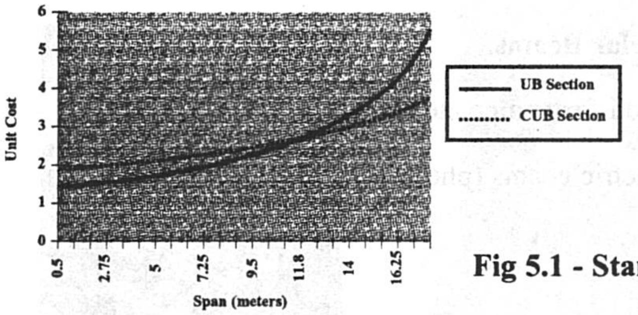


Fig 5.1 - Standard UB v CUB

The first graph shows that the CUB is only economic above spans of 12m. It does not, however, show the full picture as the cost of pre-cambering must be added.

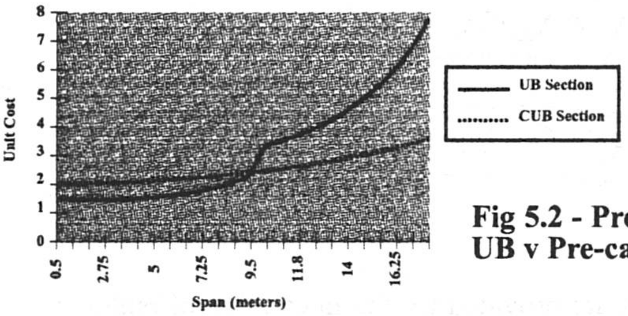


Fig 5.2 - Pre-cambered
UB v Pre-cambered CUB

The cost of pre-cambering the UB is required on spans above 10m. Thus, the difference in unit cost is enlarged on these longer spans. To complete the picture, the cost of providing service integration must be added.

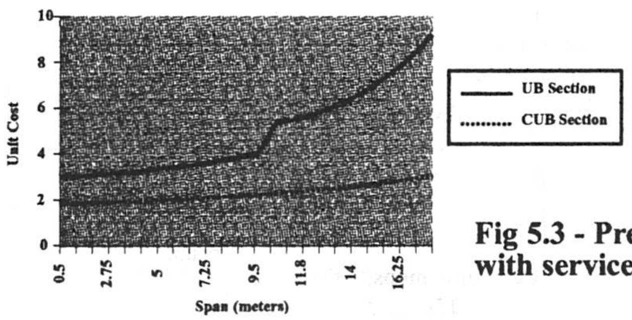


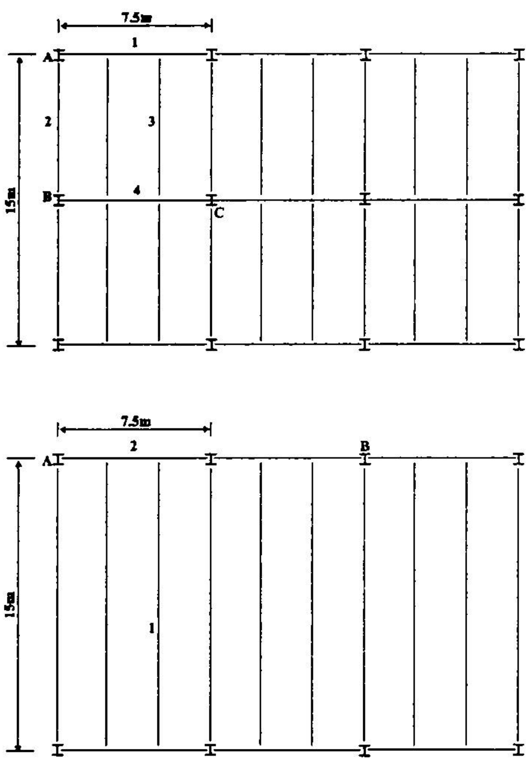
Fig 5.3 - Pre-cambered UB
with service voids v Pre-cambered CUB

The additional cost of providing service integration through a UB has been generalised in this case. Unit costs have been adopted from reference [1], where the BSCA major fabricators provided a rate of 250 GBP/tonne to create 4 service voids in the web of a UB.

Set out below is a model for assessing the true cost comparison of a long and short span option for a typical building.

6 Comparison of TOTAL Building Costs.

As expected, the long span floor has a greater overall steel weight (Fig 6.1).



Typical short span floor. Steel S355
<u>Beam</u>
1   305*165*40 UB
2   305*102*33 UB
3   305*127*37 UB
4   356*171*57 UB
<u>Column</u>
A   254*254*73 UC
B   305*305*97 UC
C   305*305*137 UC
Total weight of floor (including columns) <u>26.4 tonnes.</u>

Typical long span floor. Steel S355
<u>Beam</u>
1   633*152/191*59 CUB
2   533*210*82 UB
<u>Column</u>
A   305*305*97 UC
B   356*368*153 UC
Cell Data - 410mm dia @ 600mm c/c
Total weight of floor (including columns) <u>32.3 tonnes.</u>

Fig 6.1 - Short Span and Long Span Floor Layouts.

However, four further items must be considered before the true cost comparison is known:

1. The number of elements and connections must be assessed, as they constitute the handling time, fabrication cost and erection period.
2. The foundation arrangement and design must be carefully compared, as fewer columns leads to reduced sub-structure costs.
3. Fire rating of the floor beams, as long span beams have a lower Hp/A value and thus need less material fire protection.
4. The clear-span cellular beam frame reduces the cost of the building services.

6.1. Number of elements and connections:

Short span system = 38 elements, 58 connections.
Long span system = 24 elements, 32 connections.

Two aspects of construction are affected by the reduction in the number of elements and end connections.
1. Production time in factory.
2. Erection program and crane time.

The difference in cost for these items can be calculated in man-hours and crane utilisation time. This calculation will always be conservative as the cost difference is not only a direct capital cost for labour and crane time, but the effect of crane usage on overall program must be assessed. Keeping crane usage to a minimum is an important but often overlooked aspect of steel frame design.

6.2. Reduced foundation costs.

The SCI report [Ref. 1] puts the cost of foundations for a short span system at 21 GBP/m^2 and for a long span system at 17 GBP/m^2 for an 8 storey building (m^2 of net ground floor area). Thus, a reduction of 19% can be achieved in the foundations costs of long span buildings. The sum of the applied forces on the foundations will be equal for both buildings, however, two aspects of long span construction explain this reduction:

1. Rationalisation and reduction in the number of pile caps. On each gridline, the short span system requires 3 pile caps, for long span only 2 are required.
2. Pile groups for short span systems typically utilise three piles per column. The long span system, with higher column loads, requires four piles per column. Thus, for each gridline, the long span system has 8 to the short span 9 piles. A reduction of 11%.

6.3. Reduced fire protection costs.

Taking the secondary steelwork from figure 6.1, the Hp/A values can be calculated:

305*127*37 UB	2.11
633*153/191*59 CUB	18.9

In this case the difference is small, but still in favour of the long span element.

6.4. Reduced service costs.

The British Steel report calculates an 8% cost-saving by using circular rather than flat elongated service-ducts. Circular ducting is cheaper to produce and allows more efficient air passage (Photo 3).

7. Conclusion

The adoption of clear-span floor construction has generally been limited to need rather than choice. The recent findings that clear-span steel frames produce a cheaper total building cost will allow the clients preferred layout of clear spans, uninterrupted by columns, to become the norm.

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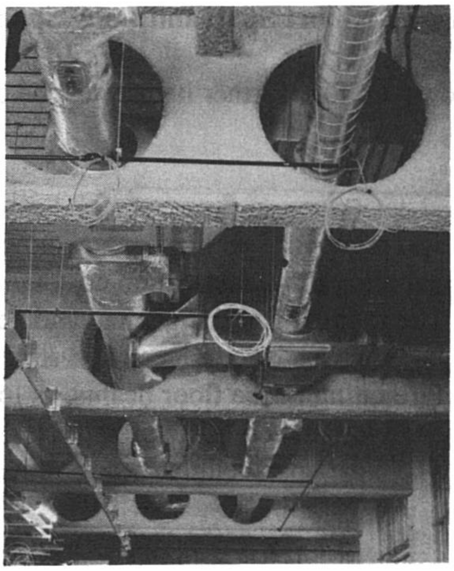

Integration of Cellular Beams and Services
(Photo 3)

Erection of Composite Bridges with Precast Deck Slabs

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Summary

This paper presents some typical erection practices of composite bridges with precast reinforced concrete deck slabs in Russia. Also it focuses on the connection details of steel to concrete, joints between reinforced concrete deck panels. Based on the obtained experience and techniques tested at construction sites, standard structural solutions have been worked out and are discussed.

1. Introduction

The first bridges utilizing the composite concept of combination the best characteristics of steel and reinforced concrete materials were constructed in Russia in the beginning of the century. The wide application of bridges of this type was begun in the 1950s. At that time the first standard designs for the spans of 21, 32.4 and 42.5m have been worked out. The tendency to shorten construction time led to more widespread use of precast deck slabs and this form of construction has become predominant in the Russian composite bridge construction practice.

The use of precast reinforced concrete deck slabs reduces additional stresses in composite structures caused by creep and shrinkage, simplifies the erection procedure, eliminates falsework and gives some advantages when an adjustment of load effects in the structure is planned.

2. Composite Highway Bridges with Precast Slabs

It is essential to consider methods of construction at an early design stage. For long-span bridges in the Russian construction practice the method of incremental launching is the most widespread. For short-span bridges, the steel girders may be erected complete using one or two cranes as necessary. The use of non-prestressed precast deck slabs is the most common practice which is now discussed.

Reinforced concrete deck slabs are normally erected when steel structure is already constructed and placed in its final position. The reinforced concrete deck panels are typically fabricated at specialized shops for reinforced concrete structures and then transported to a construction site or produced in a casting yard adjacent to the site, enabling the contractor to maintain close quality control. To minimize shrinkage and creep, the precast reinforced concrete deck panels can be stored for at least three months prior to installation in the bridge. A number of precast slab panel sizes have to be designed minimum. The choice of dimensions and weight of slab panels is generally governed by conditions of erection and transportation. Various types of cross sections of precast slabs are shown in *Fig. 1.*

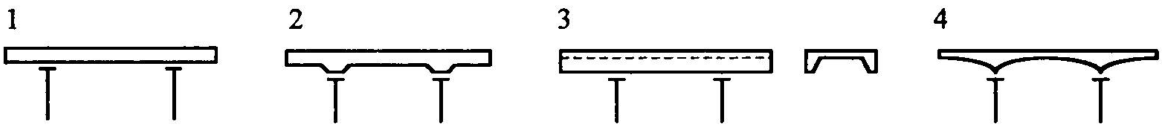

Fig. 1. Typical cross sections of precast slabs.
1 - uniform slab; 2 - haunched slab; 3 - Π - shape slab; 4 - curved soffit slab

The most usual types of precast reinforced concrete decks are slabs of uniform thickness and haunched slabs. To employ appropriate erection techniques and ensure composite action the following aspects are required to be properly considered: structural details of reinforced concrete slabs (shear connection, slab joints), hydrometeorologic effects (air temperature, wind during erection) and availability of special cranes. To achieve the required quality at the connections, an important consideration is given to a sequence and technology of works.

It was the usual practice to fabricate the precast slabs with provision of holes for shear connectors that were later filled with concrete. The disadvantages of this traditional solution for the connection of steel to concrete are the difficulties of concrete placement which, in addition, has a lack of inspection access. The concreting is distributed by small quantities over a big area of the deck. The quantity of this filling concrete may be of 10-20% of the quantity of precast deck. Filling of holes in the precast deck is required to be completed within a limited period of time. To improve the practice a new structural solution has been developed. It is based on the use of steel embeds into the precast slabs. The precast slab is connected to main girders by means of angles and high strength bolts. A typical cross section of continuous composite superstructure (84m - main span) is shown in *Fig. 2.*

Handling reasons influenced dimensions of precast slab panels which are typically taken as about 2.6 x 15 x 0.25m. The precast panels are laid with their long dimension perpendicular to a centerline of bridge carriageway and are normally erected by a crawler crane with a capacity of 25 t, working from the slab panels laid previously (*Fig.3*).

Using these techniques, the erection of precast slabs can be implemeted independent to air temperature. The only limitation is set up for the air temperature of in situ concrete placement between adjacent units. If no additional measures to assure a proper concrete hardening is planned to be implemented, the placement of concrete have to be above +5°C. Also a special concrete with additives against shrinkage may be applied for this junction between the precast slabs.

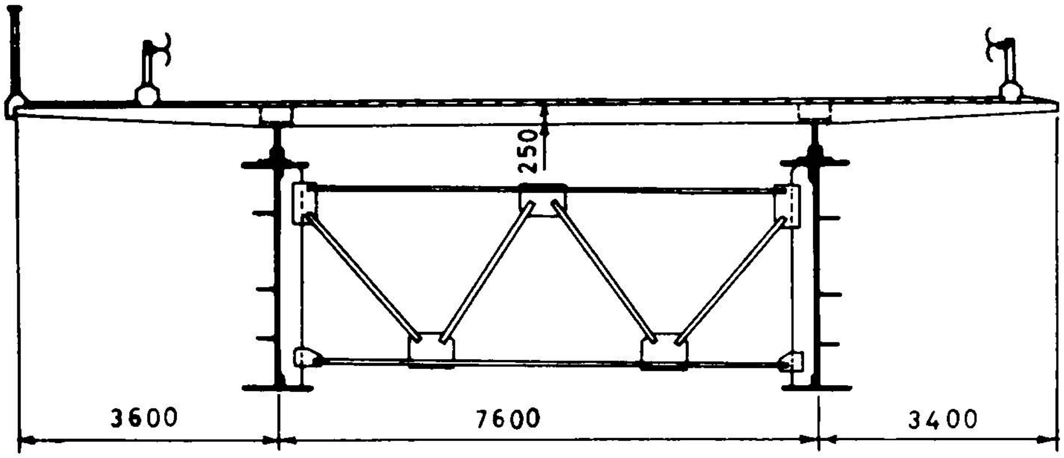

Fig.2. Typical cross section of superstructure using steel embeds for connection of steel to concrete.

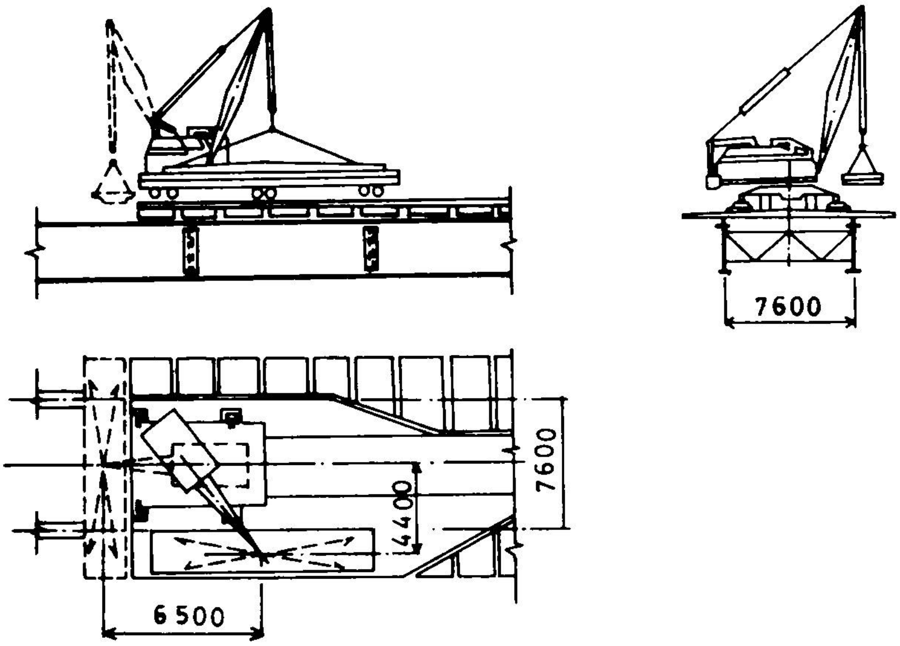

Fig. 3. Scheme of precast slab erection.

3. Composite Railway Bridges with Precast Slabs

3.1. Precast Slabs for Ballastless Track

In the Russian bridge railway construction practice steel superstructures of a through and deck truss types are widely used. Standard designs of these bridge types cover the range of spans from

33 to 154m.The application of ballastless track over precast reinforced concrete slab panel is a typical practice. This type of slab has a longer lifespan, prevents top flanges of girders against corrosion and deterioration, allows to replace wooden ties with no lifting or lowering the track line at approaches, provides safety in case of derailing. A typical section of precast reinforced concrete slab panel is shown in Fig.4.

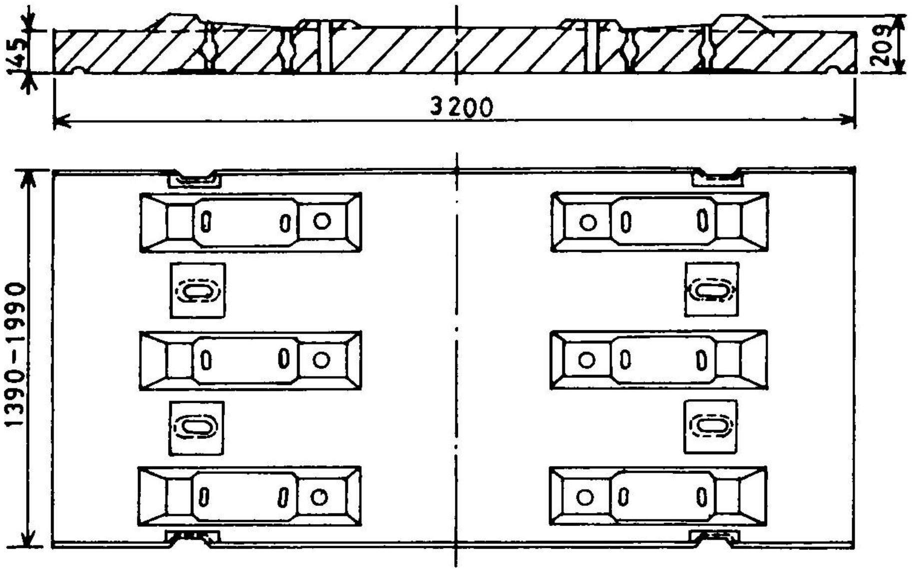

Fig.4. Typical cross section of precast reinforced concrete panel.

Erection of precast panels may be easily implemented with a cantilever locomotive crane or special cranes for track installation. For the connection of panels to steel structure stud bolts are normally employed. Gaps between the precast slab panels and bearing pads are not allowed. The connection between the panels and superstructure are filled with mortar or concrete under a thorough control. These works have to be implemented only in a warm weather period. Within a period of 5 days while concrete hardening, the force of stressing the stud bolts have to be controlled by a value of up to 80kN. When concrete strength of at least 980 kPa is obtained, the stud bolts are tightened to the design value of 200kN.

3.2. Composite Superstructures of Deck Plate Girder Type

In nowadays railway bridge construction practice composite superstructures of deck plate girder type are mainly employed. This type of single track ballasted deck superstructures was designed in 1970s for the spans of 18.2, 23.0, 33.6, 45.0 and 55m.

Steel portion of each superstructure comprises 2 welded main beams of I non-symmetrical section (Fig.5). Steel main girders for 18.2 - 33.6m spans have no erection connections. The main beam of 45m span consists of 2 segments of 22.5m length each. The main beam of 55m span consists of 3 segments of the following lengths 17.4, 21, 17.4m. To form a complete superstructure cross section, main beams are connected between each other by cross-bracings. Stability of top flange at erection is assured by cross-bracings and at operation condition by deck slab.

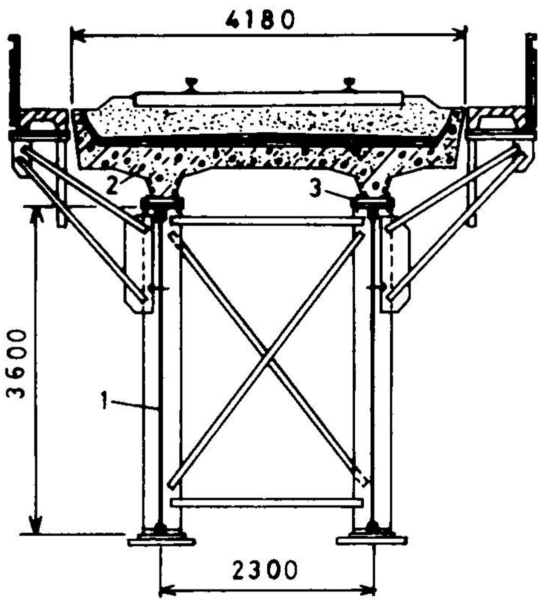

Fig. 5. Typical superstructure cross section with ballasted deck.
1 - steel girder; 2 - reinforced concrete slab; 3 - embedded item.

The slab consists of panels, length of which are taken from 2.5 to 3.2m (weight of one panel up to 11t). The position of embeds has very strict tolerances. The slab has relatively high haunches, contributing to a working capacity of the structure. To reduce cast-in-place concrete processes, high strength bolts are used for connection of precast reinforced deck panels to steel structure. Cast-in-place concrete only implemented at the connections between deck panels.

The composite superstructures of 18.2, 23.0 and 27m spans are normally erected by cantilever locomotive cranes GEK-80 and GEPK-130 with deck slabs already connected to the steel beams. The composite superstructure of 33.6m span may be erected by the cantilever locomotive crane GEPK-130 with no 4 deck panels in its final position. Assembled steel beams of 45m span may be erected by crane GEPK-130 or by crane GEK-80 but in the latter case a temporary pier in the middle of span is required. Steel structures of 55m span may be erected by the same cranes but additionally using a temporary pier in 1/3 of the span. Alternatively the erection may be implemented by the method of longitudinal launching using a nose and in this case temporary bracings at top flanges of steel beams are required. The prefabricated slab panels are supplied to erection site with the applied waterproofing and protective layer.

3.3. Composite Superstructures of Large Shop Fabricated Units

The aim of efficient construction schedules, economy and safety may be achieved by the use of large prefabricated units. To meet this aim in the construction of railway bridges a concept of composite superstructure comprising two large shop fabricated units have been worked out. Although the superstructures of various span length have been designed for a single track railway the application for a double or more track railway bridge is also possible using a common ballasted deck. The designed box girders with a ballasted deck cover standard span lengths of 18.2, 23.0, 27.0, 33.6, 45.0m. A typical cross section of superstructure comprising two shop fabricated units is shown in Fig. 6. Each composite superstructure unit comprises a sealed steel box girder

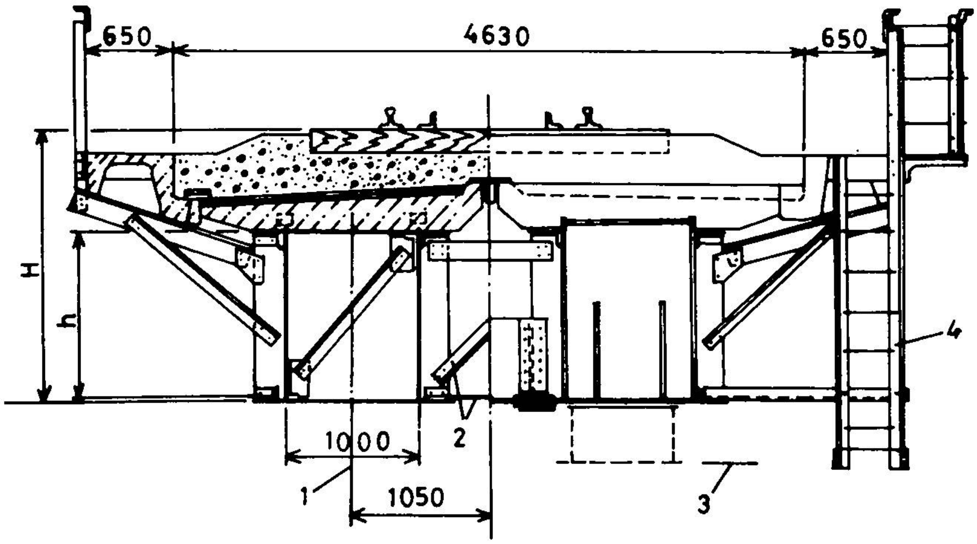

Fig.6. Typical cross section of superstructure comprising two shop fabricated units.
1 - axis of box girder; 2 - only for superstructures of 27m span and over; 3 - top of bearing upstand; 4 - ladder for pier access.

(full span) and reinforced concrete deck for ballast. Waterproofing and protective layer are applied at shop. The connection of reinforced concrete deck to steel is provided by means of stiff shear connectors.

Two composite superstructure segments are jointed at erection by cross bracings using high strength bolts. Weights of composite superstructure segments for standard span lengths are given in the Table below.

Span length, m	18.2	23	27	33.6	45
Weight of composite superstructure segments, t	34.6	45.4	54.2	74.0	110.7

Table.

The erection is normally implemented by special cantilever locomotive cranes GEK-80 or GEPK-130y with capacities of 80 and 130 t consequently.

4. Conclusion

The erection techniques and structural details described has been implemented for a large number of bridges in Russia. The wide application was started about 20 years ago, and over the years the techniques has been steadily refined. The abovediscussed structural details and erection techniques reflect the standard designs used for current construction practice.

Erection Methods for Longspan Steel Composite Bridges

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Summary

In order to reduce the time, risk and cost and in some cases, the environmental impact, longspan steel composite bridges are often erected by launching and lifting or the combination of both. The hereby obtained speed of erection is a definite advantage in comparison with prestressed concrete bridges.

1. Launching

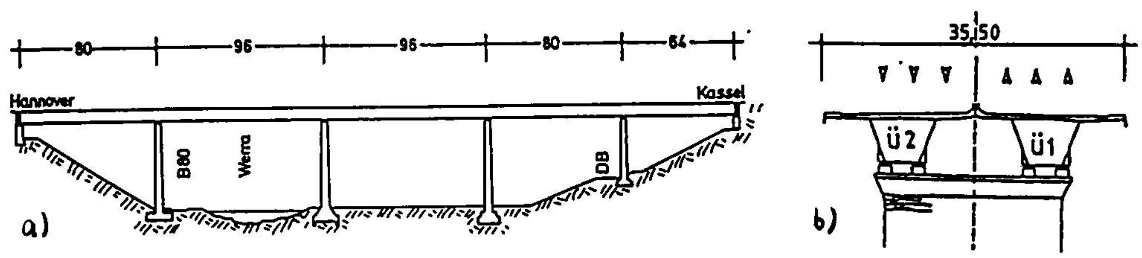
1.1 Steel Structure
Example: Highway bridge across the Werra Valley at Hedemünden Germany

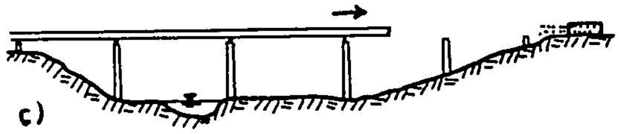
1.1.1 Design

The highway bridge across the Werra Valley has spans of 80 - 96 - 96 - 80 - 64 = 416 m. The increased traffic required to widen the bridge from 21,5 m to 37,5 m. The bridge has steel composite superstructures with trapezoidal box girders. and a construction depth of 5,85 m correspondingly to 1/16,4 of the biggest spans.

1.1.2 Construction

Due to environmental considerations, the steel structure of both superstructures was launched without intermediate piers and later the roadway slab cast using the pilgrims step method, again without auxiliary piers.





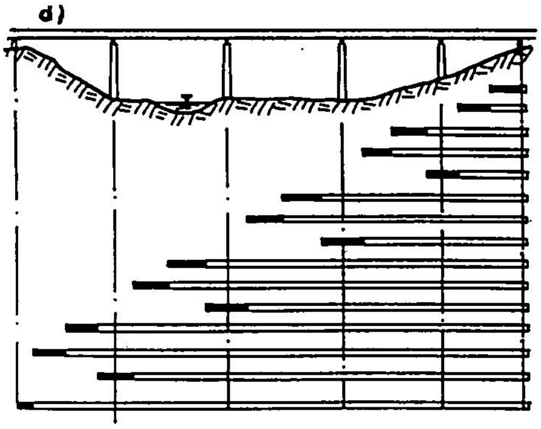
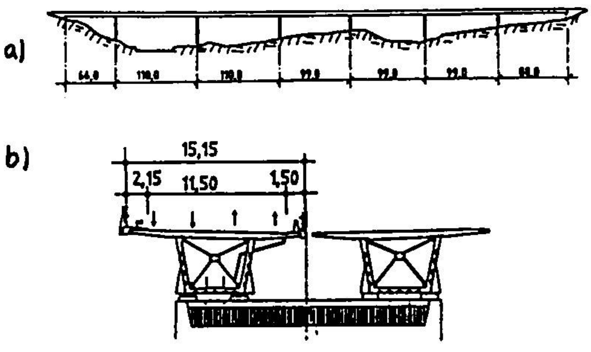


Fig. 1: Werra-Valley Bridge at Hedemünden, Germany:
a) Layout
b) Cross-section
c) Launching at the steel structure
d) Casting of the concrete slab

1.2 Complete Cross-Section
Example: Highway bridge at Wilkau-Haßlau, Germany

1.2.1 Design
The existing bridge, built in 1938/39 as part of the highway Chemnitz-Hof, crosses the valley of the „Zwickauer Mulde" with spans of 69,9 - 110,0 - 110,0 - 99,0 - 99,0 - 99,0 - 88,0 = 674,9 m. The bridge deck was a plate girder with 4 webs and had a width of 24,5 m, whilst the new bridge has a width of 30,30 m. Both roadways have steel composite superstructures with trapezoidal box girders. The construction depth is 5,08 m corresponding to 1/21,6 of the biggest span, and the roadway is longitudinally reinforced and transversely prestressed.

1.2.2 Construction
Due to the extremely reduced construction time and due to the auxiliary piers required for the dismantling of the existing bridge, the complete cross-section was launched, following a proposal of the contractor.



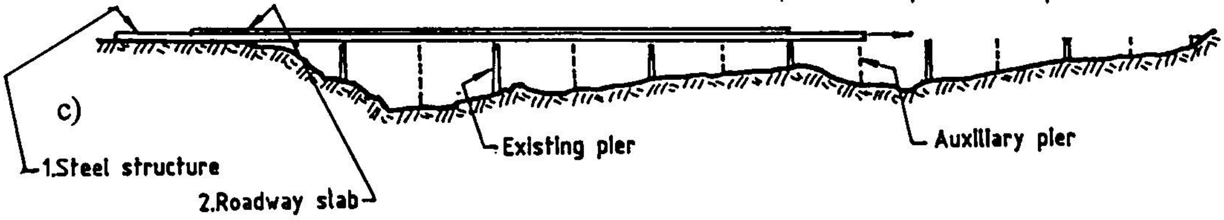


Fig. 2: Highway bridge at Wilkau-Haßlau, Germany:
a) Layout, b) Cross-section, c) Construction of superstructure

1.3 Steel Structure with Variable Depth
Example: Highway-Railway Bridge across Caroni River at Ciudad Guayana, Venezuela

1.3.1 Design

The main structure is a continuous beam with spans of 45 - 82,5 - 213,75 - 82,5 - 45 = 478,75 m, Fig. 3a. The construction depth of 5 m at the centre line and 14 m at the piers corresponds to slenderness ratios of 1:43 and 1:15 respectively.

The cross-section is a two-cell box girder in the main span and the long side spans, and an I-beam with 3 webs in the short side spans.

The bottom chord is of steel in the area of positive moments of the main span and in the short side spans, and of concrete in the area of negative moments up to the side span piers, Fig. 3b.

1.3.2 Construction

The steel erector proposed to assemble the steel structure behind the abutment and to launch it. During launching of the steel structure, the rear of the bridge deck was supported by an auxiliary pier fixed to the bridge and sliding on a runaway behind the abutment, whilst the main span slid on the intermediate and main piers respectively. The variable depth of the haunch was compensated for by an auxiliary truss girder.

a)

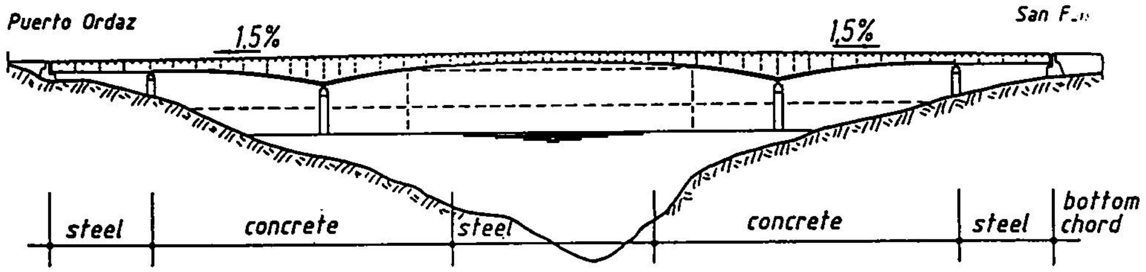
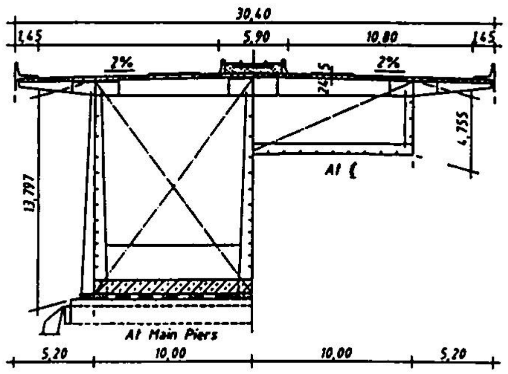


Fig.3: Highway-Railway Bridge across Caroni River at Ciudad Guayana, Venezuela
a) Layout, b) Cross-section

b)



Before casting the concrete of the bottom slab in lengths of about 14 m, the tip of the steel cantilever had to be lifted in order to reduce steel stresses virtually to zero. This was on the San Felix side achieved by an auxiliary stay cable system, and on the Puerto Ordaz side by a coupling device at the centre.

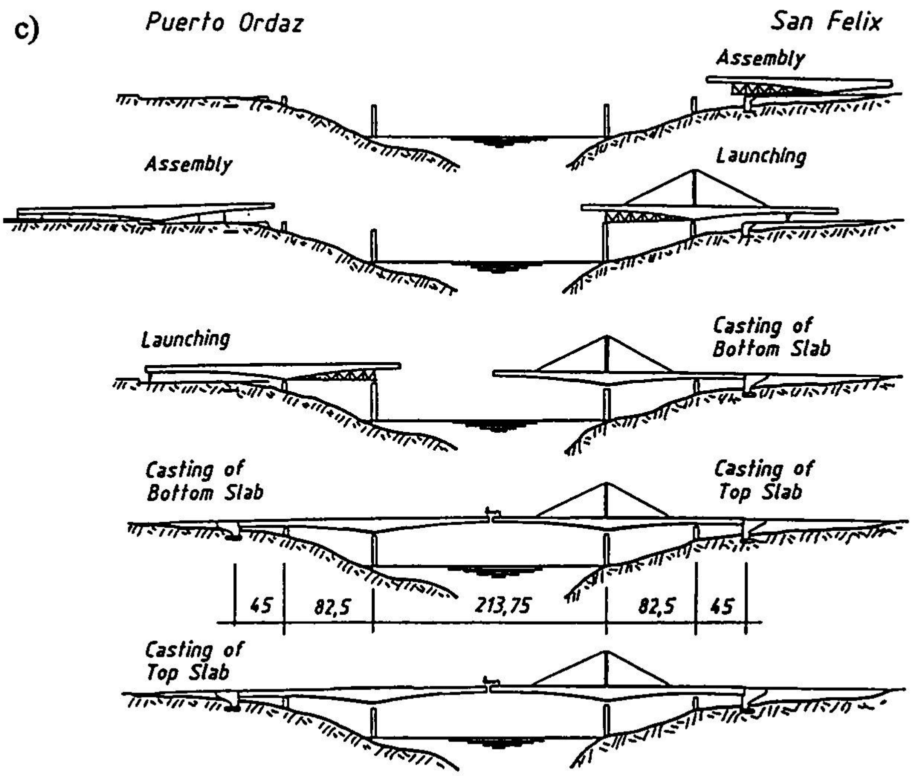


Fig. 3 Highway-Railway Bridge across Caroni River at Ciudad Guayana, Venezuela: c) Construction sequence

2. Lifting

Example: Railway Bridge across Main River at Nantenbach, Germany

2.1 Design

The double track railway bridge across the Main River at Nantenbach links the new high-speed railway line Hannover-Würzburg to the existing trunk line Würzburg-Aschaffenburg. Due to local conditions, the bridge has a slope of 1,25 % and a radius in plan of 2650 m.

For the main bridge, a continuous truss girder with spans of 83,2 - 208 - 83,2 = 374,4 m was found to be the best solution from economical, ecological and aesthetical points of view, Fig. 4. The construction depth varies between 7,66 m at the centre of the main span and at the abutments, and 15,66 m at the main piers, corresponding to slenderness ratios of 1:27 and 1:13, respectively.

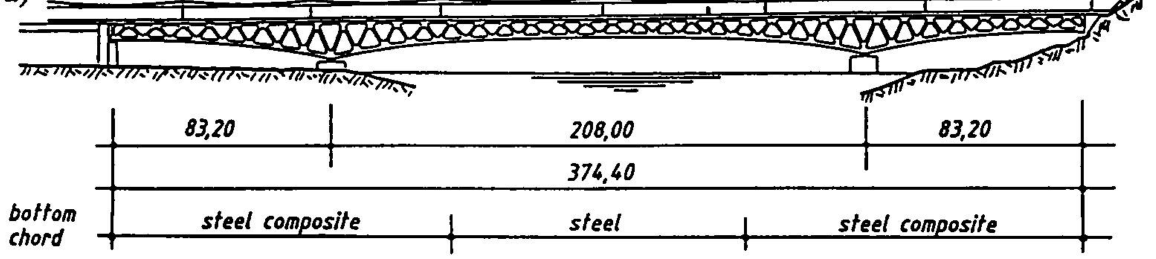
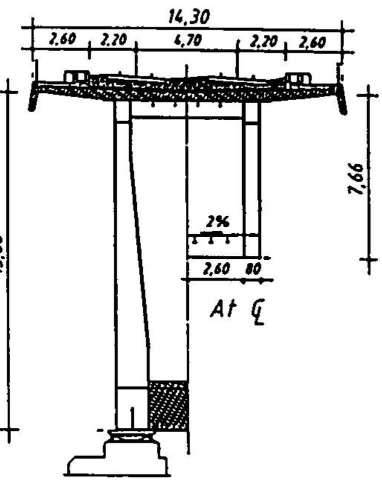


Fig. 4: Railway Bridge across Main River at Nantenbach, Germany: a) Layout

2.2 Construction

The side spans were erected on auxiliary piers, and the first 44 m of the main span steel truss were erected by free cantilevering from the main pier towards the river. After concreting the bottom chord, the central part of the main span, with a length of 140 m and a weight of about 1600 tons, was floated in and lifted, Fig. 4c. After closure of the joints, the tops slab was cast from the centre of the main span towards the abutments.

b)



c)

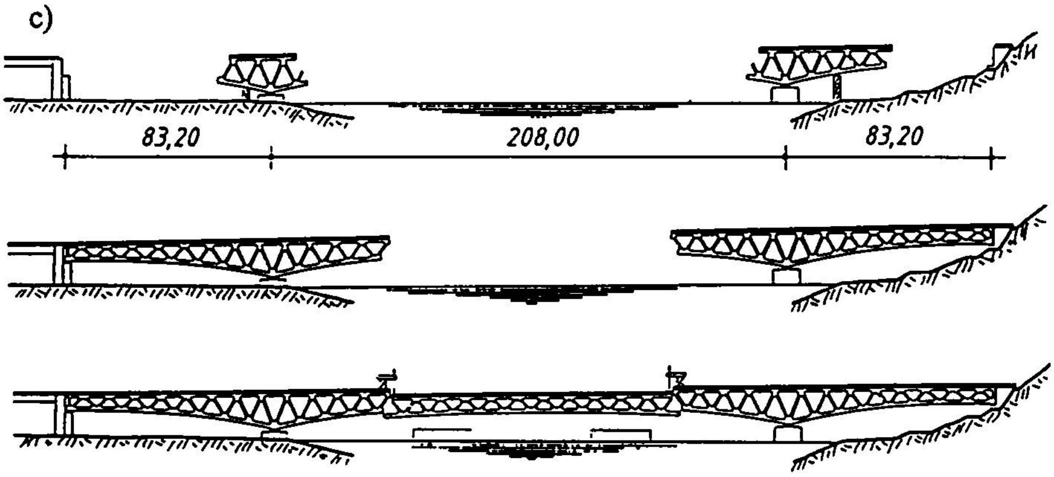


Fig. 4: Railway Bridge across Main River at Nantenbach, Germany
b) Cross-section, c) Construction sequence

3. Launching and Lifting
Example: Kap Shui Mun Bridge, Hong Kong

3.1 Design

The Kap Shui Mun Bridge is a double deck cable-stayed bridge with spans of 80 - 80 - 430 - 80 - 80 = 750 m, Fig. 5a. The prestressed concrete part of the superstructure is a 3 cell box girder, Fig. 5b. The steel composite part of the superstructure consists basically of the steel main girders and cross frames and the concrete top and bottom slabs, Fig. 5c.

3.2 Construction

The side spans were constructed by incremental launching, with a length of the segments of about 18,30 m, Fig 5d. The launching nose is formed by the first 3, specially designed elements of the steel composite main span. The steel composite bridge deck of the main span is erected by free cantilevering simultaneously from both towers, Fig. 5e. The individual elements are erected together with their slabs, yielding an erection weight of 500 tons.

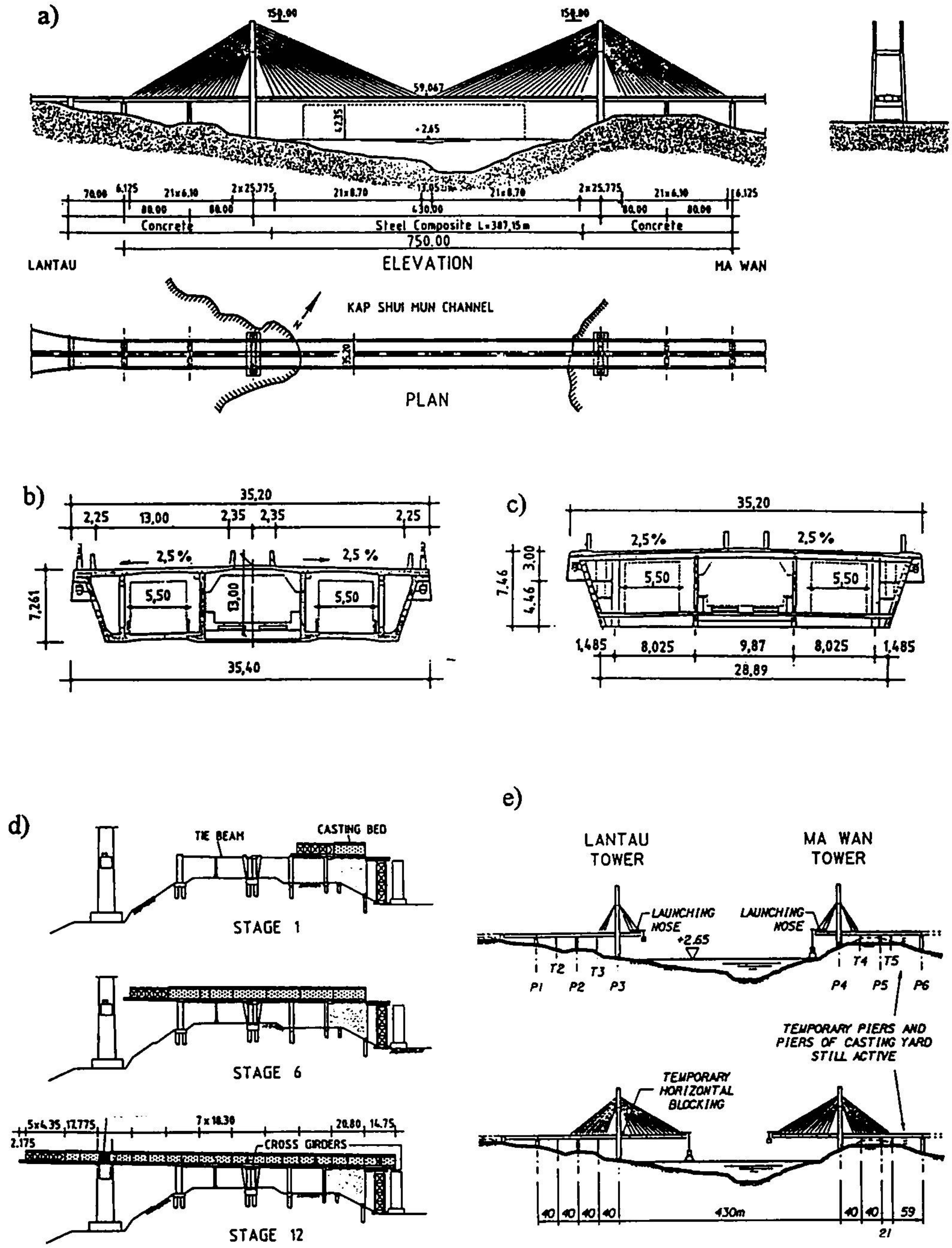


Fig. 5: Kap Shui Mun Bridge, Hong Kong:
a) Layout, b) Cross-section of side span, c) Cross-section of main span, d) Launching of Ma Wan side span, e) Erection of main span