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STATE OF THE ART REPORT COMPOSITE BRIDGES

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CONTENTS

PREFACE

In 1972 a working group (Staalbouwkundig Genootschap, TC-19) started an investigation regarding the use, design and calculation of composite bridges in The Netherlands. The reason was quite simple: this new type of bridge (in fact: new application of materials) was still in its infancy and was supposed to become of major importance.

Unfortunately the intentions were considerably better than the results. A paper, published in Bouwen met Staal, nr. 28, june 1974, was the end of an excellent initiative. Since then composite bridges were no longer in the picture in The Netherlands.
In the surrounding countries however surprising developments took place.
In England the drafting of a code on composite bridges started. In other countries the main ideas of this code were accepted and comprehensive investigation programmes launched.

As a result:
- In France about 80%, in England about 60% and in the USA about 50% of the highway bridges build in the last five years (that is: since 1986) are composite bridges.
- Switzerland is famous for its daring composite structures in mountainous areas.
- In all countries, including Germany, composite bridges are considered as a very competitive and simple type of bridge.

It seemed as if all these developments passed us without being noticed. Still, for consultants it is necessary to dispose of sufficient know how to design such bridges.

Considering the economic situation in Europe after 1992, we may expect tendering on an international basis. Contractors, unable to follow modern developments, might loose a market.

No doubt many more reasons can be found. However, the reasons mentioned up till now justify the start of a working group with the task to produce a "**State of the Art Report**.

The working group started: January 8, 1991.

Members:

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The drafting of the report was carried out by two groups:
- calculation, design, research (members indicated by *)
- fabrication and erection (members indicated by ■)

1. THEORY AND RESEARCH

1.1 STEEL

Composite bridge research has tended to be dominated by steel related research. Reasons for this dominance are varied but certainly include the following:
- Steel has a larger role to play in a composite bridge than the concrete slab. In effect the steel structure is often used as a working platform, and has an important contribution to both the composite shear and moment resistances.
- Composite bridge construction is economically more interesting for the steel industry than for the concrete industry.

1.1.1 FATIGUE

The fatigue behaviour of composite bridges is similar to that of steel bridges. The two most important factors determining bridge fatigue life remain the same: *geometry* and *stress range*. The influence of traffic and dynamic response on fatigue life is contained in Chapter 5.

Geometry
Only one detail type is unique to composite bridges: shear connectors, in particular welded studs. For a review of headed stud fatigue studies prior to 1974 the reader is referred to the following reference [Asce, 1974]. Experimental and theoretical studies have been undertaken to investigate the effects of shearing force upon the fatigue strength of headed studs subjected to repeated negative moment [Maeda, et.al., 1986]. Three failure modes have been identified for this detail [Haensel, 1992]. Two failure modes are confined to the stud itself (treated in paragraph 4.1), the third is related to crack propagation from the stud base into the supporting flange. The third failure mode is stated to be the most critical and a proposed three part design fatigue resistance curve is proposed.

Stress range
The fatigue resistance of modern welded composite bridges, especially those using high strength steels, must be carefully checked. Composite bridge details should be chosen according to their exposure to traffic loading [Smith, et.al. 1989]. Stiff elements should be bolted, not welded.

Several factors have been identified which effect the fatigue life of composite bridges by changing the stress range (the number of contributing cycles). The relationship between span length and continuous as opposed to single spans on fatigue life was studied [Jacquemoud, 1982]. Prestressing increases the concrete cracking load, bridge stiffness, natural frequency thus decreases stress range [Kennedy, 1987] [Saadatmanesh, et.al. Nov. 1989]. Ballasting of railway bridges can insulate the structure reducing the number of damaging cycles [Smith, et.al. 1989].

1.1.2 BUCKLING AND TENSION FIELD ACTION

Most studies since 1970 have dealt with two related subjects: Design taking into account inelastic reserve and the elimination of as many web and flange stiffeners as possible. Research in both fields has gained interest due to increasing costs of labour in relation to materials. It is also of particular interest to note that the majority of the available literature is concerned with multiple (two or more) plate girders with flat concrete deck slabs.

The interaction between tension-field action in web panels and the shear connection to slabs in girders with relatively light steel top flanges was examined [Allison, et.al., 1982]. An existing method of analysis for steel girders, was extended for use with composite girders [Evans, et.al., 1978]. A thin web design method, especially adapted for composite bridges, in accordance with the Swiss steel code, was published [Dubas, 1983]. It is noted that before 1979 post-critical web behaviour was ignored and longitudinal trapezoidal stiffeners were used. A design method for predicting the ultimate load of composite end panels is available [Porter, et.al. 1987]. Further research topics are suggested. Continuing developments in post-critical web design in Switzerland have been reported [Dubas, 1991]. Criteria are given for designing curved bridges, including the effects of torsion, with either open or closed cross-sections. A recent experimental and theoretical study on the post-buckling behaviour of slender composite plate girders under combined negative bending and vertical shear has been conducted [Bode, et.al., 1992].

The results of an extensive experimental and theoretical program examining the negative moment regions of multi-span composite beams was published [Martin, 1991]. This report makes use of tests performed by others [Johnson, et.al. 1990; Ho, et.al. 1990]. Other studies on the influence of local buckling and moment distribution are also available [Johnson, et.al. 1991]. The result of this research indicates that the Eurocode 4 (part I: buildings) is both safe and economical in negative moment regions for cross-sections similar to those tested. These findings may be of use for composite bridges.

1.1.3 SHEAR LAG

A comprehensive study on shear lag was published in 1987 [Dowling, et.al., 1987]. This literature review contains articles of interest for composite bridges with full and partial interaction [Moffatt, et.al. 1976; Adekola, 1974; Moffatt, et.al., 1978; Burgan, 1987]. A simplified shear lag analysis of single span composite bridges with single-cell and spine-beam box girders with full interaction was proposed [Kristek, et.al. 1990]. An analytical study was undertaken to determine the effective width of the concrete decking in a composite cable stayed bridge [Xiang, et.al. 1990].

1.1.4 CORRUGATED WEBS

The development of corrugated webs in composite bridges is a most promising new development. Corrugated webs provide adequate vertical support to girder flanges, have good vertical shear resistance, yet a low in-plane axial stiffness. Corrugated webs thus absorbs much smaller amounts of longitudinal prestressing than would normally be the case for flat webs. Corrugated webs also are thinner than their flat web counterparts, thus weight savings may be important for long spans. A list and description of composite bridges with corrugated webs has been made [Cheyrezy, et.al. 1990]. Previous work on this subject is also available [Contribution of the French group, 1984] [Causse, et.al. 1987; Cheyrezy, 1987; Combault, 1988].

1.1.5 DIAPHRAGMS AND TRANSVERSE LOAD DISTRIBUTION

Diaphragms are the subject of increasing research interest due to increasing traffic loading, the conservatism of past designs and the need to repair and renovate existing structures. It is realised that a composite bridge with properly designed and spaced diaphragms can safely support loads well in excess of similar bridges without diaphragms, due to orthotropic (two-way) behaviour. Such behaviour enables loads to be distributed between longitudinal girders. The probability of catastrophic failure due to the overload of individual members is reduced, thus increasing structural redundancy. The influence of the number of diaphragms, their aspect ratio, skew and concrete cracking on transverse load distribution has been examined [Kennedy, et.al. 1983; Kennedy, et.al. 1987].

Proposals for modifying American LFD design criteria based upon past studies of the influence of load distribution and effective slab widths have been put forth [Heins, 1980]. Three references concerned with load distribution in composite bridges are sited [Fan, et.al., 1975; Fu, et.al, 1974; Heins, et.al., 1975]. Experimental and theoretical calculations of wheel load distributions on composite bridges have been undertaken [Elgaaly, 1987]. Surveys of wheel load distributions in different countries are given [Jones, 1976; McDougle, et.al., 1976; Culham, et.al., 1977].

1.1.6 EXTERNAL PRESTRESSING

Guide-lines for prestressing the steel sub-structure of composite bridges are proposed [Saadatmanesh, et.al. Nov. 1989; Saadatmanesh, et.al. Sept. 1989; Saadatmanesh, et.al. Sept. 1989]. German experiences with externally prestressed composite bridges are presented [Frey, 1987; Eibl, 1990]. Good literature reviews are given, advantages and disadvantages of external prestressing are explored. Ideas for further developments are listed. Previous examples of external prestressing are listed [Dubas, 1991].

1.1.7 OTHER SUBJECTS

A finite element analysis of composite box girders was published [Moffatt, et.al., 1976]. This study includes a review of past studies. The design decisions, details and relative material costs for a steel girder composite cable-stayed bridge was examined [Svensson, et.al., 1986]. The use of H-sections with embossments rolled into the top flange is described [Sato, 1989]. Composite bridges using the system have been built in Japan. A new composite bridge cross-sectional type, of reduced height, was developed for composite railway bridges [Detandt, 1990]. A comparison (as a function of span length) of the heights from the bottom chord to the road surface for four different composite cross-sectional types is given.

1.2 CONCRETE SLAB

Significant new technologies have been developed for the placement of the concrete slab on the underlying steel sub-structure [Badoux, 1985; Lebet, 1990]. Erection procedures, however, are not included in this chapter.

1.2.1 CONCRETE CRACKING AT SERVICE LOAD LEVELS

Concrete cracking is not normally associated with single spans, but can occur under service loads if significant transversal moments are present in slender or wide deck slabs. In such cases the deck may be heavily reinforced or transversely prestressed. Transverse prestressing is investigated in the design example. It required a large amount of strings and was rejected because of the costs. A major challenge for designers is finding economical methods of limiting concrete crack widths in negative moment regions of continuous span composite bridges under service load conditions. Concrete cracking can have a significant detrimental effect on composite action between slab and the steel sub-structure in continuous spans. Limiting such cracking can have the following beneficial effects:

- The stiffness of the structure is significantly increased, the natural frequency is increased and deflections are reduced. Thus rider comfort is significantly improved.
- Structural durability is increased by reducing water penetration.

Localized cracking in negative moment regions was found experimentally not to significantly increase mid-span deflections [Lebet, 1981]. Such cracking is difficult to define for design calculations. A study on the effects of prestressing in continuous span composite bridges was published [Kennedy, et.al. 1982; Kennedy, et.al. 1987]. It is concluded that cracking can be eliminated at service load levels by providing an adequate number of connectors and a compressive force in the concrete slab in negative moment regions. Two models were developed for estimating the effects of prestressing and it is concluded that concrete cracking in negative moment regions does not significantly influence transverse load distribution in the slab [Kennedy, 1983]. The problem of concrete cracking in negative moment regions was treated using alternative shear connection devices and a heavily reinforced concrete slab [Leonhardt, et.al., 1987]. The conclusion is made that reinforcement is cheaper (and less complicated) than prestressing, but that large shear forces must be transferred to the steel. The new type of shear connector was especially developed to transfer such shear forces. Concrete cracking in negative moment regions was reported to cause a 10% reduction in moments near internal supports [Lebet, 1990]. The elimination of longitudinal prestressing to reduce concrete cracking in a composite railroad bridge constructed in Germany is seen as a major cost reduction factor [Haensel, 1992]. This was achieved by careful concrete sequencing along the span.

1.2.2 NEW DECKING TYPES

Three new composite decking systems were found in the literature. Pre-cast deck units bolted to the steel sub-structure have been examined [Inoue, et.al., 1990]. Bolting is accomplished using a plate welded to top flanges. A layer of reinforced concrete bonded to an unfilled exothermic steel grid (which initially acts as formwork) has been studied [Bettigole, 1990]. After concreting the composite grid is attached to the underlying steel sub-structure by headed studs in blocked out openings especially provided for this purpose. Precast thin-walled decking units, which are then provided with a concrete overlay on site have also been examined [Detante, 1990].

1.3 SHEAR CONNECTORS

Since 1970 welded, headed shear connectors have been used in an overwhelming majority of composite bridges and countries. One exception to this general rule has been France, where alternative shear transfer devices are frequently used. These include welded angles with reinforcement passing through holes in the vertical leg and steel hoops.

The development of welded, headed shear connectors (in the 1950'^{S} and 1960'^{S}) pre-dates the scope of this report. This means that the vast majority of push type tests, used to determine maximum strength, ductility and the influence of concrete type, were performed prior to 1970. Very good reviews of previous testing programs and new studies using old test are, however, present in the more recent literature [Asce, 1974; Oehlers, et.al., 1987; Hiragi, et.al., 1990].

In the last few years new type of shear transfer devices have been investigated. This includes cold fired angles attached by shot-fired pins, plates welded to the top flange of girders which are provide with cut-outs and deformations rolled into girder flanges. These Hilti-connectors are not yet used in bridges.

1.3.1 FATIGUE STRENGTH

Prestressing may increase welded stud fatigue life in negative moment regions due to a decrease in the stress range [Kennedy, 1987]. In cases where heavy reinforcement instead of prestressing is used in negative moment regions, the fatigue strength of headed, welded studs may be insufficient due to concrete crushing near the base of the connector [Leonhardt, et.al. 1987]. A new shear transfer device consisting of a plate welded to the top flange and provided with cut-outs with an improved fatigue resistance was developed.

The fatigue strength of welded, headed shear connectors subjected to simulated wheel loadings has been studied [Matsui, et.al., 1990]. Failure was due to shearing of the studs at their base. It is stated that the fatigue strength of studs subjected to the wheel loadings is much lower than that obtained from push type tests. A parametric analysis of existing fatigue tests on welded, headed studs was recently performed [Hiragi et.al., 1990]. A multi-variable regression analysis was undertaken. Findings are that stud diameter, stud height and concrete strength influence fatigue strength. A large number of previous push type tests have been used to examine the fatigue resistance of stud connectors [Oehlers, 1990]. Parameters having a significant effect upon fatigue resistance were reported to be: the range of the cyclic shear load, the maximum applied shear load and the static strength of the connection. A fatigue testing procedure for estimating the residual strength of stud shear connectors is also proposed.

1.3.2 STRENGTH, STIFFNESS AND DUCTILITY

The shear stiffness of stud shear connectors has been estimated using the results of a large number of existing push tests [Oehlers, et.al., 1986; Oehlers, et.al., 1987]. Initial shear stiffness, slip during a cycle of load and permanent set caused by a cycle of load are estimated. A simple formula to determine permanent deformations caused by load cycles is suggested. The strength and ductility of steel strips, with circular interior cut-outs, welded to the top flange, was found to be governed by deformations of the steel strip between the cut-outs [Andrä, 1990]. The strength and ductility of three different shear transfer devices have been compared

[Trinh, 1990]. Of particular interest, the influence of normal force (both tension and compression) was examined. The grouping of welded studs on 1 meter centres does not significantly effect composite bridge behaviour [Lebet, 1990].

1.4 LOADS AND LOAD DISTRIBUTION

Loadings on highway and railroad bridges, while not strictly within the scope of this state-of-the-art report, are intertwined with the subjects of fatigue, transverse support and intermediate diaphragms, which are included in this report. As such, an effort has been made to investigate reports of measurements and or tests on existing composite bridges.

1.4.1 GENERAL

A study of real highway traffic loads on a composite highway bridge was reported [Hirt, et.al., 1977]. The purpose of this study was to collect data for future fatigue evaluations. The proof loadings of five composite bridges to verify design assumptions is described [Perret, et.al., 1978; Lebet, et.al. 1979; Crisinel, et.al., 1981]. The behaviour of composite bridges subject to overloads are examined [Cheung, et.al, 1990; Trouillet, et.al., 1990]. The dynamic-fatigue response of continuous composite bridges has been studied [Grace, et.al, 1984; Grace, et.al. 1986]. The influence of fatigue loading at resonance frequency on the structural response were investigated. The effects of concrete cracking on the natural frequency were observed. Full-scale vibrational tests on a composite cable stayed bridge have been performed [Stiemer, et.al. 1988]. A study of traffic loads on bridges, applicable to composite bridges except for the fatigue limit state, will be shortly published [Vrouwenvelder, 1993].

1.4.2 TEMPERATURE

Changes in ambient temperature cause important strains. These are normally due to daily temperature changes between night and day. The magnitude of such changes depends upon the bridge location, cross-section and choice of materials. For continuous composite bridges concrete cracking must be minimised in negative moment regions when subjected to service loads *and* temperature changes. Temperature changes must be taken into account during construction depending upon when and how the concrete deck is to be connected to the steel sub-structure.

A design method for calculating stresses resulting from temperature changes in single and multi-span composite bridges was published [Soliman, et.al., 1986; Kennedy, et.al. 1987]. The effects of extreme temperature differences on composite bridges has also be examined [Lebet, et.al. 1987]. A summary of past studies resulting in modifications to the Swiss steel code (SIA160) are reviewed.

1.5 SERVICEABILITY

Researchers and design codes are turning their attention to serviceability limit states other than ultimate limit state. This is because of two factors. First, the ultimate limit state has previously received almost undivided attention (as a result few publications are available on the subject of serviceability). Secondly, bridge owners are beginning to accept that initial construction costs

represent only a small part of the "life-cycle-cost" of a structure. Life-cycle-cost can be defined as all costs required by a structure during its life time. This includes construction, maintenance, repair, upgrading and removal.
A review of limit states for composite bridges has been published [Johnson, 1987]. A review of unanswered questions concerning the performance of bridges is available [Head, 1991]. It is suggested that probabilistic approaches can be used to treat each bridge as a component system in the infrastructure system for both design and assessment.

1.5.1 VIBRATION / DYNAMICS

Vibration or dynamic problems may occur due to unsatisfactory structural response to wind, earthquake or live loads. The problem of free vibrations in beam-slab type highway bridges (similar to the majority of composite bridges) has been studied [Ng, et.al., 1972]. The free vibrational behaviour of continuous beam-slab composite bridges, with and without prestressing in negative moment regions, has been studied [Grace, et.al., 1984; Grace, et.al., 1986]. It was found that prestressing has an important effect upon natural frequency. A conference on vibrations in composite bridges was recently held in November 1991 in Hamburg, Germany.

1.5.2 DEFLECTIONS

In a composite bridges shear deformations, as well as flexural deformations, may be important [Lebet, 1981]. The effects of the effective width of the concrete slab, modulus of elasticity of concrete, concrete cracking and movements at the concrete-steel interface were also discussed. Long term movements due to prestressing relaxing and movement at the steel-concrete interface may occur. Cracking in negative moment regions also has an effect on the long term deformations.

1.5.3 WATER CONTROL AND CORROSION PROTECTION

The penetration of water and salts into the concrete slab and the exposure of the steel sub-structure to salt water are major causes of structural degradation. Water control represents efforts by the designer to drain water away from the structure in such a manner that corrosion is *prevented.* This can be done by a judicious choice of the following: good detailing, limiting concrete crack widths, reducing (or eliminating) expansion joints and by not using precast concrete units (thus eliminating the joints between each unit). Different methods used to control concrete cracking in negative moment regions are discussed in paragraph 3.1. It has been reported, however, that composite bridges can be constructed using a strict minimum of expansion joints [Godfrey, 1988]. Distances between expansion joints may exceed 200 meters.

Good reviews of recent developments in corrosion protection for new and existing steel bridges are available [Andre, D., 1992; Ramsay, W, et.al., 1992]. A review of the past uses of weathering steels in composite highway bridges has been published [Godfrey, 1988]. Limitation to the use of such steels due to chloride (salt) and sulfate (air pollution) corrosion are given. Cost considerations are explored.

1.6 MAINTENANCE, REPAIR, REHABILITATION

Information is becoming available about the relative costs of bridge maintenance, repair and rehabilitation. This is due to the fact that the majority of highways in Europe and North American have been in service for at least 20 years. Several methods of repairing or rehabilitating an existing steel or composite bridge have been studied. These include external prestressing, adding or modifying diaphragms and adding composite action to non-composite bridges. Heat straightening of traffic damaged steel elements is a well established procedure that can be used in composite bridges.

Experiences to date with composite bridges are encouraging. Rehabilitation and widening projects have been completed on composite bridges, in which the concrete deck has been modified or removed. Even when the deck was badly weathered the steel sub-structure was found to be in good condition [Dubas, P., 1991; Tschemmernegg, 1992]. The relative cost of maintenance of composite bridges, as opposed to concrete or steel bridges is estimated.

1.7 TRENDS INFLUENCING RESEARCH FOR COMPOSITE BRIDGES

In general, economics may be achieved by designing composite bridges due to a reduction in dead weight when compared to concrete alternatives. When compared with non-composite steel alternatives, greater stiffness and smaller members sizes may be achieved when composite action is taken into account. The driving force influencing composite bridge research, however, is the continuing effort to reduce costs significantly below those for conventional reinforced and prestressed concrete alternatives.

Few (if any) technical impediments inhibit engineers from designing economical composite steel-concrete bridges with span lengths up to and greater than 300 m. Indeed, a wide variety of composite bridges have been extensively studied and built, including:

- Multi-girder bridges with flat concrete slabs. This includes variants with in-plane and corrugated webs, internal and external prestressing, straight bridges and curved bridges and varies types of composite decks.
- Box and multi-cell bridges with a flat concrete slab.
- Cable-stayed bridges.

Various connection methods and types have been studied and employed in composite bridges. These include: headed studs, groups of headed studs, angle connectors and welded plates with holes or cut-outs. Various construction and erection techniques have been studied and successfully used. These include the following: launching, central casting and pushing, casting in place using movable and permanent formwork and the erection of completed steel sub-assemblies and or entire composite spans. Various methods have been examined and tested in practice to control concrete cracking. These include: reinforcement, prestressing of the concrete slab (both longitudinal and transverse), external prestressing and the lowering of internal supports. Little information is available in the technical literature concerning the design of composite details. Such details may be present near supports or diaphragms.

These advances have met with varying success. In some countries (or regions of a country) composite bridges are commonly built, and yet in others not a single significant design can be sited. The explanation for such differences are not purely technical or economic. Countries (or regions) with a significant number of composite bridges also tend to have the following:
- Research or design oriented organisations which actively participate in writing competitive steel and concrete design specifications. These same organisations publish commentaries, design examples and design guides. In many cases they lend assistance during the initial design phase.
- A competitive bidding system and owners which recognise the importance of life-cycle costs (as opposed to initial costs) or those who are willing to pay a premium for innovative designs and architecturally pleasing structures.

In most countries composite construction remains a field in which the average designer has limited experience and generally fears to explore without the luxury of design examples, guides and a generous work schedule. Composite bridges, however, are capable of providing a high degree of user comfort and significant life-cycle cost benefits for their owners, when compared with other bridge alternatives.

1.7.1 EVOLUTION OF COMPOSITE HIGHWAY BRIDGES

In the last 20 years a number of factors have combined to significantly increase the competitivity of steel and composite highway bridges [Owens, G., 1992; Dowling, P., 1992]. Most of these factors are integrally related to advances in fields of research which are external to civil engineering.

First, and foremost, is the continued increase in labour costs relative to materials. More, and larger, components are being shop fabricated and welded. Shop fabrication costs are being reduced due to increased automation. Shop fabrication and welding quality is superior to those done on site. Designers take best advantage of these advantages when they simplify cross-sectional geometries (such as by eliminating stiffeners) and use repetitive sections where possible. In addition, job-site formwork should be reduced. This can be done using in a variety of ways, depending upon job-site location and accessibility.

Second, erection costs are being substantially reduced due to the price and availability of high capacity cranes (both land and water based). Large steel sub-assemblies can be lifted directly into place. In some cases an entire composite spans can be lifted into place. Other construction methods, such as launching, have proved their worth many times over. A designer should choose the construction and erection method best suited the conditions at the job site.

Third, most design codes now use the limit state philosophy. This implies that sophisticated computer analysis can (should) be used to calculate structural behaviour at service and ultimate loading conditions. Such analyses are now within the reach of all designers, and allow composite structures to be designed with relative ease compared to "simplified" procedures mandated several years ago.

Fourth, recent innovations promise to change the cross-sectional geometries of many future composite bridges. These innovations include the use of corrugated webs, external prestressing and new types of composite decks and new types of shear connectors.

Lastly, designers now have access to previous experiences for a wide range of composite bridge systems. This allows more competitive construction cost estimations to be made, particularly if the contractor has previously experience with composite construction. Information is also available on repair and long term behaviour. The long term behaviour of many difference crack control methods can be compared. Serviceability aspects, such as deflection, vibrations and proper water control have been examined.

1.7.2 EVOLUTION OF COMPOSITE RAILROAD BRIDGES

The number of references for railway bridges is limited. This is thought to be due to the fact that many railway bridges are "in-house" designs. This may be due to the fact that railway authority control every aspect of a bridges life. This includes:
- *Loadings*. Trains (dead load, wheel spacing, etc.), the weights they carry, and the speed at which they may cross the structure are determined by the authority.
- *Repair and maintenance*. The same authority that designs the bridge must repair and maintain it.
- *Replacement*. The same authority that designed the bridge will eventually be responsible for replacing it.

Railway designers are more concerned with life-cycle costs than with initial construction costs. The most important trend in composite railway bridge construction, however, are identical to those identified for composite highway bridges:
- The continued increase in labour costs relative to materials.
- Reduction in erection costs are being substantially reduced due to the price and availability of high capacity cranes (both land and water based).

Ballasted bridges are becoming popular, due to the following advantages:
- *Lower maintenance costs*. Track laying and maintenance machines can cross bridges without any special provisions.
- *Noise reduction*. A ballasted structure has more mass than a non-ballasted structure. The ballast itself isolated the bridges from the train.
- *Fatigue resistance*. The structure is isolated from individual wheel loads, this limits the number of damaging cycles (one per train). The stress range is reduced due to the effect of increased dead load.

The proliferation of high speed trains in the last ten years has opened a new field of study for railway bridges. Two serviceability criteria are of utmost importance for such structures: vibrations (noise) and increased deflections due to dynamic forces (passenger comfort).

Composite construction has been shown to be competitive with both all steel and reinforced or pre-stressed concrete alternatives.

1.8 RESEARCH NEEDS

In recent years the ultimate load carrying capacity of composite bridges has become less important as a research topic. Increasingly researchers and code writers are examining the following subjects:

- Serviceability. This refers specifically to the following areas:
- Deflections (user comfort), vibrations (noise reduction).
- Corrosion prevention (water control).
- Concrete cracking.
- Maintenance, repair and rehabilitation of existing bridges. As traffic loads and volume increase, existing structures must be upgraded or replaced. Methods of upgrading existing structures include providing composite action, external prestressing and adding diaphragms to increase two-way action.
- Developing simple design guides and commentaries. For designers with little previous experience in composites. Such rules should be adapted to the present realities of the service limit philosophy and the availability of computer based design and analysis packages.

2. DESIGN

2.1 BRIDGE TYPES AND DESIGN

In section 2.1.1 an inventory is given of composite bridge-types found in literature.
In section 2.2.2 composite bridge designs are discussed more elaborately for frequently encountered bridge types.

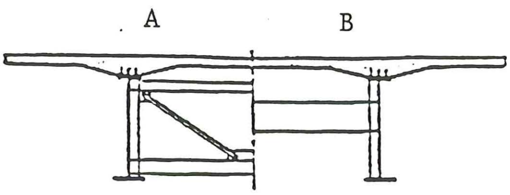
2.1.1 BRIDGE TYPES

The following types are found in literature:

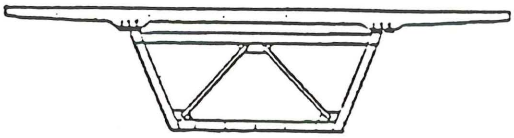
1. Beams (plate girders) with K-bracing or a cross-brace;
2. Open caisson (closed on top by the slab), made of plate and bracing;
3. Closed caisson, made of plate and bracings;
4. Two or more closed caissons;
5. Parallel embedded rolled beam-sections ;
6. Parallel rolled or plate girders with a concrete slab;
7. Triangular shaped steel-section with a heavy bottom-girder, filled with concrete and side bracings between the bottom-girder and concrete plate;
8. Concrete top and bottom slab connected by steel braces;
9. Prestressed embedded rolled girders (preflex);
10. Ballasted concrete upper-slab with K-braced plate girders;
11. Concrete top- and bottom slab connected by side-girders;
12. Concrete slab supported by truss-girders;
13. Concrete bridge in which the concrete deck and cantilevers are supported by steel braces;
14. Other types, for instance external prestressing.

An overview of these composite bridge types is given in figure 2.1.

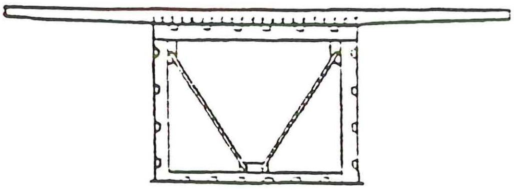
In this inventory no difference is made between road bridges and railway bridges. There is one exception: type 1 and type 10 are similar types of composite-bridges. The main difference between these types is the ballast-foundation for railway-tracks present in type 10. This fact and the difference in dimensions and spans were reasons to separate these two types. Only the examples are split up in railway- and road bridges.



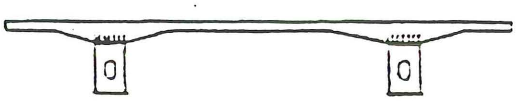
1. Beams (plate girders) with K-bracing or a cross-brace.



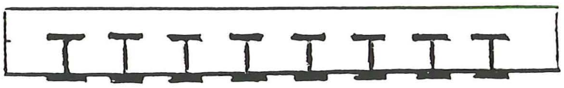
2. Open caisson (closed on top by the slab), made of plate and bracing.



3. Closed caisson, made of plate and bracings.

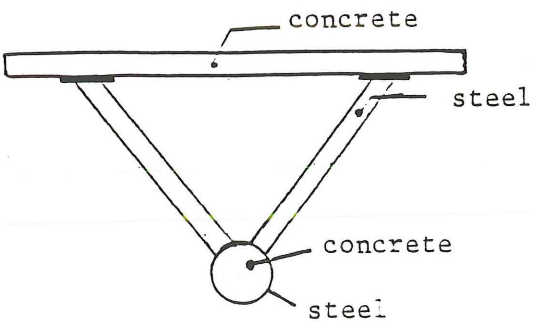


4. Two or more closed caissons.



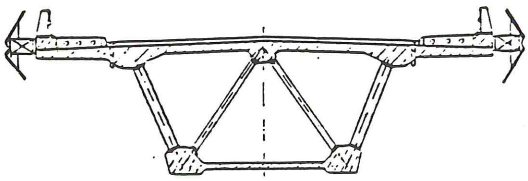
5. Parallel embedded rolled beam-sections.

6. Parallel rolled or plate girders with a concrete slab

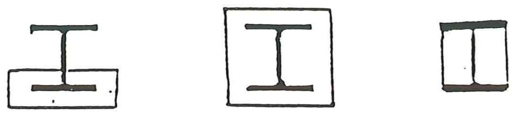


7. Triangular shaped steel-section with: a heavy bottom-girder, filled with concrete side bracings between the bottom-girder and concrete plate

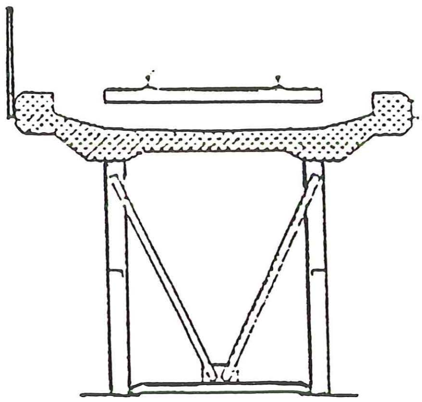
Figure 2.1 : Bridge types



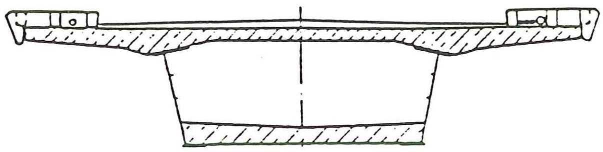
8. Concrete top and bottom slab connected.



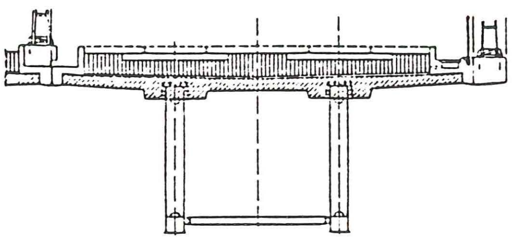
9. Prestressed embedded rolled girders (preflex).



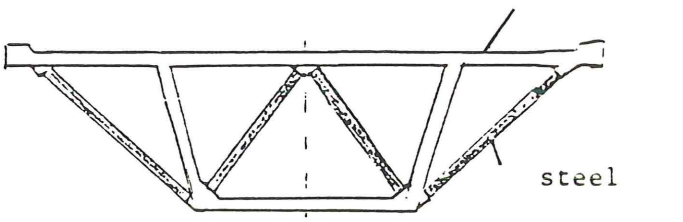
10. Ballasted concrete upper-slab with K-braced plate girders.



11. Concrete top- and bottom slab connected by side-girders.



12. Concrete slab supported by truss-girders.



13. Concrete bridge in which the concrete deck and cantilevers are supported by steel braces..

Figure 2.1 : Bridge types (continued)

Some bridge types are particularly suitable for designing road bridges or railway bridges. Other types of bridge designs do exist for both traffic-types, using other dimensions or spans.

In the next part of this section each type of composite bridge is shortly described and examples are presented.

Type 1 - Beams (plate girders) with K-bracing or a cross-brace

This is an accepted construction type for road bridges with one or more medium/large span(s) (20 - 100 m). The main-girders in the span are supported in transverse direction for the next reasons:

1. stability of the flanges during erection.
2. truss-action for lateral and unbalanced vertical load.
3. form stability.

Most designs have a strong transverse connection between the plate girders above the supports.

Examples of railway bridges:

- Eisenbahnbrücke OW III in Dortmund, Germany
  The bridge is part of a combination of railway- and two road bridges. The example is discussed as a part of the example of road bridges [2.36].
- Obere Limmatbrücke Wettingen, Switzerland
  A three span double-track railway bridge. See also figure 2.2.

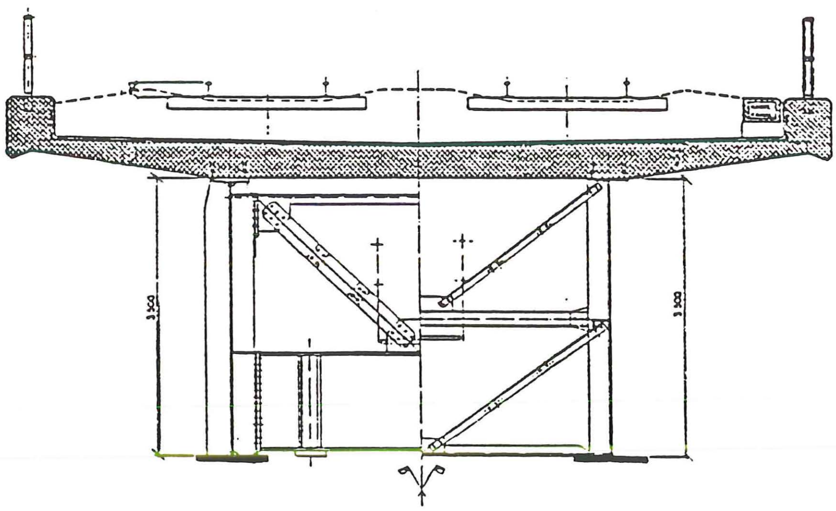


Figure 2.2 : Obere Limmatbrücke Wettingen, Switzerland [2.46]

Examples of road bridges:

- Four lane road bridge across a railway track in the highway Luxembourg-Bruxelles (O-AT5).
  A continuous two span construction with spans of 20 and 30 m. There are eleven main-girders installed. In transverse direction the main-girders are three times connected near the supports and only one time at mid span by transverse girders. The main-girders have a butt-connection near the zero point of the bending moment [2.2].

- Multi span road bridge 'Brookstreet' M25 Motorway, England.
  A multi span road bridge with a width of 34.8 m and a total length of 278 m. The ten main-girders with a depth of 1.2 metres are connected in two different ways. Above the supports transverse girders of thin plate are present and at mid span the main-girders are connected by a truss. Special about this bridge is that the transverse girders are supported instead of the main-girders. This is done because of visibility throughout the length of the bridge seen from under the bridge.

- Multi span road bridge 'Syredale' across a railway-track at the highway Luxembourg-Trier.
  No further information available [2.2].

- Ilford-viaduct in the A406 between South Woodford and Barking, England.
  A 20 m span bridge with a total length of 715 m. The main-girders are connected in two different ways. Above the supports transverse girders of thin plate are present and mid span at the main-girders are connected by a truss. This is the same kind of bridge as the bridge 'Brookstreet' at the highway M25 [2.2].

- Multi span road bridge across the 49th street near Saskatoon, Canada:
  The total length of the bridge is 275 m. The depth of the main-girders follows the course of the bending moments.

- Naarajoki-bridge near Lieksa, Finland:
  This is a three span road bridge with a main span of 54 m. The bridge has continuous girders. No further information available.

- Pont de Joigny across the Meuse, France:
  This is a classical single span road bridge with a main span of 75 m [2.2].

- Autobahnbrücke OW III in Dortmund, Germany:
  The cluster of bridges exists of two seven-span two-lane road bridges, one on each side next to the seven span two-track railway bridge. All bridges are of type 1.
  All bridges have the same construction-depth, probably for architectural reasons. Spans vary between 38.3 and 63.2 m. The steel parts were transported to the site in portions up till 45 tons. Concrete quality is B45 [2.36, 2.41]. See figure 2.3.

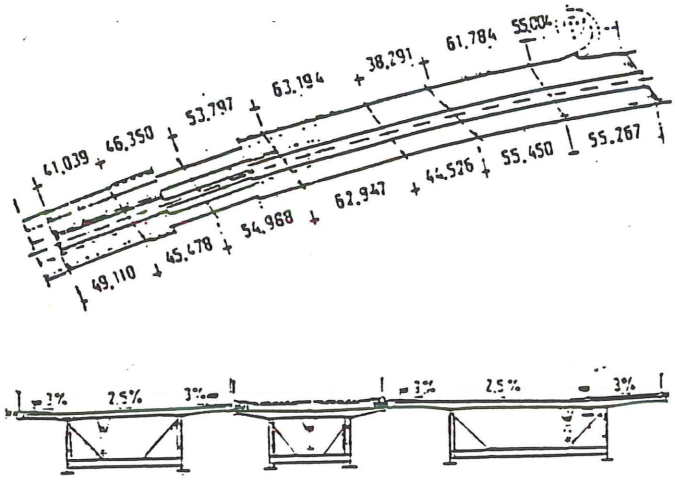


Figure 2.3 : Railway- and road bridges OWIII in Dortmund, Germany

- Eifeltor-Brücke at the Autobahn A4 near Köln, Germany:
  The existing composite type 1a (left side of type 1 in figure 2.1) Eifeltor-brücke is broadened by fixing a type 1b composite structure on each side. Originally the designers planned to fix a same type of composite bridge to the existing one. Despite this a type 1b bridge (right side of type 1 in figure 2.1) was built connected directly to the old bridge. The 6 track road bridge has two spans of 48 m [2.39]. See figure 2.4.

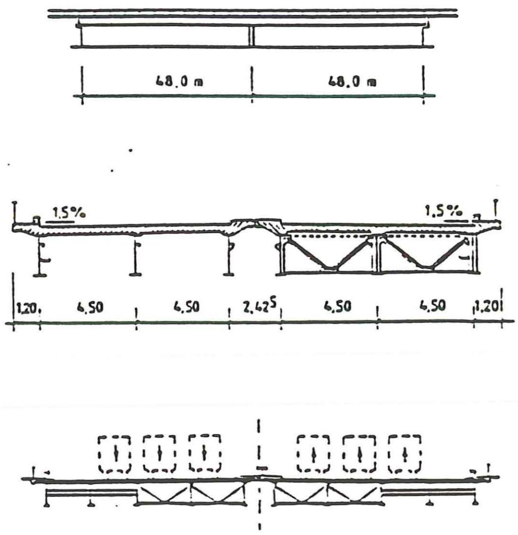


Figure 2.4 : Eifeltor bridge near Koln, Germany

- Bridge over the river Main in the road BAB A 70 Bamberg-Schweinfurt near Eltmann, Germany [2.32]:
  This composite bridge has two spans of 149 m. At the piers the steel section is made of 3000*1500*250 mm steel plate. Because of plate thickness special attention had to be paid to the welding procedure. [2.41]. See also figure 2.5.

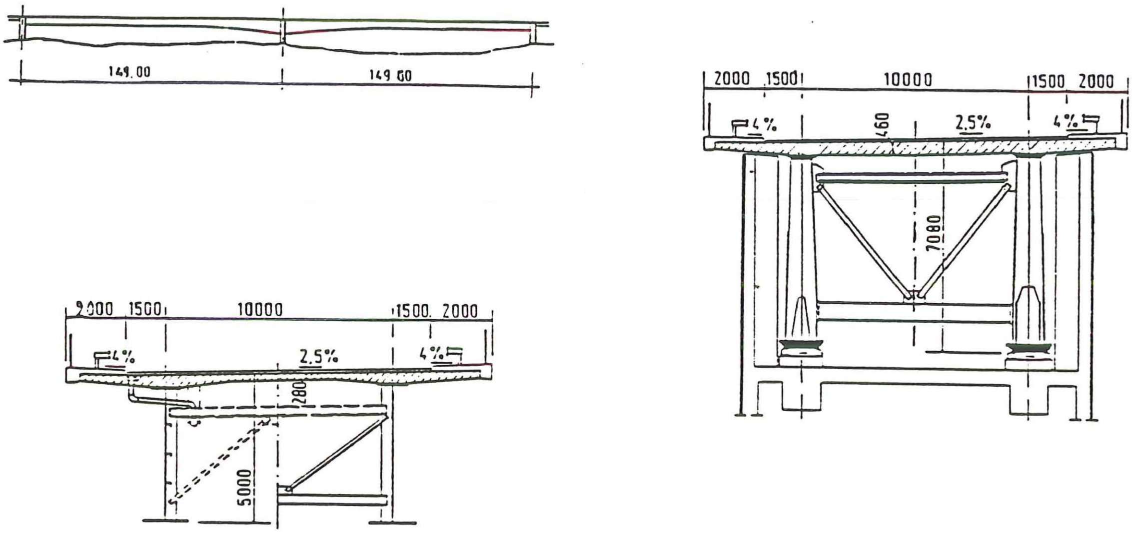


Figure 2.5 : Bridge across the river Main near Eltmann, Germany

- Milton bridges in the highway M74 between Draffan and Poniel near Lesmahagow, Great Britain:
  Two double track continuous three span road bridges with a main span of 54 m[2.48]. See figure 2.6.

- Bridge near Pill im Ötztal, Austria.
  No further information available [2.52].

- Steinbruchbrücke at the Brenner-highway, Austria.
  No further information available [2.52].

- Mietzenerbrücke at the Brenner-highway, Austria.
  No further information available [2.52].

- Gschnitztalbrücke at the Brenner-highway, Austria.
  No further information available [2.52].

- Altersberg-bridge between Salzburg and Villach, Austria.
  Multi-span road bridge with a total length of 839 m and a main span of 90 m.
  No further information available [2.53].

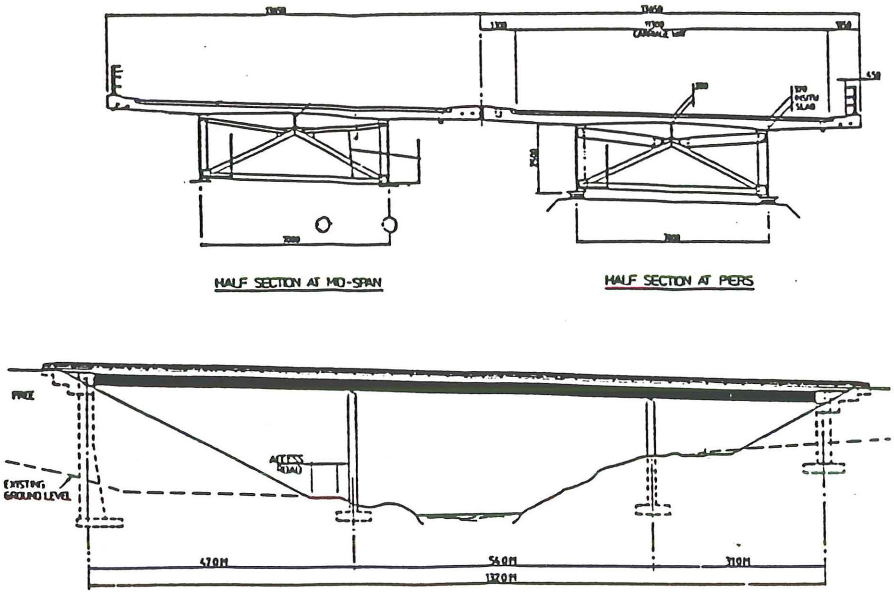


Figure 2.6 : Milton bridges, Great Britain

- Bridge in the Westfaliastrasse, Dortmund, Germany:
  Multi-span road bridge with a total length of 100 m and a main span of 29 m. See figure 2.7 [2.56].

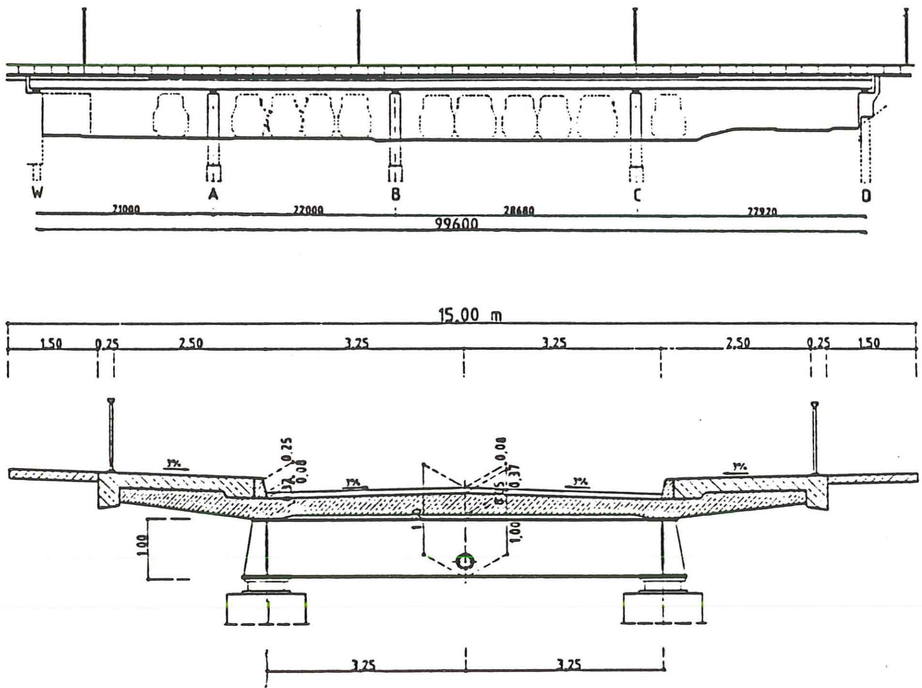


Figure 2.7 : Bridge in the Westfaliastrasse, Dortmund, Germany

Type 2 - Open caisson (closed on top by the slab), made of plate and bracing

This type of composite bridge is particularly useful for large spans or heavy loads (railway bridges). Railway bridges with spans up to 100 m have already been built.
When built as a continuous structure the concrete deck above the piers is usually partially prestressed in longitudinal direction to reduce the tension-forces and control crack widths. The construction-depth of this kind of composite bridge is somewhat larger than in case of an alternative in concrete.

Examples of railway bridges:

- Railway bridge across the river Werra near Hedemünden (Hannover-Würzburg), Germany:
  A continuous (five span) two track ballasted railway bridge with a main span of 96 metres having a construction depth of 6.6 m and a width of 14.3 m. In the transverse direction the concrete deck is partially prestressed. In the longitudinal direction no prestressing is introduced. In later years research has made clear that it is recommended to prestress the concrete in the longitudinal direction in case of statically indeterminate structures.
  The large construction depth was chosen because of the expected high velocities of the trains. The concrete quality is B35. In transverse direction the concrete slab is slightly prestressed. The thickness of the concrete slab varies between 504 and 310 mm.
  Originally it was planned to design this bridge as a steel structure. Similar to the road bridge next to the railway bridge, which was designed as a composite structure, the railway bridge was also designed as a composite structure [2.2, 2.17, 2.41, 2.43]. Also see figure 2.8.

- Bridge at Val Sogn Placi, Switzerland:
  A one-track railway bridge with a span of 54 m. The bridge is built in a curved railway-track with a radius of 400 m. The temporary supports were removed after the solidification of the concrete. In this way the concrete was slightly prestressed. Erection took place in 9 months in 1984 [2.16]. See figure 2.9.

- Baanhoek railway bridge (near Rotterdam), The Netherlands:
  Two statically determinate one-track railway bridges with a span of 58.4 m. The bridge was erected in 1974 and is the only existing composite bridge with a large span in Holland. Calculations learned that a concrete alternative would have been about 5 percent cheaper. Nevertheless the composite structure was chosen because of lower dead load. Reduction of dead load was necessary to keep the loading on the foundation within allowable limits. [2.37]. See figure 2.10

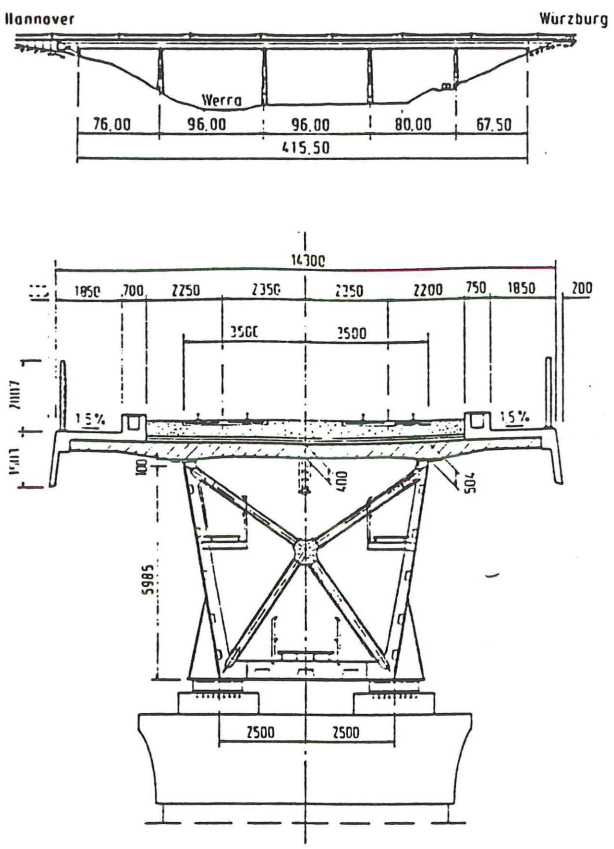


Figure 2.8 : Werra Tal bridge

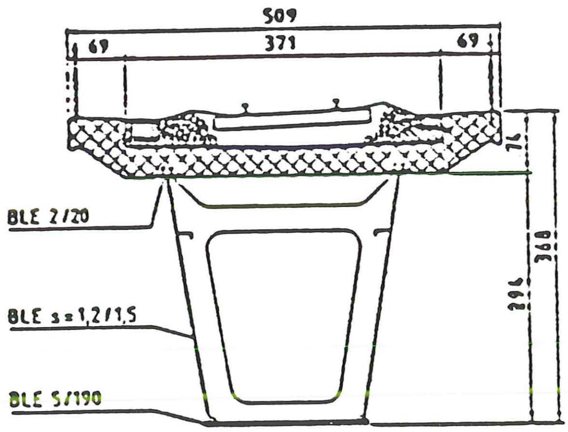


Figure 2.9 : Bridge at Val sogn Placi, Switzerland

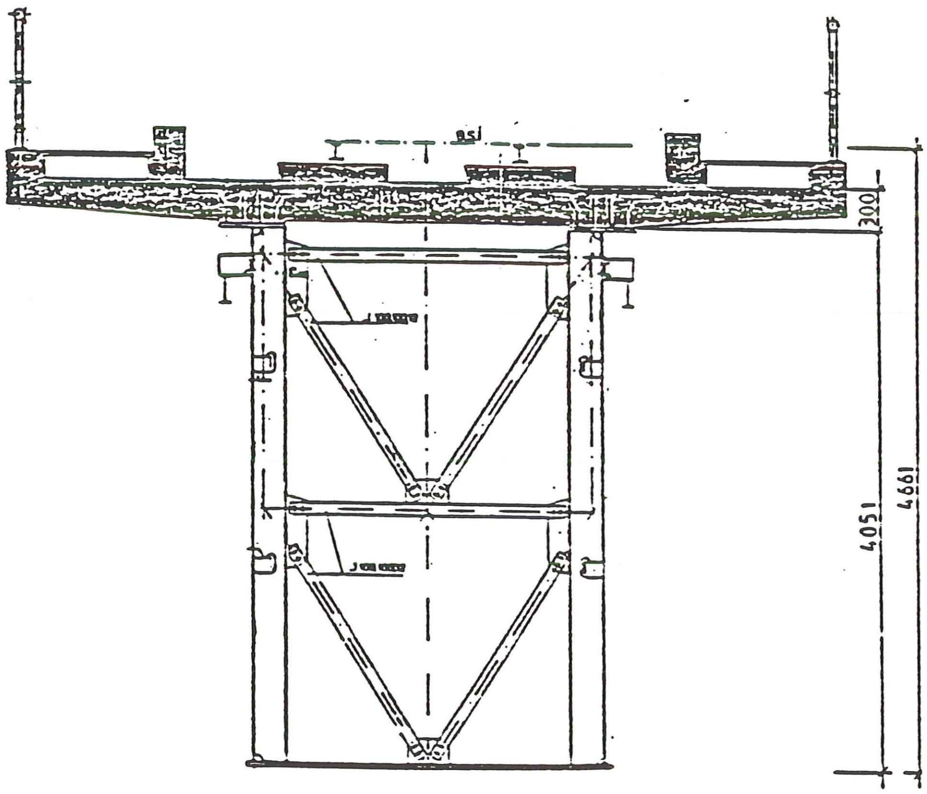


Figure 2.10 : One-track railway bridge Baanhoek, The Netherlands

Examples of road bridges:
- Motorway-bridge near Köln-Eil, Germany:
  A four span bridge with spans between 40.6 and 74 m. The bridge is a continuous structure. Construction height is about 3.2 m and the concrete quality is B45. The thickness at the edge of the concrete slab is only 230 mm [2.2]. See figure 2.11.

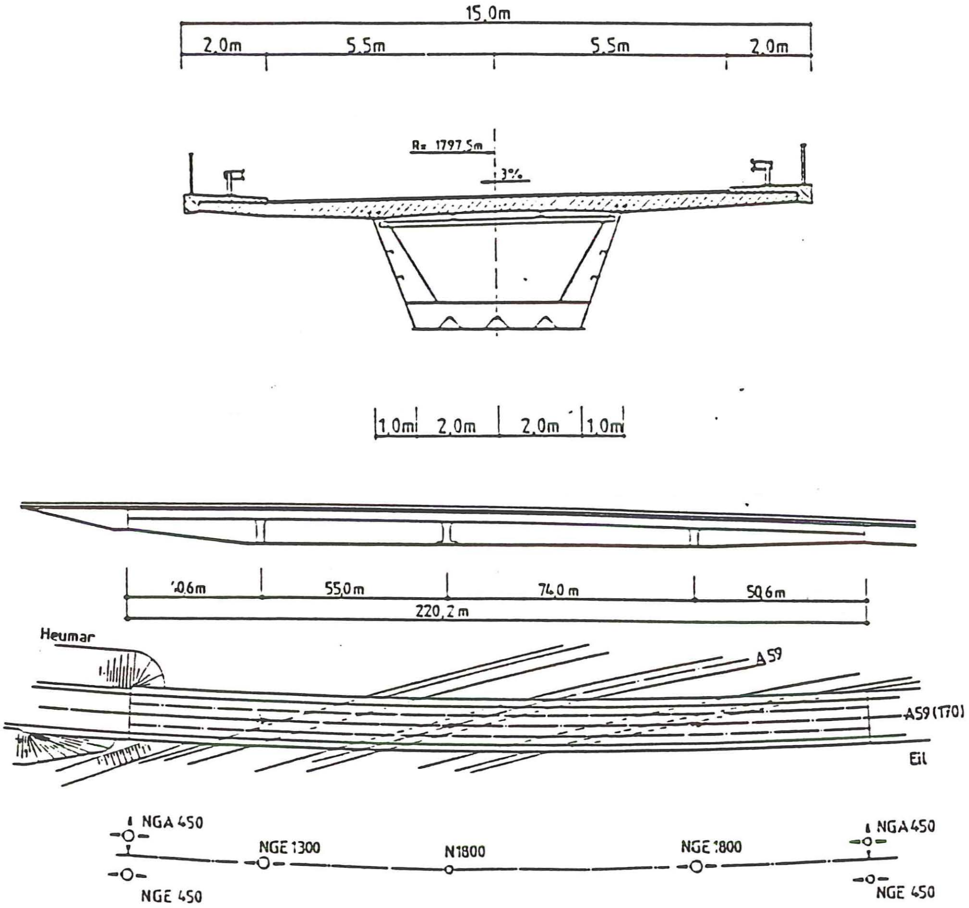


Figure 2.11 : Motorway bridge between Köln-Eil, Germany

- Road bridge over the Hawksbury-river, North America.
  No further information available [2.2].

- Motorway-bridge BW577 across the highway A6 near Kaiserslautern, Germany:
  Skewly (60 degrees) supported two-span bridge with a main-span of 50 m [2.2]. See figure 2.12.

- Motorway-bridge over the Werratal near Hedemünden, Germany
  No further information available [2.17, 2.43].

- Bois de Rosset bridge near Avenches and Freibourg, Switzerland:
  A multi-span road bridge with a total length of 617 m and a main span of 51.3 m. Width of the concrete deck is 13 m and the construction-depth measures 2.5 m. The bridge is externally prestressed. The prestressing units are situated in the caisson. The value of prestressing is chosen so that, under dead load, the concrete is in compression [2.41] (See figure 2.13).

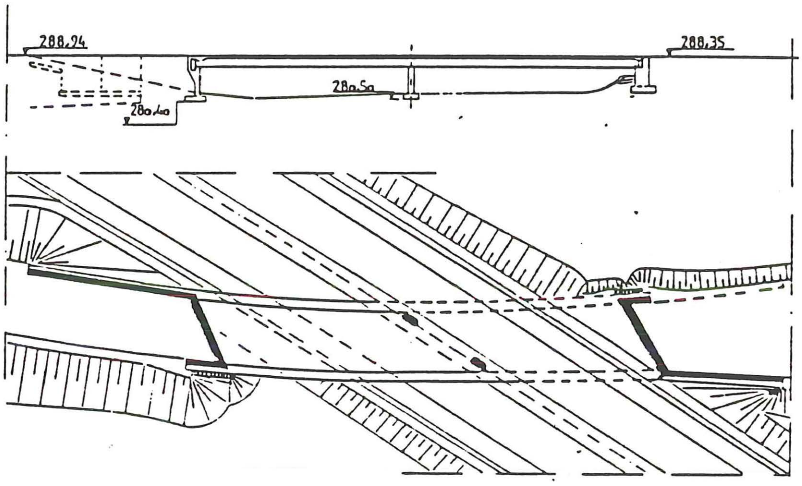
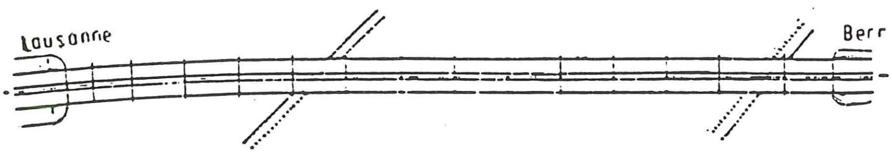


Figure 2.12 : Motorway bridge BW577 across the highway A6 near Kaiserslautern, Germany



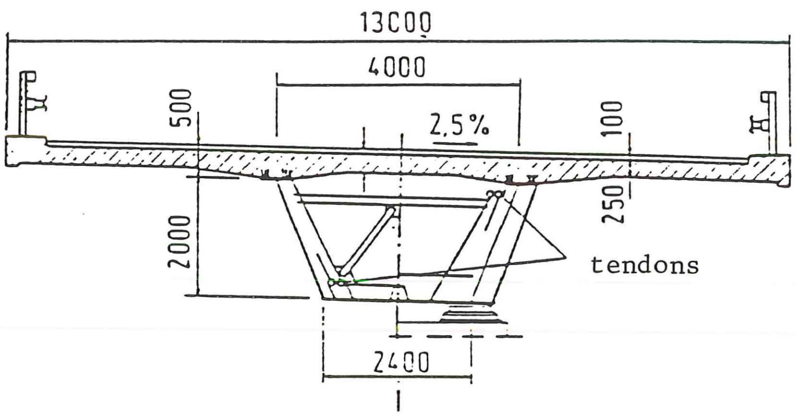


Figure 2.13 : Pont Bois de Rosset, Switzerland

- Viaduct in the road A48 across the river Wye at Chepstow, Great Britain:
  A continuous five-span road bridge with a main span of 93 m and a total length of 302.5 m [2.51].

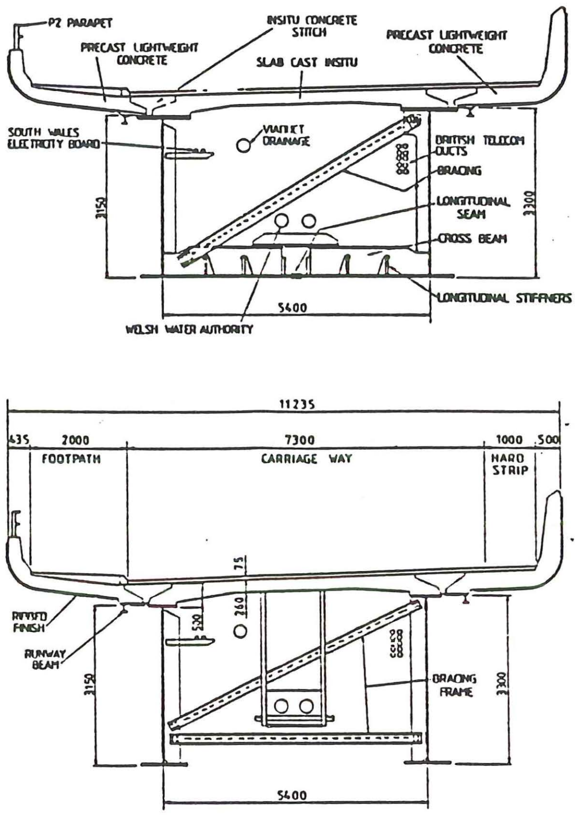


Figure 2.14 : Viaduct crossing river Wye, Chepstow, Great Britain

- Bridge across the river Veveyse, Switzerland.
  The girders are designed as box sections. No prestress is applied, nevertheless the concrete deck is made continuously.
  No further information available [2.53].

Type 3 - Closed caisson, made of plate and bracings

This construction type is similar to type 2. However the caisson is closed on the top side. Advantage over type 2 is the fact that the distribution of forces between steel and concrete is more steady. Perhaps this is a right solution for fatigue. Another advantage is that the formwork is less complicated. The concrete is poured directly on the upper steel plate. This type of bridge is very attractive for large spans and/or heavy loads.

Example of a railway bridge
- Railway bridge across the Inn near Jettenbach, Germany:
  Single track railway bridge with continuous girders. Near the supports the concrete slab is partially prestressed to reduce crack width. Further information not available (could also be a type 2-construction) [2.2].

Example of a combined railway/road bridge
- Combined railway- and road bridge across the Rio Caroni, Venezuela. See [2.55]. See figure 2.15.

Type 4 - Two or more closed caissons

For road bridges this is a very common design. It is mostly known as multi-span bridges. With this construction type it is possible to make large side cantilevers, for instance for a footpath and cycling track. This construction type is also attractive because of the relative small construction depth in relation to large spans.

Examples of road bridges:

- Bridge near Calgary, Canada:
  This bridge is a multi span road bridge. The caissons are connected above the supports only. Total length of the bridge is 190 m. Some composite bridges in Canada are constructed in weathering steel [2.2].

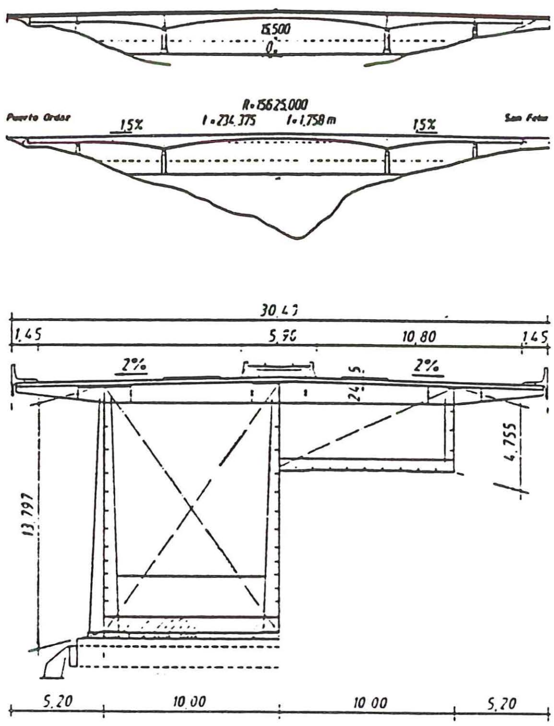


Figure 2.15 : Bridge across the Rio Caroni, Venezuela

- Nene-bridge, Peterborough, Great Britain:
  Five-span continuous road bridge. The total length of the bridge is 155 m, built in a curve of 1450 m. The width of the bridge is 25 m. The main span is about 40 m. The caissons are <u>not</u> connected over the total length of the bridge. Special about this bridge is that the depth (1.2 to 2.5 m) of the caissons follows the course of the bending moments . The distance between the centre lines of the caissons is 6.8 metres. The width of the caissons is 2 m. The concrete slab has a thickness of 300 mm. The transverse span for the concrete is about 4.8 m [2.2, 2.49]. See also figure 2.16.

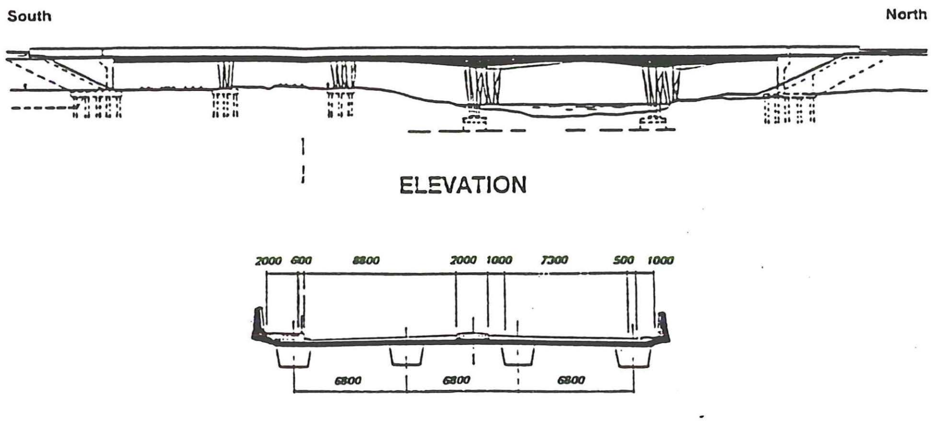


Figure 2.16 : Nene bridge near Peterborough, Great Britain

- Dala-bridge between Leuk and Varen, Switzerland:
  The total length of the bridge is almost 210 m. The main span is 85.4 m. The sloping piers are hinged at the supports and connected to the deck by prestressing [2.41]. See figure 2.17.

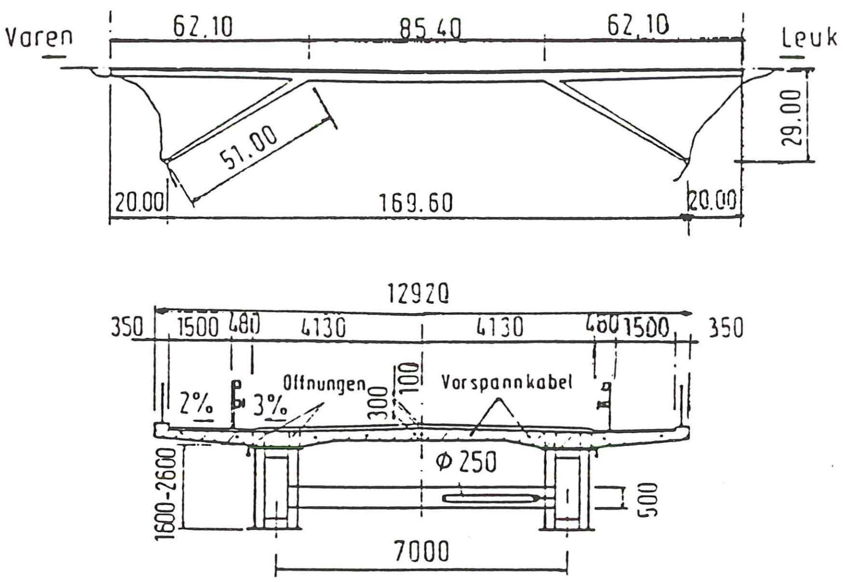


Figure 2.17 : Dala bridge between Leuk and Varen, Switzerland [2.41]

- Eau Rouge-viaduct, in the motorway E42 between Francorchamps and Malmedy, Belgium: The total width of the bridge is 27 m. The main span is 270 m. Both box arches are square 2.7 m and are 14 m spaced. The arches are not connected, except temporarily during erection. The cantilever is 5 m wide and the clear span between the box arches is 11.3 m. The thickness of the concrete slab varies between 180 to 505 mm [2.47]. See figure 2.18.

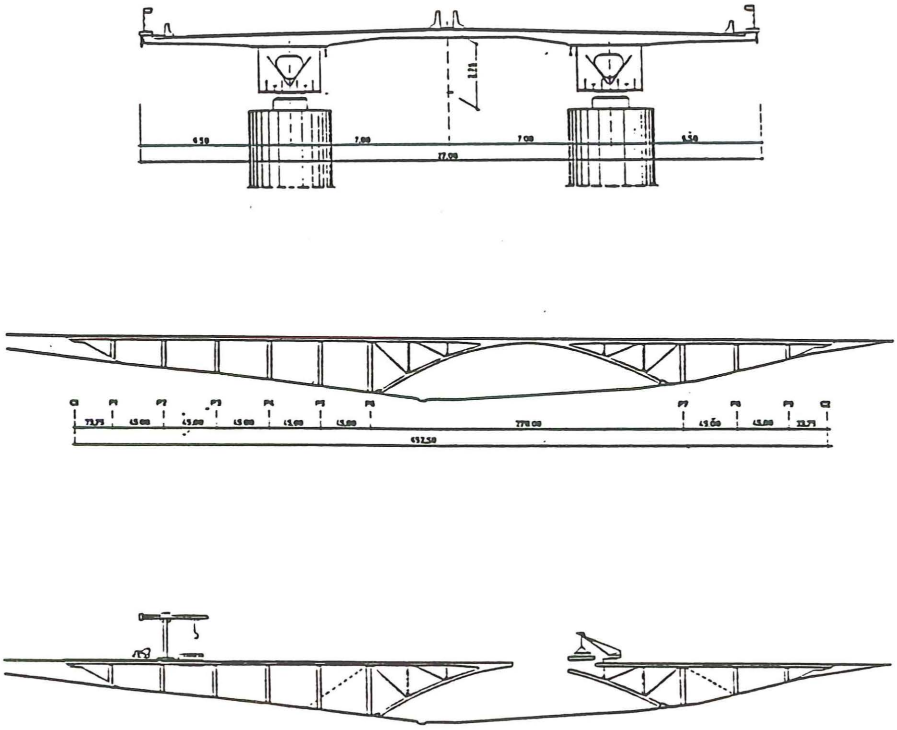


Figure 2.18 : Eau Rouge bridge [2.17]

Type 5 - Parallel embedded rolled beam-sections

This kind of composite bridge is probably the first type of composite bridge ever made. In the past, a lot of these bridges have been built as statically determinate bridges with small spans. This construction-type easily allows for widening (e.g. from two to four tracks) of existing bridges and renovation of bridges in circumstances with limited construction depth. For road bridges this type of construction will not give any problems. For railway bridges the relative large deflection under full load (optimizing on construction depth) often gives restrictions in applicability. See figure 2.19.

Examples of railway bridges:

- Pedestrian tunnel at Utrecht Central Station, Netherlands:
  A single span multi-track ballasted construction. No further information available. In the Netherlands many of these constructions were built and are still being built in complex situations.

- Hansaring Köln, Germany:
  A three span four track ballasted railway bridge. No further information available [2.2].

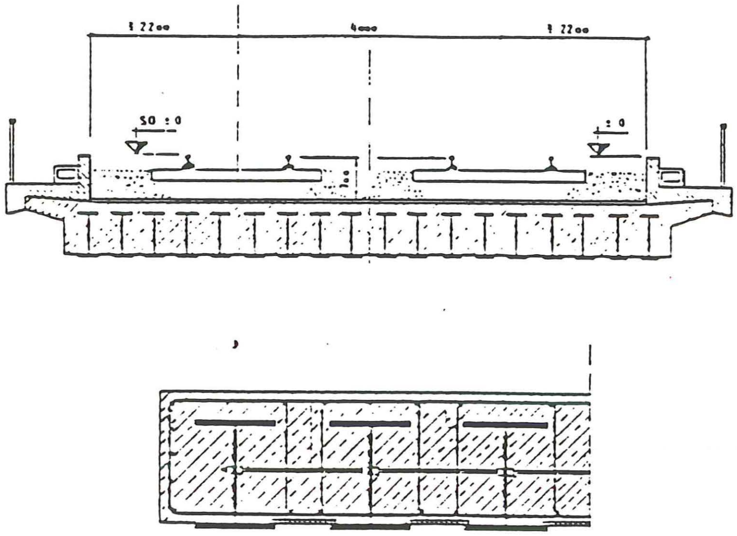


Figure 2.19 : Composite bridge with embedded standard profiles [2.2]

Type 6 - Parallel rolled or plate girders with a concrete slab

This is a very standard model of a composite bridge. It is derived from type 5. Because of the limitations due to large deflections of type 5, one looked for possibilities of increasing the second moment of inertia of the construction. This was found by placing the concrete deck on the top of the steel members and connecting the steel members with the concrete deck by means of shear studs.

Doing this, one introduces other problems (lateral torsional buckling, plate buckling). Because of these new failure mechanisms the increasing of spans was kept limited.

To avoid these problems, type 1 was developed. In type 1 the main-girders are connected in transverse direction to reduce lateral torsional buckling and plate buckling. This results in possibilities to increase the spans.

Because type 6, the composite bridge without transverse connections, has its limitations in length of span it is not common practice any more. To the knowledge of the authors, there is no example available.

Type 7 - Triangular shaped steel-section with a heavy bottom-girder, filled with concrete and side bracings between the bottom-girder and concrete plate

This is a new development within composite bridge structures. In France one example of this type of bridge has been designed. The bridge consists of a concrete slab which rests on a triangular shaped beam made of steel plate. Instead of a bottom slab there is a large steel tube filled with concrete.
The concrete deck and the steel beam at the bottom are connected by corrugated steel webs.

Torsion-stability at mid span is being looked after by both the corrugated webs and the concrete slab. The torsion-stability at the piers is taken care of by the characteristic form of the piers.

Example of a road bridge:

- Charolles bridge across the valley of Maupré, France:
  A seven span road bridge with a total length of 324 m and a main span of 53.5 m. The degree of prestressing is chosen such that the upper concrete slab is always in compression, also above the piers [2.2, 2.41]. See figure 2.20.

Type 8 - Concrete top and bottom slab connected by steel braces

This composite design is composed of an upper and a bottom concrete slab connected by a Warren-truss system. The structure is statically indeterminate and therefore it is necessary to prestress the concrete. Prestressing cables are both placed internally (in the 160 mm thick bottom slab) and externally.

To reduce the losses of the prestressing force because of the lateral stiffness of the truss the designers chose a Warren-truss system which has a relatively low stiffness in longitudinal direction. See figure 2.21.

Example of a road bridge:

- Arbois bridge, France:
  A three-span road bridge with a main span of 40.4 m. Total width of the deck is 11 m by a thickness of 200 mm. The bottom slab has a thickness of 160 mm. Total construction depth is 2.86 m. External prestressing is used [2.41].

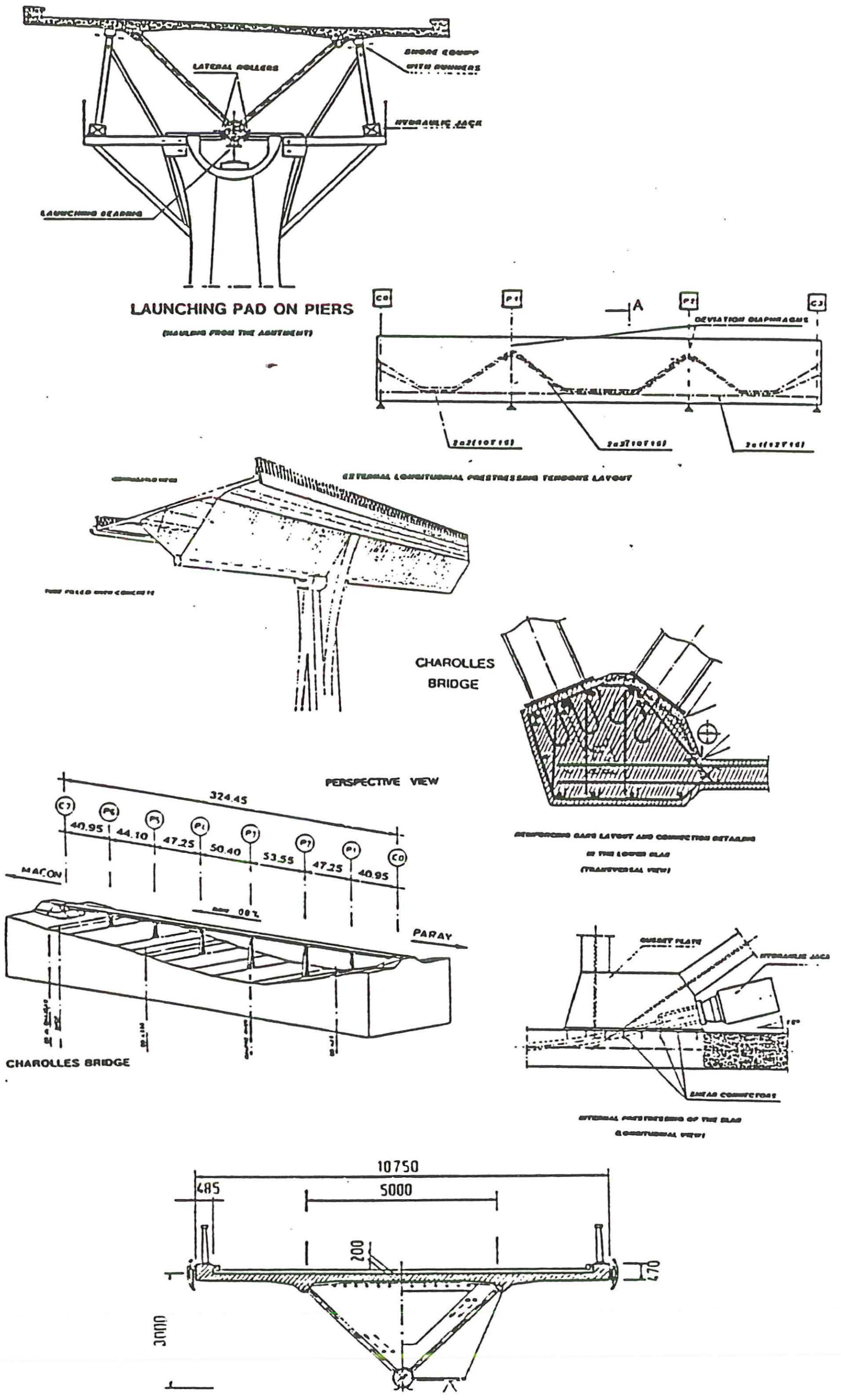


Figure 2.20 : Charolles bridge across the valley of Maupré, France

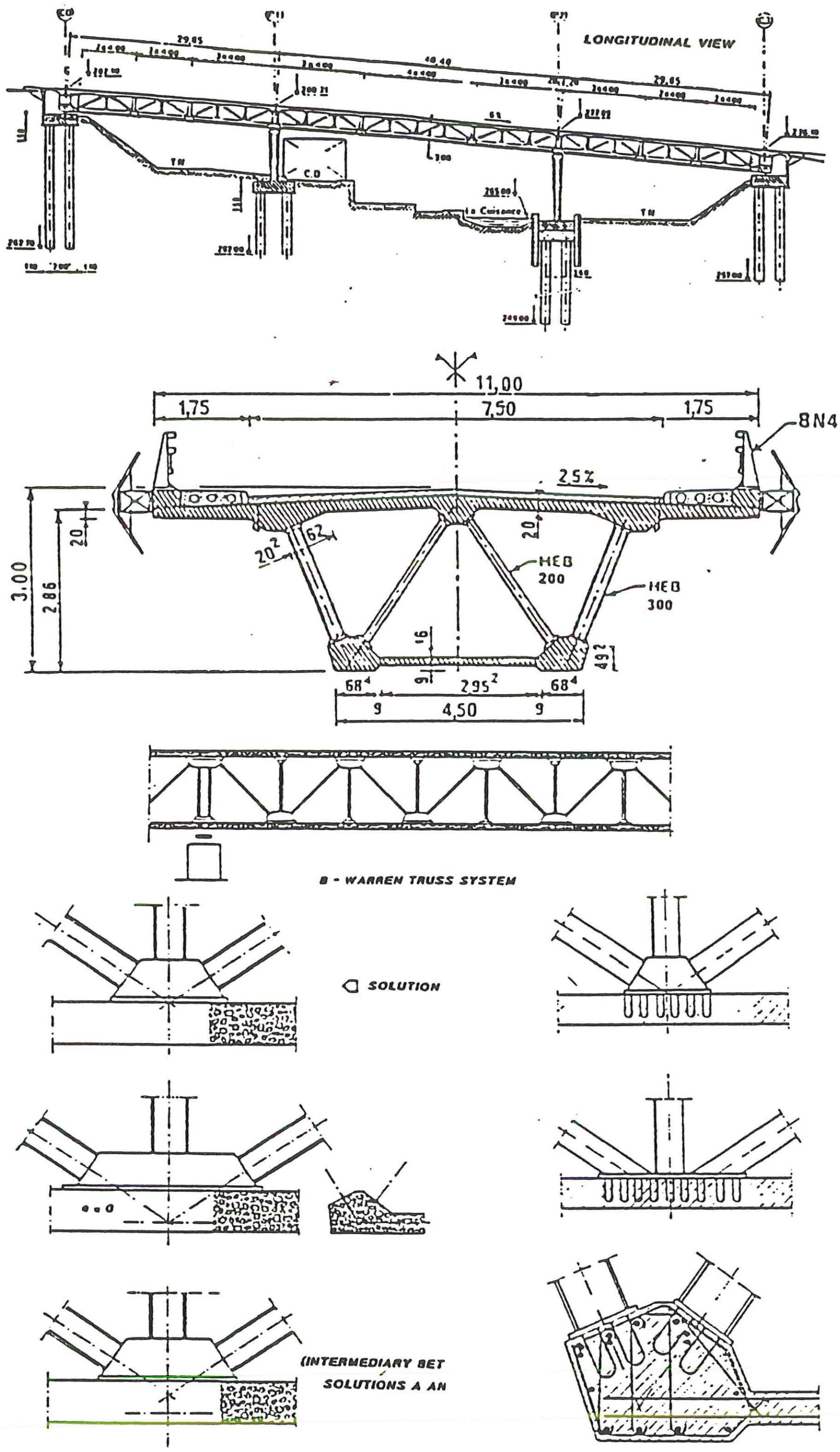


Figure 2.21 : Arbois Bridge, France

Type 9 - Prestressed embedded rolled girders (preflex)

Preflex composite beams are prefabricated beams. In erection this system may therefore have major advantages. Another advantage of the preflex system is the low construction depth. This low construction height in relation to payload is made possible by prestressing the concrete.

This is done as follows (see also figure 2.22):
- first the rolled section is deformed by an external force;
- the formwork is fixed according to the deformed rolled section;
- the concrete is poured in;
- after solidification of the concrete the formwork is released;
- after solidification of the concrete the external deformation-force is released;
- by releasing the external force the deformation of the rolled section is reduced;
- by decreasing the deformation of the rolled section the surrounding concrete is prestressed.

This system was developed in the 50's. Today it is no longer applied as the deflection, (though better than type 5) still causes problems.

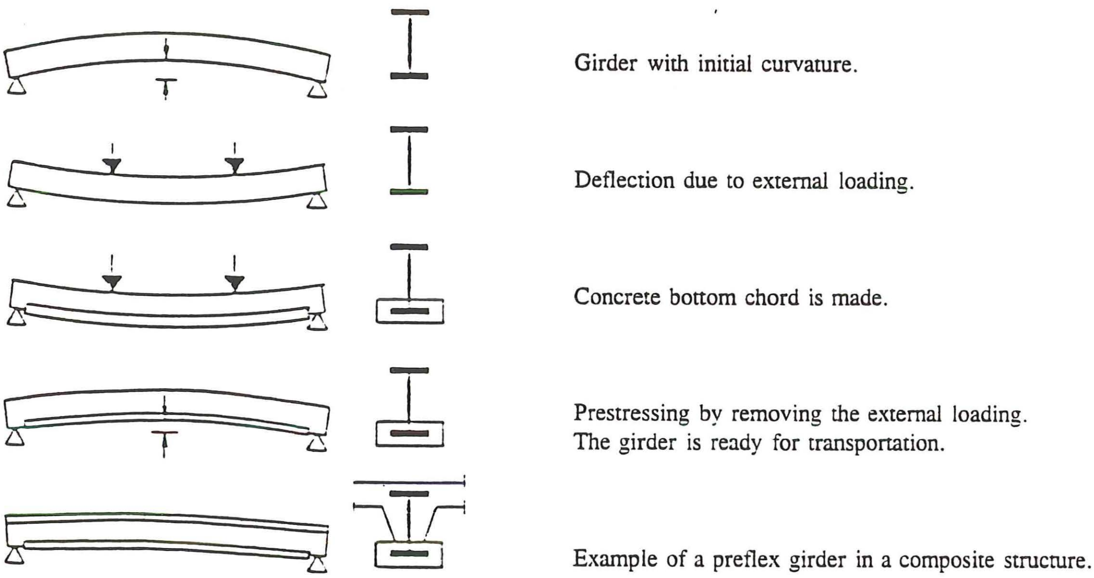


Figure 2.22 : Preflex system

Example of a railway bridge:
- Railway bridge Koblenz-Lindenstrasse, Germany:
  This bridge consists of two one-span double track railway bridges. They originally were designed in steel, nevertheless carried out as a composite structure. The span is about 16 m [2.44].

Type 10 - Ballasted concrete upper-slab with K-braced plate girders

This type of construction is virtually the same as type 1. Type 10 is a special design for railway bridges. There is a ballasted rail track on top of the concrete deck. The dimensions, necessary to design a railway bridge, are quite different from the dimensions of a road bridge with the same span(s).

No examples available.

Type 11 - Concrete top and bottom slab connected by side girders

This construction type is very suitable for statically indeterminate designs. The material is being used to the best of its properties. Concrete used for compression and steel used for tension. Instead of using steel plates as a connection between the concrete upper and bottom slab it is also possible to use a truss-system (type 8).

In statically indeterminate bridges this type of composite structure is combined with type 2. Type 2 is used at mid span and type 11 is used near and above the piers.

Examples of railway bridges:
- Design of the bridge Veitshöchheim: Two track ballasted railway bridge.
  No further information available [2.2].

Examples of road bridges:
- Innbrücke near Wasserburg (between Traunstein and München), Germany:
  The bridge has the following characteristics: two tracks, four spans of 104 m, continuous (figure 2.23).

  This composite structure is a combination of types 11 and 12 as described in this report. Above the pier the cross-section is a type 11 construction. At mid span the bridge looks like a type 2 construction. So the material is used to the best of its properties: concrete under compression and steel in tension. The concrete bottom slab is present over 19.6 m, measured on one span from the pier. The thickness of the concrete varies over 16.6 m between 210 and 650 mm. Near the piers over a distance of 3 m, the thickness of the bottom slab changes from 0.65 to 2 m. For the connection between the steel bottom plate and the concrete bottom slab 350 mm long shear studs were used. In article [2.41] it is stated that this kind of structure also was built in Switzerland [2.2, 2.41].

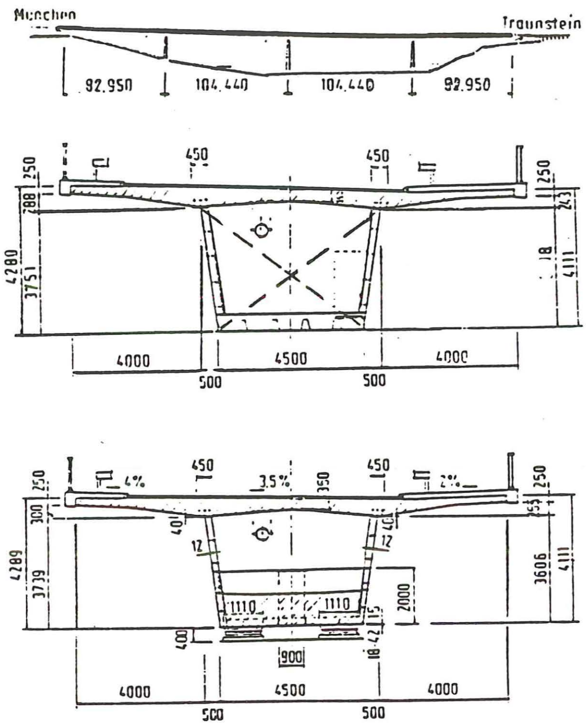


Figure 2.23 : Innsbrücke near Wasserburg, Germany

- Cognac bridge, France:
  This bridge is a three-span road bridge with a construction depth of 2285 mm. The main span of the bridge is 43 m. The thickness of the upper slab is 20 centimetres. The total width of the slab is 12.1 m and the main transverse span for the slab is 4.1 metres. Special about this bridge is the use of external prestressing. To improve the effect of the tendons corrugated steel webs are used. [2.2]. See also figure 2.24.

- Salbris bridge, France:
  One span (39.6 m) road bridge with external prestressing. Total width of the bridge is 7.5 m. Same kind of construction as Cognac bridge. [2.2, 2.41].

- Niedbridge between Saarbrücken and Luxembourg near Rehlingen, Germany:
  This project has two composite bridges with each four spans. Main span about 43 metres. No further information available [2.53].

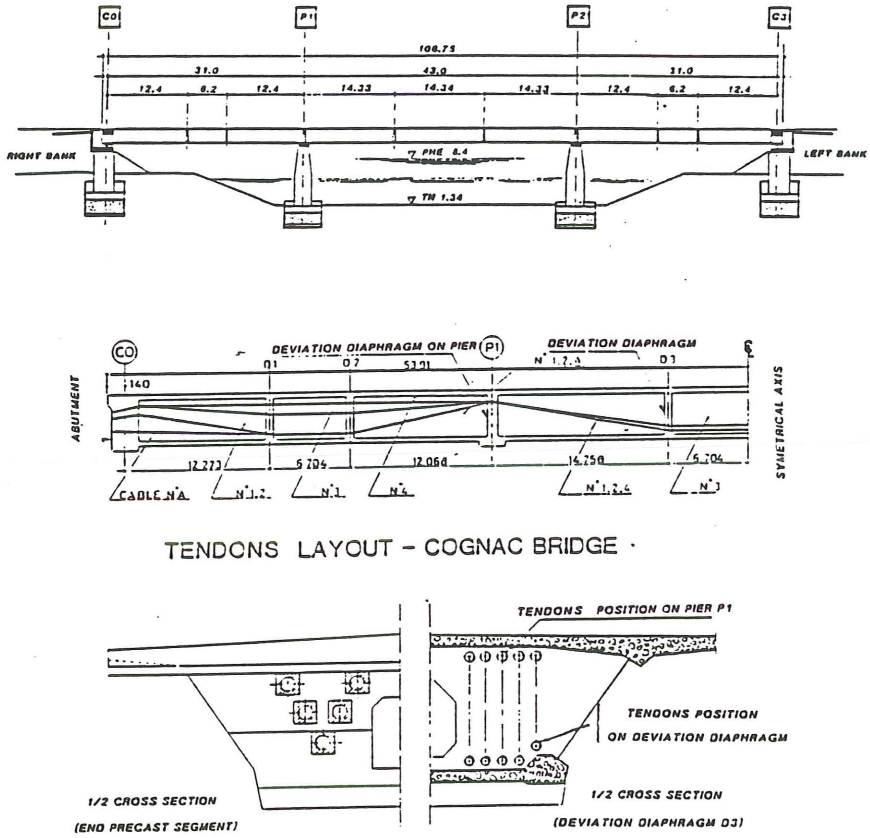


Figure 2.24 : Cognac bridge, France

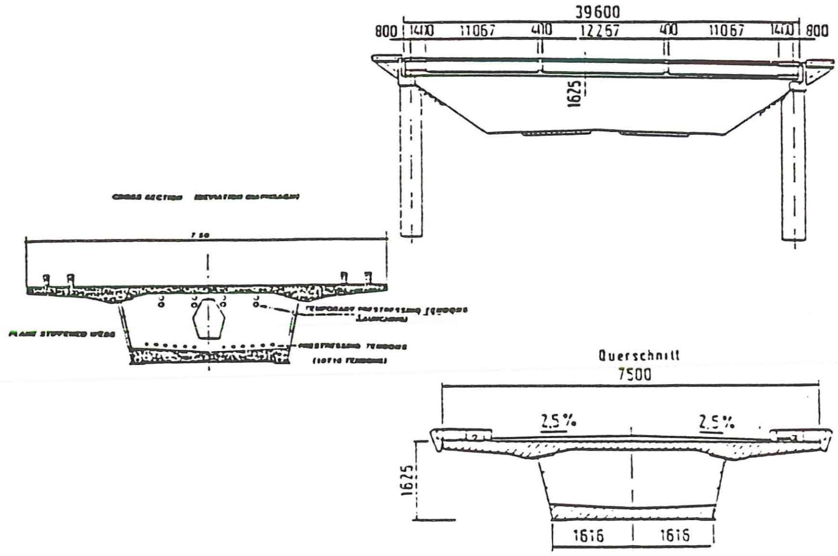


Figure 2.25 : Salbris bridge, France

Type 12 - Concrete slab supported by truss girders

Type 12 is a composite design suitable for large spans and high loads. The main girders consists of a truss, normally situated almost directly beneath the centre of each track of the double track bridge. At some distances the two trusses are connected by a steel beam.

Examples of railway bridges:

- Kragenhöferbrücke over the Fuldatal near Kassel (Hannover-Würzburg), Germany (figure 2.26):
  The double track four span composite railway bridge was chosen instead of a steel bridge, because of the expected lower noise-emission and for landscape-architectural reasons. The main-span of the bridge is 72 m. During erection (sliding method), the statically determinate spans connected by welding, to get a continuous girder.
  After solidification of the concrete deck the connection between the spans is removed. This technique also was applied at the Isar bridge near Grosshesselohe.

  No further information available [2.2, 2.17, 2.41].

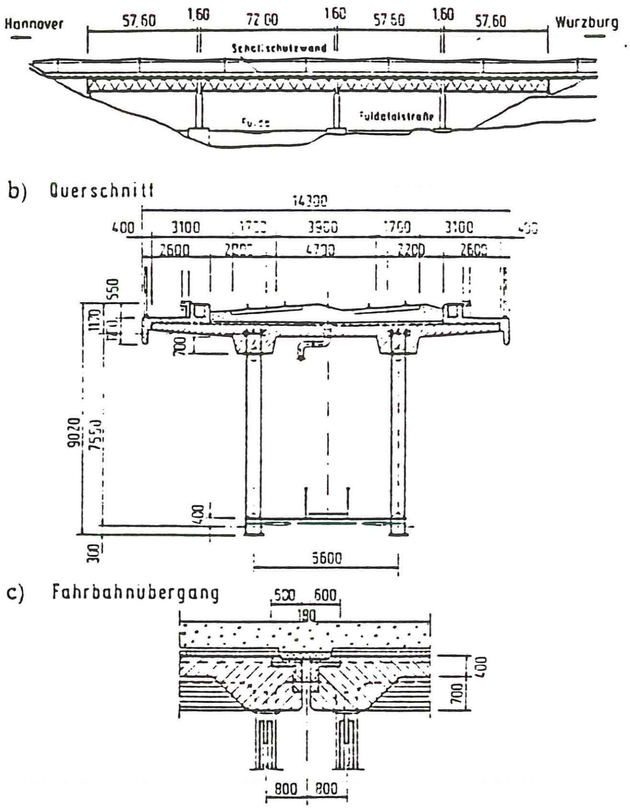


Figure 2.26 : Fulda Tal bridge, Kragenhof

- Nesenbachviaduct between Stuttgart Hauptbahnhof and Böblingen/Flughafen, Germany (figure 2.27):
  This is a two track ballasted railway bridge. The bridge consists of three spans; each statically determinate. The main-span is about 43.5 m. The distance between the main-girders is 4 m. Construction depth is 3.76 m. Special about this bridge is that originally the bridge was completely designed in steel, with orthotropic deck. Nevertheless, in the end, a composite alternative was built, the first composite railway bridge in Germany. The concrete quality B45. The width is 10.23 m. [2.2].

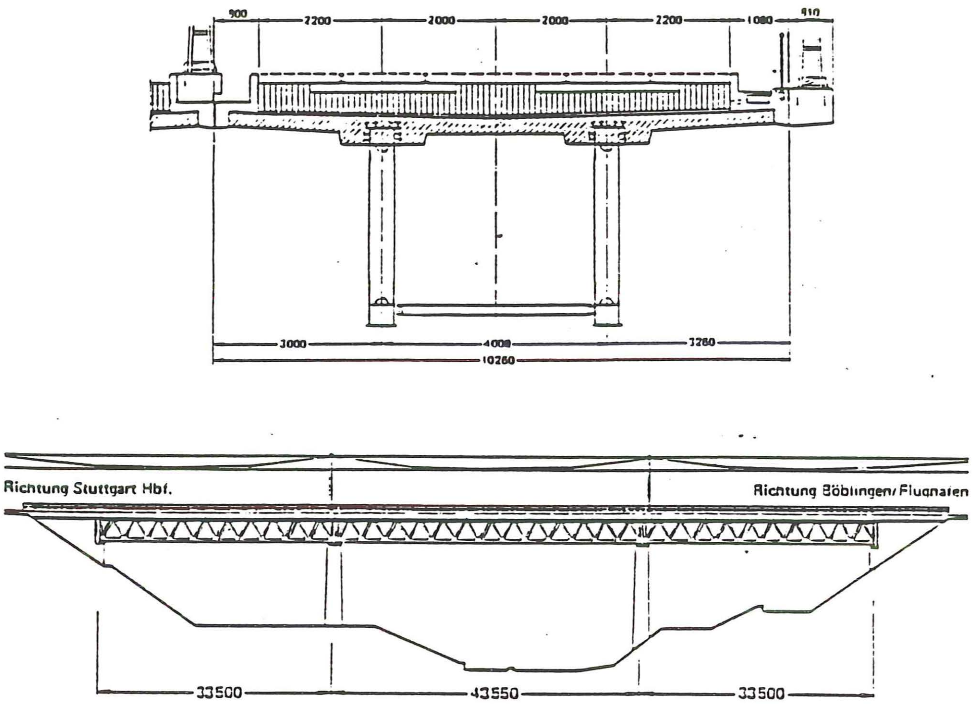


Figure 2.27 : Nesenbach viaduct

- Isarbrücke Grosshesselohe between München and Lenggries, Germany:
  A double track ballasted railway bridge. This four-span bridge has a main-span of 65.7 m. During erection (sliding) the statically determinate spans were connected to get a continuous girder. After solidification of the concrete deck the connection is cut. This technique was also applied at the Fuldatalbrücke.
  The distance between the main-girders is 5.45 m. The top beam of the steel truss is completely embedded in the concrete deck. Therefore this part of the truss is responsible for the composite action between steel and concrete. To optimize the connection between concrete and steel this beam is supplied with studs on three sides. During erection the top beam is placed in compression. Construction depth is 7.2 m [2.2, 2.41].

- Railway bridge over the Weser near Vennebeck, Germany:
  This one-span bridge is suitable for speeds of 250 km/h. Construction depth is about 7.32 m. In addition to shear studs the connection between concrete and steel is also made by an embedded steel beam which is supplied with shear studs. This steel beam provided the compression-side of the truss during erection-period.

  No further information available [2.18]. See also figure 2.29.

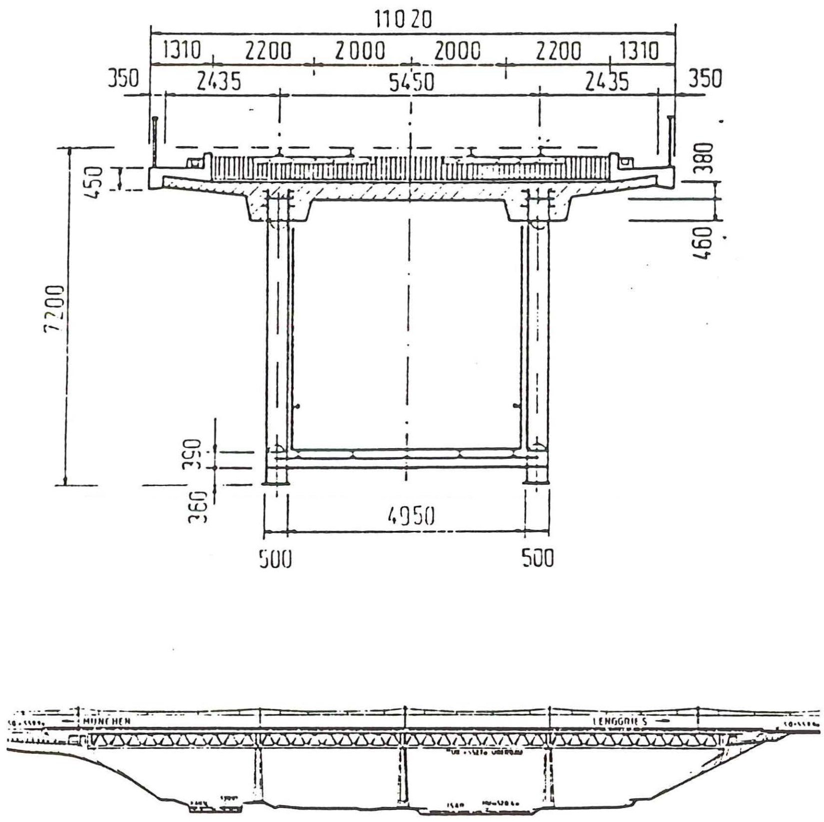


Figure 2.28 : Isar bridge Grosshesselohe between München and Lenggries

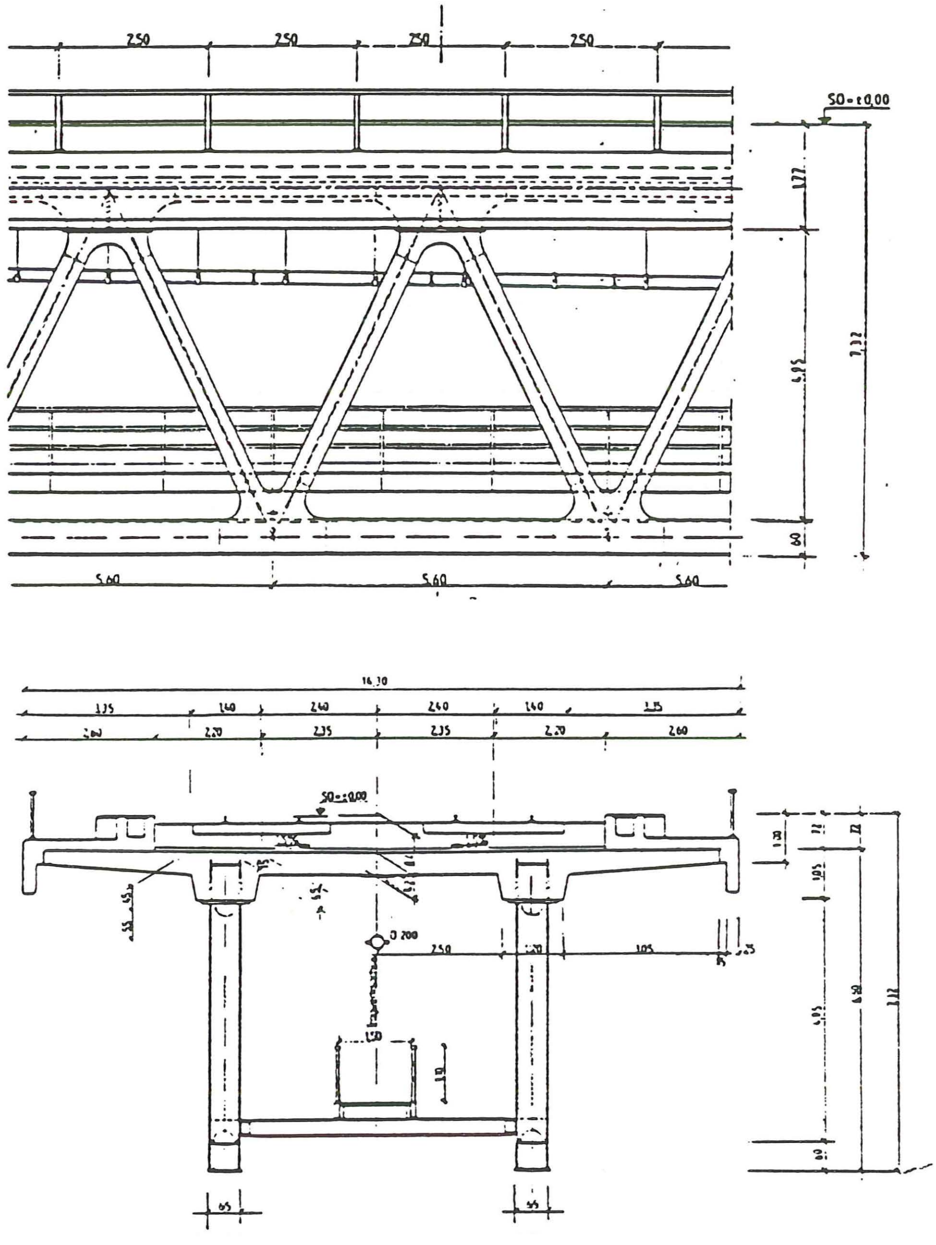


Figure 2.29 : Railway bridge across the Weser near Vennebeck, Germany

Type 13 - Concrete bridge in which the concrete deck and cantilevers are supported by steel braces

This composite structures looks like a standard concrete caisson bridge. In this construction the steel beams are inserted to reduce the thickness of the concrete and reduce the dead load of the bridge. The concrete is supported at the weakest places, the middle of the transverse span and at the cantilevers.

Examples:
- No examples available.

Type 14 - Other types

Examples of road bridges:
- Motorway-bridge over the Kauppental, Germany.
  This bridge is a six span four-track road bridge with a main span of 70 m. Distance between the main girders is 20 m (figure 2.30), [2.2].

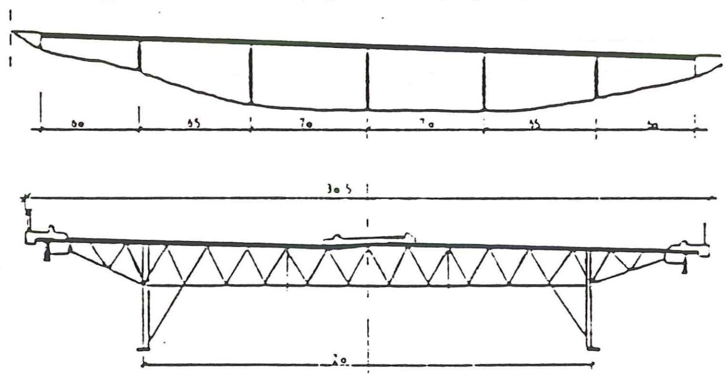


Figure 2.30 : Motorway-bridge over the Kauppental, Germany

  Special about this bridge is the shear-connection between the main girders. This connection is expensive because of the labour costs but economical in terms of material. Because of the price of labour it is not a very likely design these days [2.2].

- Donau bridge Fischerdorf at road BAB A92 to Deggendorf, Germany.
  This road bridge is a special example of composite-bridge of type 2. To increase the possible span/construction-height ratio both composite bridges are supported by an arch. By this method a single-span bridge of almost 102.5 m is realised. [2.41, 2.53]. See figure 2.31.

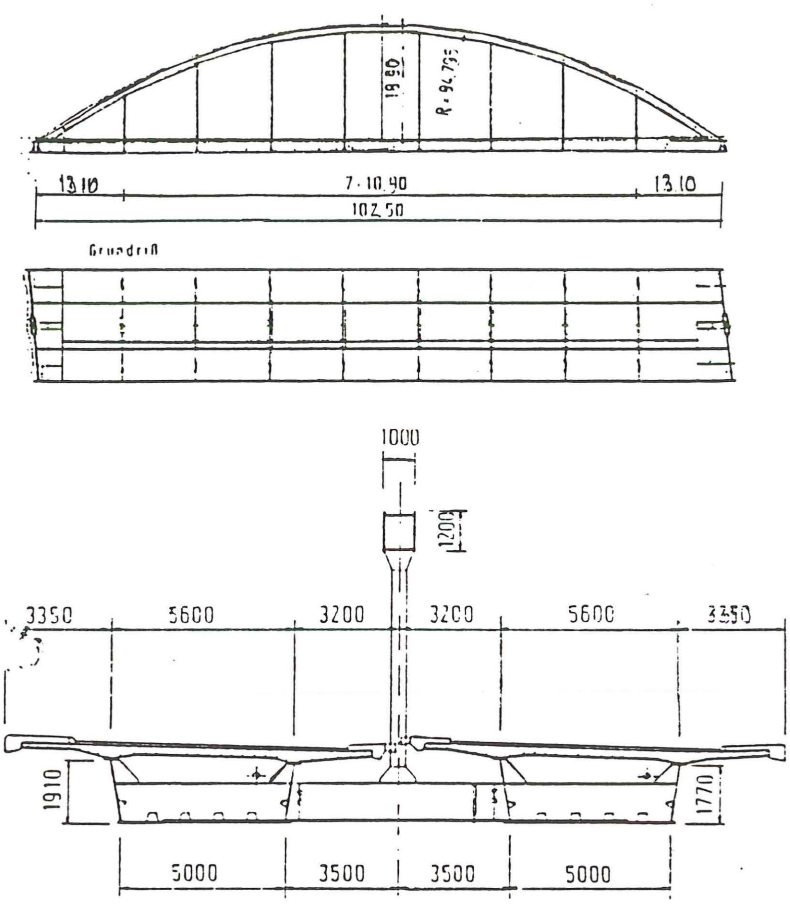


Figure 2.31 : Donau bridge, Fischerdorf

- Bridge of the Rhone near St. Maurice, Switzerland.
  The main span of this cable-stayed bridge is 111.7 ms. The bridge itself is a type 1 construction. Because of the cables this bridge is regarded as a special type. See figure 2.32 [2.41]

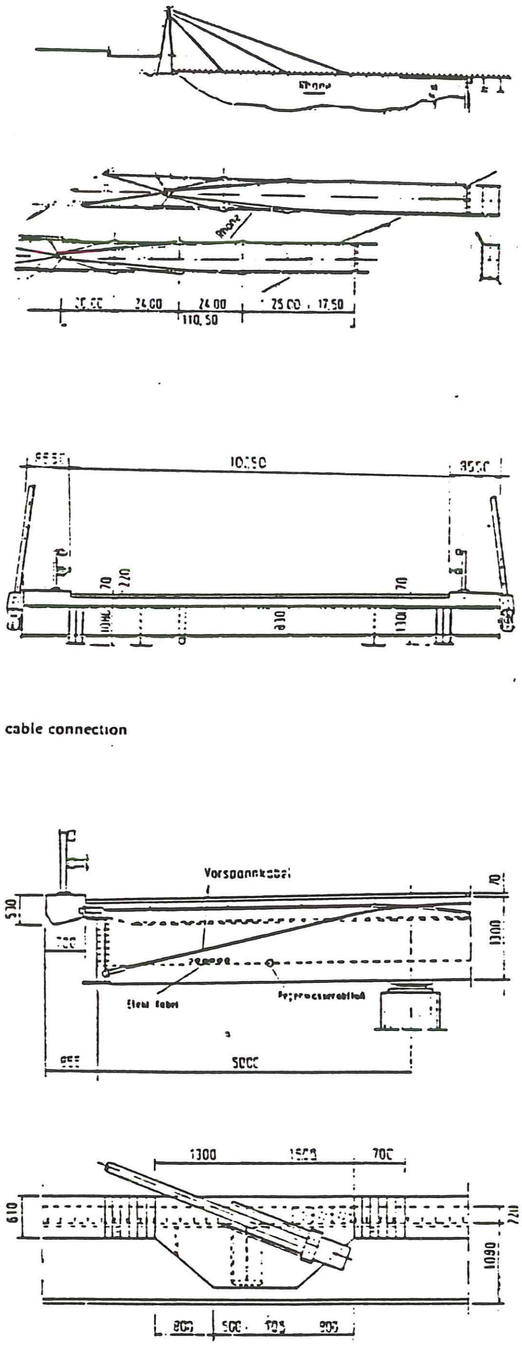


Figure 2.32 : Bridge across the Rhone near St. Maurice, Switzerland

Examples of combined road/train bridges:

- Bridge across the Tocantins river in the north of Brasil.
  Multi-traffic bridge with spans varying between 30 till 60 m. The steel quality is Fe 510 weathering steel. According to [2.18] several of these bridges have been built in Brasil. The most important one should be the bridge crossing the Rio Branco (1200 m). See figure 2.33.

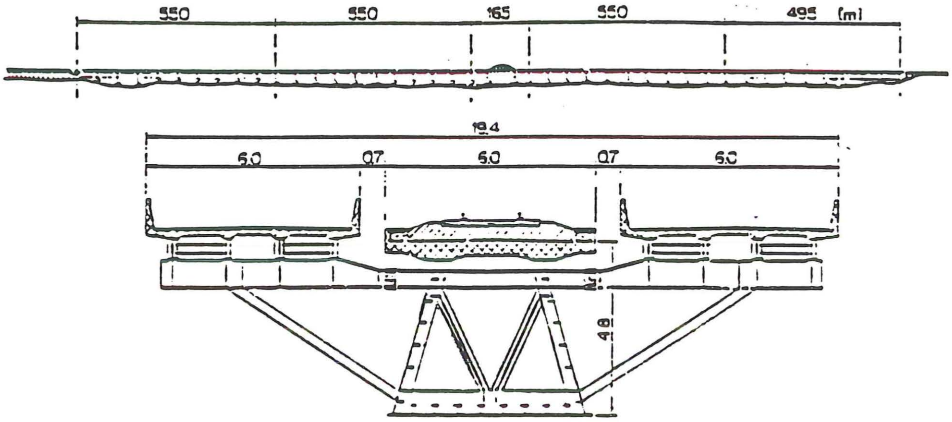


Figure 2.33 : Combined bridge across the Tocantins river (Brasil).

Example of a pedestrian-bridge:

- Pedestrian bridge over the Main-Donau channel near Berching, Germany:
  The bridge is a steel caisson-type suspension bridge. The caisson is completely filled with concrete. The span is 63.25 m. No further information available. See figure 2.34 [2.40].

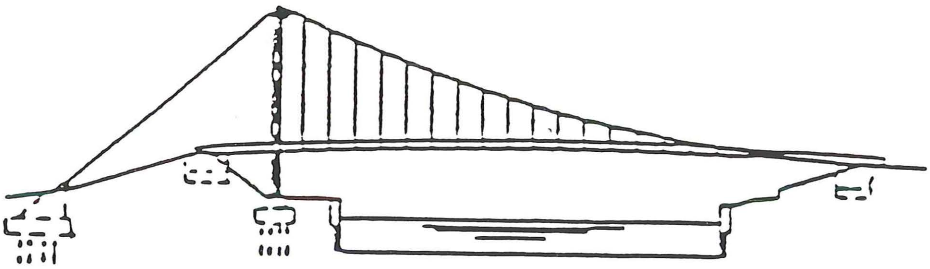


Figure 2.34 : Pedestrian bridge, Berching

Further examples of bridges:

- Biberbach bridge across the Main-Donau channel near Berching, Germany:
  The bridge is a five span continuous bridge with a main span of 67 m. The main spans are designed as a concrete slab with tension elements underneath. No further information available (figure 2.35) [2.40].

Figure 2.35 : Biberbach bridge

- Steinbrückenbach bridge, Austria:
  A suspension bridge with the configuration of a type 1-bridge. Special about this bridge is that nowhere, not even above the piers, a transverse connection between the girders is made. The concrete deck is continuous [2.53].

2.2.2 BRIDGE DESIGNS

In this section, two examples of bridge designs are discussed more elaborately. They concern the bridge types 1 and 12 being the most frequently encountered bridge types in literature.

The type 1 bridge considered, is the Obere Limmat bridge in Switzerland (figure 2.2). This bridge is treated in section 2.1.2.1. The information is taken from [2.46].

The type 12 bridge to be considered is the Nesenbach-viaduct (figure 2.27. This bridge is discussed in section 2.1.2.2. The information is taken from [2.2].

Both bridges to be considered are railway bridges. Type 1 however, is also suitable for road bridges; of course with different spans and dimensions. Type 12 is in fact a special type for railway bridges: no examples of type 12 road bridges are available.

2.1.2.1 OBERE LIMMAT BRIDGE

This bridge is a typical double track type 1 railway bridge (figure 2.36). The bridge consists of three spans. The supporting structures, which date back to 1875, could be used again, almost without modification. The composite bridge, with its closed concrete deck, is a good choice from the viewpoint of noise emission. Other design criteria are:

- The steel structure is easy to produce. Automatic welding processes are allowed for. There is little variation in plate thicknesses along the length of the steel girder. Good access, in order to improve weldability of sections, is achieved.
- The concrete was poured on the steel structure in situ in stead of using prefab concrete deck parts. This was facilitated by structural detailing in order to make easy formwork possible. Bracings were used and the width of flanges was kept constant.
- Shear studs were equally distributed along the length of the steel girder. They build a full strength connection based upon plastic design.
- Prestress was introduced by lifting the bridge at the outer supports thus introducing compression in the concrete deck.

The structure was calculated, split up into two different models. The main girder was calculated as a steel structure (in the fabrication stage) and as a composite structure (for live load). The model used was a plane bar model. The second model used was a plate model for the concrete railway deck. The fact that the bridge is skewly supported was neglected. For the main girder, the following steps were made:

- Cross sectional forces and moments were calculated for 14 cross sections from outer support to half way the field in the middle of the bridge.
- Dimensions were adapted. More intermediate cross-sectional forces and moments were used.
- Structural dimensions and details were chosen. Changes in cross-sectional dimensions were made at different places for flanges and webs.
- A renewed calculation of cross-sectional forces and moments was *not* carried out.

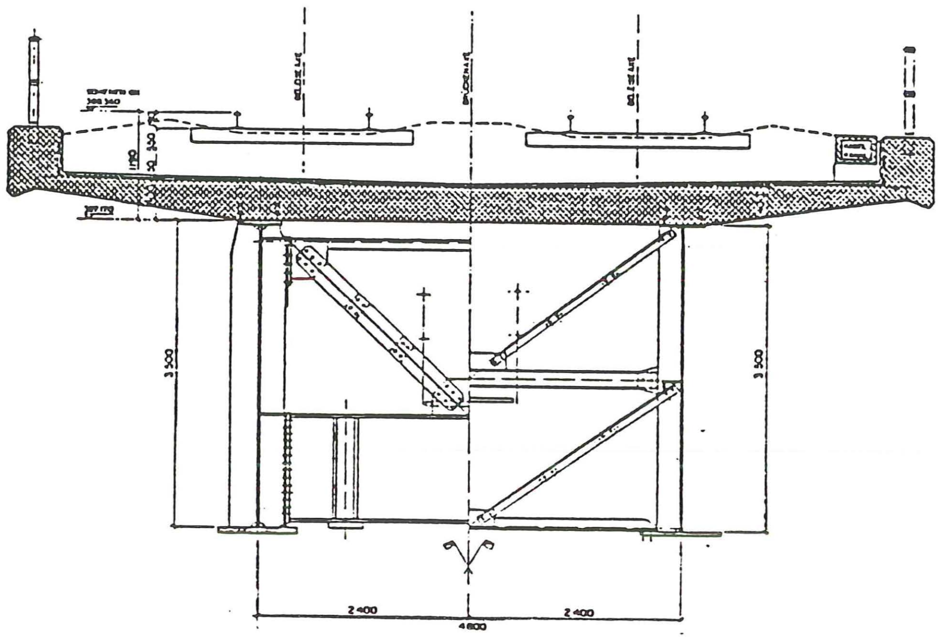


Figure 2.36 : Obere Limmat bridge Wettingen, Switzerland

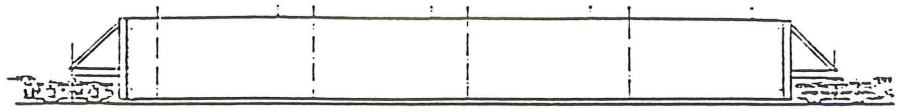
For the concrete deck the following steps were considered:

- Cross-sectional forces and moments were calculated for symmetric loading regard to the central axis. This was done for a plate of 30 m length having elastic supports to represent the main steel girder.
- The stresses from this analysis were combined with those from composite action.
- Structural dimensions of reinforcement were chosen. The serviceability limit state was checked for crack width limitation.
- A calculation for torsion was carried out to analyze one track live load on the bridge.

The crack width limitation calculation controlled the design resulting in a fine mesh of reinforcement with bars in two directions. Many calculations were necessary to check the fabrication stage of the bridge.

In figure 2.36 the cross-section of the bridge is shown. Two parallel main girders were welded, having flanges of 840 mm and 520 mm respectively with thicknesses ranging from 20 to 67 mm. The web thickness varies between 17 and 30 mm. The material used is a fine grain steel FeE 355. The ballasted track is placed on top of the concrete deck. The deck thickness ranges from 200 to 400 mm. Normal reinforcement steel is used. Locally, near the edge of the deck, prestressing is used to minimize crack formation. Compression in the concrete deck is introduced by lifting the outer supports by 300 mm leading to zero prestress in the locally prestressed trough edges. The concrete had a minimum compression strength of 42,5 N/mm^2.

The steel structure was fabricated at the manufacturing plant in sections, thus avoiding much welding in situ. The sections were bolted together using high strength fitting bolts. The sections were transported by rail to the erection site. They were placed on top of the old bridge. Also see figure 2.37.



	Achsstand (Drehzapfen)	Länge über Puffer	Gewicht der Ladung	Achslast	Länge der Ladung	
Schuss I	32,84m	37,59m	53,1 to	18,5 to	28,2m	
Schuss II	30,84m	35,59m	66,7 to	21,9 to	26,2m	
Schuss III	33,24m	37,99m	62,9 to	20,8 to	28,6m	
Schuss IV	30,84m	35,59m	66,7 to	21,9 to	26,2m	
Schuss V	32,84m	37,59m	53,1 to	18,5 to	28,2m	

Figure 2.37 : Transportation of the Obere Limmat bridge

The two old single track bridges were used during erection of the bridge. One of them was used for railway traffic. The other was used to build the new bridge. Therefore, this old bridge was moved in transverse direction and used as a support for the new steel structure. The new steel structure was positioned on top of the old bridge. After completing the new steel structure, it was placed on the sliding tracks and the old bridge could be lowered down in such a way that the new steel structure carried the old bridge. The old bridge was removed. The concrete deck was fabricated in situ on top of the new steel structure. The second old and the new bridge were then moved simultaneously over a distance of about 6 m. This was done in "one night" of 16 hours. The bridge was completed in 1988.

2.1.2.2 NESENBACH VIADUCT [2.2]

This bridge is a typical type 12 railway bridge (figure 2.38) for double track. The bridge consists of three simply supported spans (33.5 m - 43.55 m - 33.5 m). The composite truss bridge is very suitable for railway bridges. The Nesenbach viaduct is the first composite truss bridge built in Germany for Deutsche Bundesbahn. The cross-section is shown in figure 2.38. The concrete plate has two webs supported by two steel trusses. The steel used is R St 52-3 (DIN 17100); the concrete used is B45. Normal reinforcement steel is used in the concrete slab in longitudinal and transverse direction. At the supports, transverse truss girders are made. They contain facilities to lift the bridges for accurate positioning or replacement of supports. Each span has one fixed support, one support which is movable in one direction and two supports movable in two directions. Horizontally, the bridge is fixed by a special plate structure which also functions during e.g. replacement of bearings.

Water on the bridge leaves the bridge through a drain system. At the supports and between the bridges the deck is closed. Joints allow for 30 to 200 mm movement. The concrete deck is made impermeable to water by one layer of "isolation", two layers of tightening material and on top of that a copper strip material. On top of all that, a 50 mm protection layer of concrete B25.

The sides of the concrete deck are made of reinforced concrete B25. Deck and sides are split by one layer "isolation" and one layer tightening material. The steel structure is protected by 5 layers (2x60 μ and 3x70 μ) after cleaning SA 2½.

The braces of the trusses were welded to the chords, in a way quite different from the usual connection with either rivets or high-strength friction grip bolts. The side view of the joints is therefore very clean and joint plates are saved. Joint stiffness however has to be taken into account when calculating the force distribution. Therefore the truss system was calculated with fixed joints in stead of hinges. The structural detailing of the joints was largely determined by fatigue. In figure 2.39 the chord to braces joint is shown.

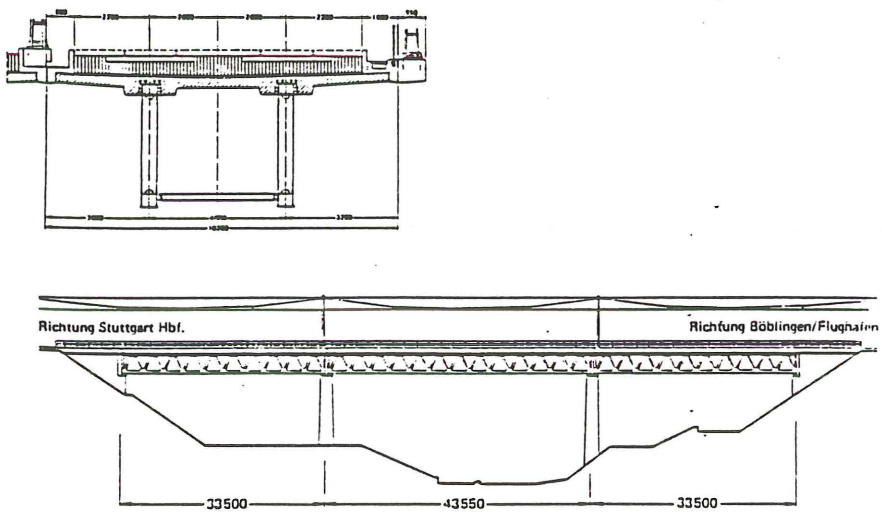


Figure 2.38 : Nesenbach viaduct

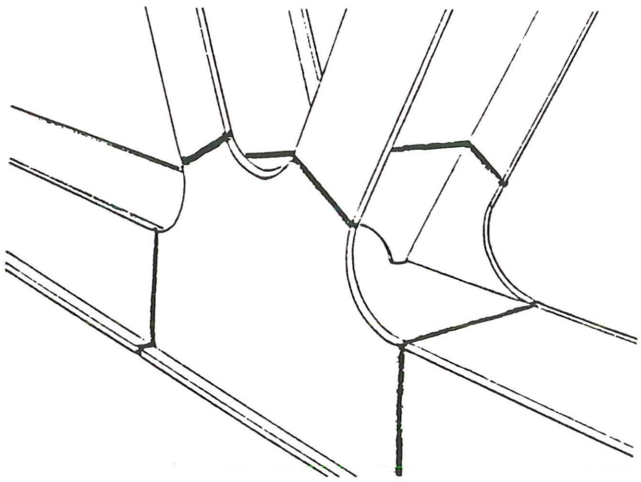


Figure 2.39 : Connection of diagonals to the chord

Sudden changes in plate thickness are avoided and the joint between chord and diagonal shows large diameter changes of geometry to improve fatigue conditions. A second important structural detail is the connection between braces and concrete deck plate.
In most designs, shear studs are used for the direct connection of the braces to the concrete deck plate. For the Nesenbach viaduct this is not the case. Here, a steel top chord with diagonal joints is embedded in the concrete deck plate, which has a locally increased thickness. Headed studs, arranged at three sides of the top chord provide for the connection between steel and concrete. See figure 2.40.

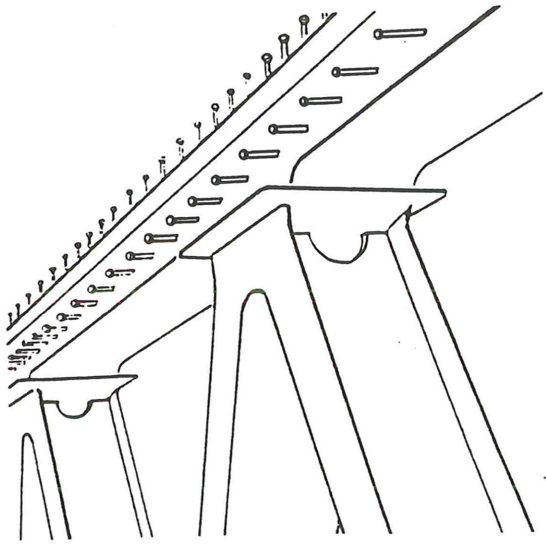


Figure 2.40 : Connection between steel and concrete

At the point of force transmission a protective plate covers the concrete. Force introduction into the concrete deck plate is facilitated by giving this plate webs near the top chords (figure 2.38).

In the manufacturing plant the steel trusses were fabricated as a whole (per span) by welding. The fabrication includes a camber compensating deflection due to dead load and live load (figure 2.41). Two truss girders per span were transported to the erection site by rail. At the erection site they were pre-mounted. In this stage the top chords were stabilised by transverse girders which were removed after completion of the composite girder. During the night the three steel bridges (without concrete deck plate) were placed in their final positions by cranes.

The concrete plate was poured in five sections. After finishing one section, the formwork was released and together with the scaffolding moved in position for the next section. The following stages can be observed:
- pre-mounting the steel structure at a place near the erection site;
- bringing the steel structure (two steel truss girders) per span to the erection site and putting it in place;
- building the scaffolding in the steel structure;
- putting formwork and reinforcement into position;
- pouring concrete in five sections.

These stages form the load cases due to dead load. Also shrinkage and creep have to be accounted for. Live load determines the final loading on the bridge. Also see section 2.2.

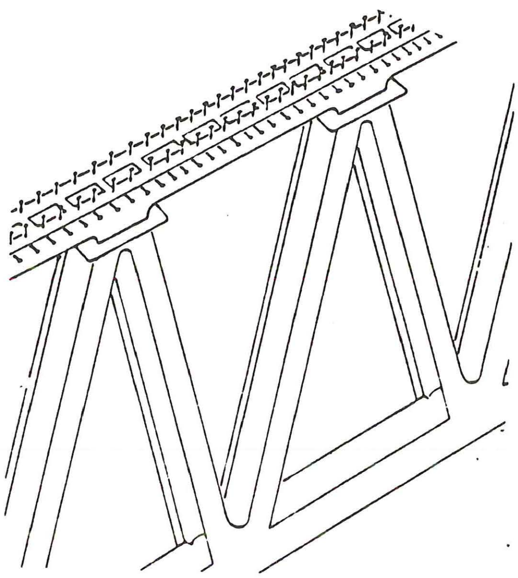


Figure 2.41 : Truss girder

2.2 DESIGN CONSIDERATIONS

2.2.1 DESIGN AND PRODUCTION

For the design of all structures the designer at least has to know one way of producing the elements and erecting the structure. For composite structures the relation between design and production is more important because the production method is affecting the mechanical behaviour of the structure (in terms of internal stresses and strains.

The character of the material gives the designer the opportunity to influence the distribution of internal stresses in the composite structure in the fabrication phase. This can be compared to designing a prestressed concrete structure. For composite structures the stress distribution has to be determined before the solidification of the concrete as well as afterwards.

It is possible to produce the composite girder in three alternative fabrication methods, gaining different distributions of internal stresses and some difference in mechanical behaviour (see figure 2.42). The composite girder can be fabricated:
- propped with a negative curvature of the steel girder;
- propped and strainless;
- unpropped.

When the steel girder is propped with a negative curvature the deformation will lead to tensile stresses at the interface steel - concrete on the upper side of the girder. After hardening of the concrete slab and removal of the supports the stresses in the steel girder will relax and the weight of the girder will be carried by composite action. The stresses in the steel and concrete section will reach an equilibrium at a certain deflection.

If the steel girder is propped over its full length the steel girder suffers no imposed external stresses during fabrication. After removal of the supports only the weight of the girder will cause a stress distribution governed by composite action.

In most circumstances the composite girder will be fabricated unpropped. The weight of the just poured concrete, that has no stiffness, is loading the steel girders. The initial stress of the steel girders is due to the total weight of the composite girder.

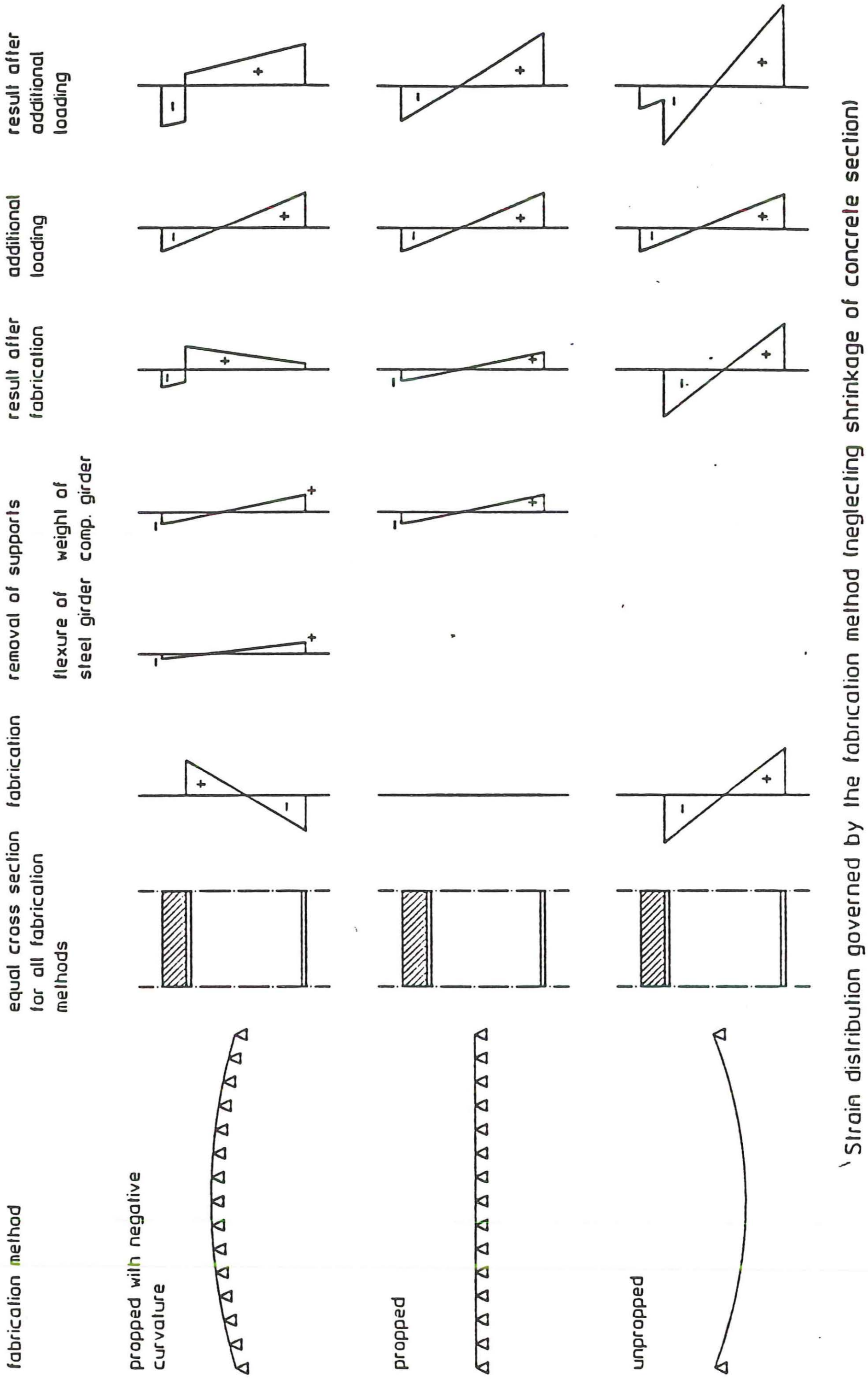


Figure 2.42 : Stress distribution

The stress distribution caused by the shrinkage of the concrete is independent of the production method. The additional stresses in the composite girder caused by shrinkage will be discussed in section 2.3.4 on composite action.

The deflection of the composite girder after production depends on the fabrication method. The designer can easily meet the limitation of the relative deflection by choosing the precamber of the steel girders in relation to the fabrication method and the total weight of the composite girder (see figure 2.43).

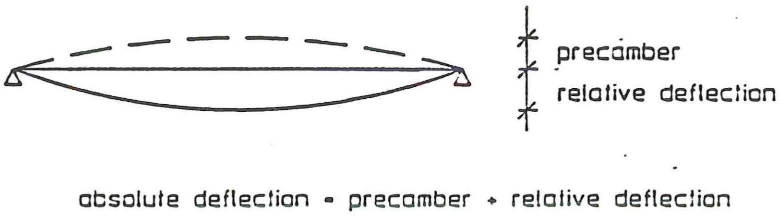


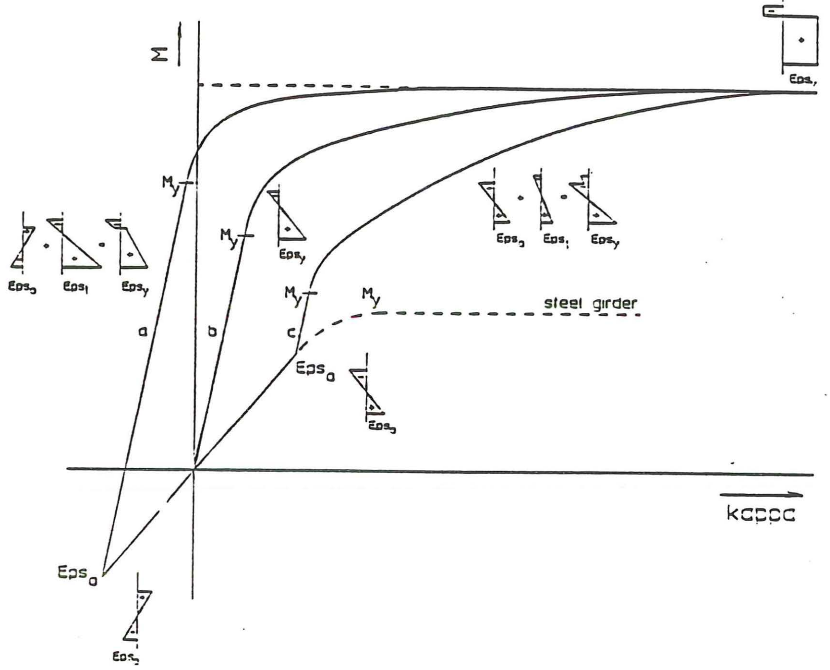
Figure 2.43 : Precamber, relative deflection and absolute deflection.

If the composite girders have the same cross section, additional load will lead to the same additional stress distribution, independent of the production method (see figure 2.42). This also applies to the stiffness and the ultimate load carrying capacity of the composite girder (see figure 2.44), provided the deformation capacity, especially of the shear studs, is sufficient.
The ultimate elastic capacity[1] varies with the production method. The steel girder with the negative curvature in the fabrication phase has an initial negative stress distribution in comparison with the stresses due to additional loads. The steel section will reach the yield stress at a larger load than the steel section of the composite girder with the same cross section but fabricated unpropped. It is a consequence the reserve capacity after reaching the yield stress is larger for the unpropped fabricated composite girder (see fig. 2.44).

In fact the designer is not free to choose his favourite fabrication method. The fabrication of the composite girder is in most cases determined by limiting conditions as:
- the length of the total bridge;
- the length of the span of a girder;
- the width of the concrete top flange of the composite girder;
- the possibilities of reaching the erection site with large elements (prefabrication or fabrication on site);
- the height under the bridge;
- the proportion of the additional loads compared to the dead load of the composite girder;
- the erection method

Although these conditions will severely influence the design of the composite girder, it is important to be aware of the possibilities to influence the internal stress distribution by varying the fabrication method.

[1] The ultimate elastic capacity agrees with the stress distribution at which the maximum stress equals the yield stress



a. propped with a negative curvature
b. propped
c. unpropped
Eps_0 : strain distribution during fabrication
Eps_l : strain distribution due to dead weight and permanent load
Eps_y : yield strain
M_y : first yield strain in the steel section

Figure 2.44 : External moment - curvature diagrams

2.2.2 STEEL SECTION

Steel girder designs depends on the fabrication method of the composite girder. The unpropped fabrication method leads to large strains in the steel section because the weight of the composite girder is carried by the steel girders alone. The initial stresses have to be added to all additional stresses.

If the steel girders are not supported during the fabrication phase large compressive stresses occur in the proportionally small top flanges. To avoid lateral instability the top flanges have to be braced temporarily during fabrication, by formwork, supports, bracings, etc.

Eventually the steel webs in the completed composite girder have to be braced to provide for significant truss action between the girders when subjected to lateral loads. The bracings also distribute unbalanced loads on the bridge, avoiding large transversal bending moments in the concrete flange.

In case buckling of the steel webs might occur, the bracings and vertical stiffeners on the webs can be integrated.

The number of shear studs and their configuration govern the dimensions of the top flanges of the steel girders. The contribution of the steel top flanges is of minor importance, as they are located close to the neutral axis of the composite section.

The concrete top flange prevents large compressive stresses in the steel section. Checking the instability of the steel section can be confined to checking web buckling.

Fatigue in the steel section is of minor importance in the composite structure. The weight of the composite girder is relatively large in comparison to steel girders. This implies that a stiffer hybrid section is required to limit the internal stresses. The increased stiffness will also reduce the stress range due to live load.

Fatigue problems are often present in a steel structure, directly under an attached rail. Every wheel load of a passing train may produce a stress alteration, resulting in an enormous quantity of cycles during lifetime.

The concrete slab acts as an intermediary between the wheel loads and the steel structure, reducing the stress range in the steel section. The diagrams for fatigue show quite steep lines for the relationship between a certain stress interval and the number of cycles (S-N lines). This implies that even a small stress reduction will cause improvement of the fatigue problems because of the large amount of stress intervals. Fatigue of the concrete slab is not considered.

2.2.3 CONCRETE SLAB

The material properties of concrete vary with time due to shrinkage and creep. This implies that long term and short term material properties are formulated for the behaviour of concrete under loading.

The mechanical behaviour of concrete for short term loads is much better than for long term loads. It is possible to neglect the better material properties for short term behaviour by only taking the long term properties into account. If the advantage of short term behaviour is taken into account it is necessary to divide all loads in long term and short term loads.

Long term loads:
- the weight of a composite girder if the girder is fabricated using the propped method;
- the strain distribution in the composite girder due to the propped fabrication method with negative curvature of the steel girders;
- shrinkage of concrete (see section 2.3.4);
- permanent loads on the composite bridge.

The changing material properties of concrete due to long term loads will have mechanical consequences for the composite girder. This issue will be discussed in section 2.2.4 on composite action.

Short term loads:
- a difference in temperature of the steel and concrete section;
- live loads.

The elastic behaviour of the composite girder for short term loads will be discussed in section 2.2.4 on composite action.

If the composite girder is statically determinate, the concrete flange will be loaded by compressive stresses in span direction. Transverse to the span direction tensile stresses will occur. The top flange reduces to a concrete structure which is supported by the steel girders. All concrete techniques as reinforcing, prefabrication and prestressing are available.

Despite recent research and publications on the behaviour of concrete under live load, the fatigue of concrete is not considered here.

2.2.4 COMPOSITE ACTION

Both reinforced (or prestressed) concrete and composite structures are hybrid structures with tensile stresses in the steel and compressive stresses in the concrete. The steel section in the composite girder has a large second moment of inertia whereas in the concrete structure it is negligible. This implies that the composite structure is internally statically indeterminate (see figure 2.45).

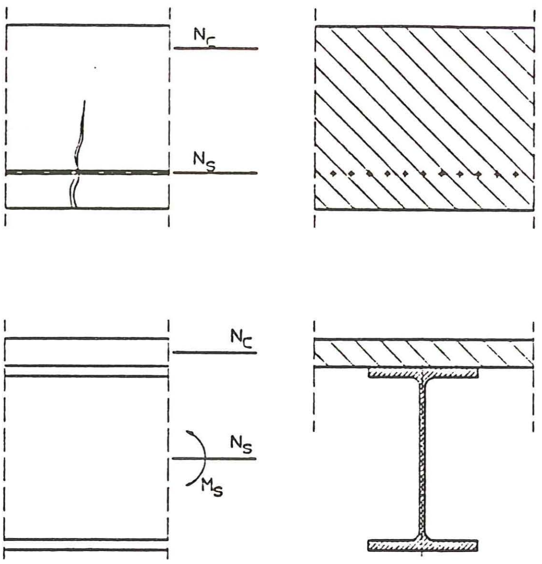


Figure 2.45 : Comparison of a concrete and composite section.

Because of the relatively high bending stiffness of the steel section in composite structures, the changing of the mechanical properties of the concrete section has more consequences than for concrete structures.

The consequence of shrinkage of the concrete in a composite girder is comparable to a difference in temperature of the steel and concrete section. The difference in strains of the sections cause a displacement in the interface steel - concrete that is prevented by the shear studs. This leads to internal stresses in the composite section. Both effects must be taken into account as additional load cases. [2.57].

The initial stress distribution in the composite girder directly after loading will redistribute if the load is permanent. The creep of concrete causes stress relaxation in the concrete section and increasing stresses for the steel section. This also results in an additional deflection of the composite girder, which can be reduced by cambering of the steel girder.

The eventual stress distribution in the composite girder after creep of the concrete can be computed by taking the long term material properties into account (see section 2.4.5).

Short term loads give the concrete no time to creep. The elimination of the creep leads to a stiffer, elastic material behaviour. The absolute deflection of the girder by live loads is elastic and has to meet the demands of the serviceability limit state. This gives the opportunity to use the advantage to take the short term material properties of the concrete into account.

2.3 DESIGN EXAMPLE

2.3.1 PREMISES

With the intention to give the design of a composite bridge a certain realism an existing situation is chosen to take the premises from. The third approach span of the new bridge over the Oude Maas near Dordrecht was suitable for this goal. This choice also held the opportunity to compare a composite bridge to a concrete alternative because the concrete design had to be made in the same period.

The bridge across the Oude Maas consists of two parallel railway tracks, a four meters centre to centre. On the approach spans the tracks are ballasted. The third approach span is above water, 35 m long and 12 m wide. No restrictions were imposed on the clearance underneath the composite bridge or on the construction depth.

For the comparison of the concrete design and the composite alternative the same characteristic live loads, dynamic factors, load factors and safety factors have to be used. The safety factors differ for the materials. The stress distribution is computed for the characteristic loads. The factors are taken into account afterwards, depending on the material of the section to check.

2.3.2 DESIGN REGULATIONS

The computation of the composite bridge design is based on the first draft of the 'Regulations for designing steel bridges VOSB 1988' [2.57].

The live load on the composite bridge is derived from 'Directions for the designing of steel bridges VOSB 1963' [2.58].

For the material properties and behaviour of the concrete section the 'Regulations for concrete 1974/1984' [2.59] has been used.

2.3.3 INITIAL CHOICES

The approach span can be designed as one double track bridge or two parallel single track bridges (see figure 2.46). The two single track bridges are rejected because the concrete flange is eccentric with regard to the steel webs underneath the two rails. The bridge will be unbalanced and might overturn in case of a derailment.

The live loads of trains passing the bridge are of great influence on the design. The shear forces are directly transmitted to the steel webs by situating the webs under every rail. It is necessary to brace the two inner webs to avoid large internal stresses in the centre of the concrete slab during unbalanced loading by one train.

The design with four webs is rejected because of two reasons. Firstly, the welding costs will reduce using only two webs. Secondly, the webs do not add much to the stiffness of the girder. This is more important than the shear strength because answering the demand for the absolute deflection by live load in the serviceability limit states usually controls the design.

The design with two webs is preferred above the design with four webs. The webs will be centred under each railway track. Because of maintenance considerations a box girder is applied. This also has advantage from the viewpoint of noise emission, box sections produce less noise than two I - shaped girder. The result is a composite box girder with a steel bottom flange and webs and a slightly tapered concrete deck.

Considering the approach span is over water and the dimensions of the composite bridge, the envisioned erection procedure is the following:

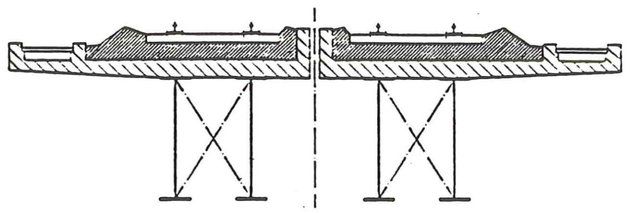
- fabrication of the entire steel section off site;
- transportation of the completed steel girder over water;
- erection by floating crane;
- pouring the concrete deck slab in situ.

Pouring the concrete deck after placing the steel girder in the definite position implies an unpropped fabrication. The initial strains in the steel section due to this fabrication method have to be minimized by reducing the thickness of the concrete slab.
The sequence that has led to the design of the composite bridge is:

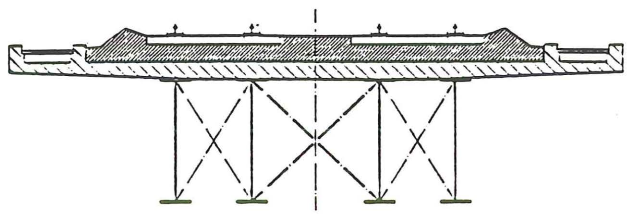
- an inventory of all loadings, additional factors, demands for ultimate limit state and serviceability limit state;
- design of the composite girder in span direction after estimation of section dimensions;
- detailed design of the steel structure;
- detailed design of the concrete slab;
- detailed design of the shear connection in the interface steel - concrete.

The results of every step in the designing process has interactive consequences for other steps.

The designing process will be discussed in the sequence as stated above. The discussion will confine to principles, results and comparison in order not to loose the general overview by presenting calculations as well.



Two parallel single track bridges



One double track bridge with webs centred under every rail

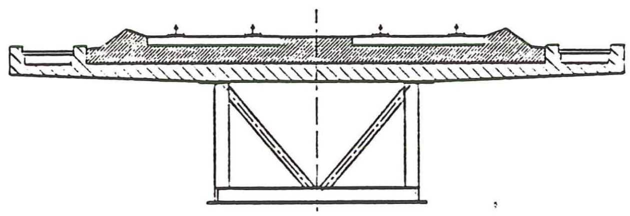


Figure 2.46 : Initial design alternatives

2.3.4 DESIGN VARIABLES

In the serviceability limit state the bridge design has to answer the severe demand of an absolute deflection due to live load of less than or equal to 1/1000 of the span length [2.57]. To answer this demand is often decisive for a steel bridge design. This implies that all loads have to be divided in long term and short term loads.

The long term loads are:
- the ballast bed on the composite bridge;
- the concrete applies on the bridge poured after hardening of the concrete slab, the electric cable conduit, etc.;
- shrinkage of the concrete flange after hardening and reaching composite action;
- the unloading by removal of the formwork.

The short term loads are:
- the live loads by passing trains;
- a difference in temperature of the steel and concrete section.

  The weight of the composite bridge fails in the above survey of loads. In the fabrication phase the total weight of the composite girder and the formwork is carried by the steel girder. The initial stresses in the steel section caused in the fabrication phase have to be added to all additional stress distributions (see section 2.4.5).

Live loads have to be multiplied by a dynamic factor to allow a quasi static calculation. The limiting conditions that govern the dynamic factors of steel and concrete are of such proportions that the factors for both materials are equal. This allows to design with only one live load multiplied by one dynamic factor.

2.3.5 DESIGN OF THE COMPOSITE GIRDER

The thickness of the concrete slab has to be minimized to reduce the loading on the steel section during fabrication. A high grade concrete can be used for this purpose. But the slab has to be poured in situ. To avoid the necessity of taking special measures to reach the high material properties of the high grade concrete, the grade has been limited to a B35.

The steel section will mainly be loaded by tensile stresses in the composite structure because of the large concrete top flange. The compressive stress in the steel section are small. The chance that instability might occur is reduced. In this circumstances it is logical to choose a steel grade Fe E 355.

Headed studs will be used as shear connectors. The regulations VOSB 1988 [2.57] presume that no significant end slip in the interface steel - concrete will occur. No reasons are foreseen to reject this assumption, so full composite action of the girder is taken into account.

Preliminary calculations for initial section dimensions converted to a composite bridge design. The calculations have to be split in three parts (see figure 2.47):

- *the fabrication phase*
  A simple steel structure is loaded by the just poured concrete. The strains in the steel section due to the total weight of the composite girder can easily be calculated.
- *the long term loads*
  Full composite action is reached. The neutral axis of the hybrid structure has to be determined taking Young's modulus of concrete for long term loads into account. All section properties have to be determined in relation to the appropriate neutral axis. The stress distribution and absolute deflection can be calculated. The shrinkage of the concrete is a special load case, calculated apart from the permanent loads.
- *the short term loads*
  The neutral axis of the hybrid section is closer to the concrete slab because of the higher Young's modulus of the concrete for short term loads. The stress distribution and absolute deflection can be calculated after all section properties have been determined.

  The eventual stress distribution in the composite girder is the superposition of the former calculated partial stress distributions (see figure 2.47).

  The stress distribution in the ultimate limit state has to answer the material stress limitations.

  The absolute live load deflection in the serviceability limit state has to answer the limitation of 1/1000 of the span length. The relative deflection by permanent loads can be reduced by designing the precamber of the steel girder in relation to the absolute deflection (see figure 2.43).

2.3.6 DETAILED DESIGN OF THE STEEL STRUCTURE

The bottom flange of the girder is more than four meters wide. Despite the large width the entire flange is found to be effective at midspan according to the regulations VOSB 1988 [2.57].

The design of the steel section is controlled by minimizing fabrication costs. This goal can be achieved by reducing the cost of labour that is the main part of the overall costs. This view for instance results in the following measures:
- minimize welding length;
- minimize welding details that need additional preparation of the steel plates before welding;
- make no use of horizontal web stiffeners to avoid complicated details, crossing the vertical plate stiffeners;
- avoid labour intensive details in general.

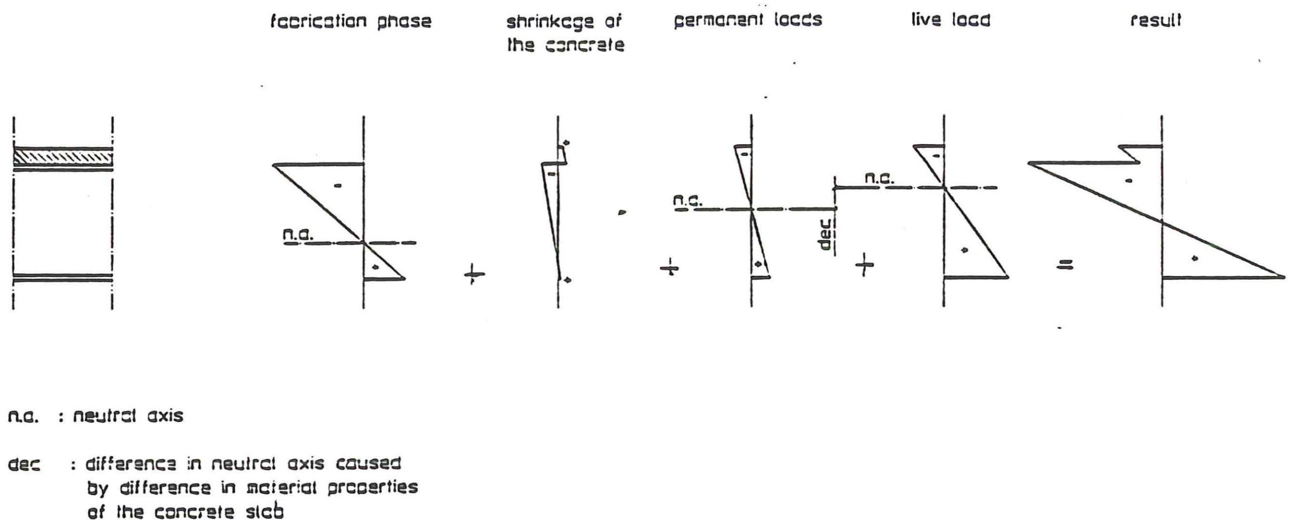


Figure 2.47 : Stress distribution in the resulting composite bridge.

In the steel girder K-bracings have been placed every five meters (see figure 2.48). The bracings provide for truss action, lateral stability of the top flanges during unpropped fabrication and are integrated with the vertical web stiffeners (see section 2.3.2).

In this particular design the K-bracings also provide stability for the steel girder during welding, transportation and lifting. The wide bottom flange is also supported by the bracings every five meters. To reduce costs the bracings are made out of rolled I - beams.

On both ends of the composite girder the tube is closed by steel plates improving the maintenance conditions of the inside (see figure 2.48). The ends transmit the resulting lateral loads by wind on the bridge and horizontal live load on the top flange to the bearings. The vertical support reaction will be distributed by the end plates into the webs of the girder.

Beyond the improvement for fatigue due to the design of a composite bridge in general, no fatigue problems for this particular design are foreseen. The approach span is ballasted increasing the permanent load the composite girder. The live load is relatively decreasing compared to the permanent loads, causing smaller stress ranges in the steel section (see section 2.3.2). In parts of the steel section with relative large stress ranges due to live loads (bottom flange) fatigue sensitive details are avoided.

2.3.7 DETAILED DESIGN OF THE CONCRETE SLAB

The composite bridge has an extraordinary wide concrete top flange. Despite the width nearly the entire flange is found to be effective at mid span [2.59].

The composite bridge design is statically determinate, with compression in the concrete slab in the longitudinal direction. The compressive stresses due to composite action are of relatively low level. Only some minor longitudinal reinforcement will be used to distribute stresses of the transversal reinforcement and to control possible cracks due to shrinkage.

Two important variables for the transversal reinforcement design of the concrete flange have to be determined. Firstly, the crack width of the longitudinal cracks have to be minimized to ensure that the concrete slab is impermeable for water. Secondly, a minimum concrete cover of 35 mm is required to prevent corrosion of the steel reinforcement. The thickness of the concrete cover has the disadvantage of reducing the internal lever arm (reinforcement to compressive stress resultant) in the concrete section.

The transverse direction is of major importance for the slab reinforcement design. The large transversal span of the slab supported by webs centred under the tracks cause large bending moments and shear forces due to live load and weight of the ballast bed. The large amount of reinforcement is governed by the small slab thickness and the severe limitation of crack width. This results in a very tight reinforcement design with large bars on very small centres.

Even more reinforcement has to be added because the concrete slab has no shear strength for live loads according to the concrete regulations VB 1974 - 1984 [2.59]. Shear reinforcement has to be added to take over the shear forces of the concrete. Except the above mentioned fatigue of the slab due to shear forces caused by live load, fatigue of the concrete flange is not considered.

As a result of the above mentioned conditions and limitations the reinforcement ratio of the concrete slab is very high.

Transverse prestressing of the slab has been taken into consideration. The steel cables can not be curved because of the small thickness of the slab. This implies that only eccentric prestressing is an alternative for the reinforcement. The tensile stresses in the concrete slab are of such level that the prestressing force has to be enormous to cause only transverse compressive stresses. This alternative has to be rejected for this particular design. At this stage of the design, a disadvantage of the chosen erection procedure comes up. Pouring the wide outer parts of the concrete slab requires temporary additional steel construction to provide for installing the formwork.

2.3.8 DETAILED DESIGN OF THE SHEAR CONNECTION

According to recent publications much attention is paid to fatigue of the shear connectors under live loads. This literature has not been used for the design of the shear connection. The minor instruction of the regulations VOSB 1988 [2.57] accounting for fatigue of headed studs were used.

The calculated number of headed studs according to the regulations have to be interpreted as a minimum. The eventual quantity is governed by a practical configuration in relation to the reinforcement grid of the concrete slab. Transverse reinforcement bars are crossing the steel top flange and the headed studs.

2.3.9 CONCLUSIONS

The composite bridge design, which is the result of the discussed designing process, is shown in the figures 2.48 and 2.49. A few conclusions can be derived from the figures.

The construction depth of the bridge, is large in comparison to concrete troughs and steel truss bridges with a track on bottom chord level. The consequences of this particular disadvantage of composite bridges in general will be discussed in section 2.4.

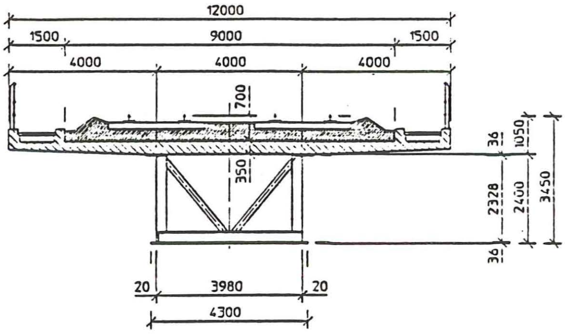
The concrete top flange is very wide. The transverse direction is controlling the slab design resulting in a tight reinforcement net. Longitudinal stresses due to composite action are small compared to the transverse stresses. This implies that the concrete slab is suitable for much larger spans without any adaption, as the transverse conditions are independent of the span of the bridge. It is obvious that the steel section has to be increased to reach the larger span. The design is very simple, without additional labour or complicated details. Longitudinal plate stiffeners are avoided by using thicker plates for the webs. The steel section is composed of the large produced plates without unnecessary adaption to the varying bending moment in longitudinal direction. Increasing material costs due to increasing steel weight is exceeded by the reduction of labour costs due to decreasing welding length, leading to decreasing overall costs.

Figure 2.47 shows that the initial stresses due to the unpropped fabrication method are large compared to all other additional stress. The initial stress can be avoided by using other fabrication methods. The reduction of the steel section due to reduction of these stresses is limited by the severe stiffness demand in the serviceability limit state for the absolute deflection by live load. This implies that the reduction of the initial stresses does not offer the great advantage, to be expected according to the stress distribution figures.

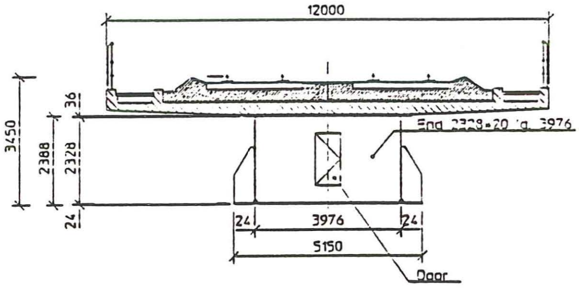
Composite bridge sections have a more complicated mechanical behaviour than a single steel or concrete section. The mechanical response of the composite girder to variation of the steel and/or concrete section is less predictable. This is mainly caused by the fact that the hybrid structure has more degrees of freedom. The mechanical response is governed by two materials instead of only one. In addition the behaviour of the concrete depends on the nature of the loads.

A disadvantage of the chosen erection procedure is the additional steel construction that has to support the formwork and just poured concrete of the large outer parts of the concrete flanges. Two alternatives can improve this:

- Fabricate the complete composite girder off site. On both sides of the steel box girder scaffolding can easily be erected to support the formwork. The finished composite girder can be transported and floated into position.
- Prefabricate the concrete slab in sections of about three meters. The prefabricated sections can easily be transported to the erection site and placed on top of the steel box girder using a light crane. This alternative requires a review of the shear connection and concrete slab design. The configuration of the reinforcement net and headed studs have to be adapted to the alternative erection procedure.

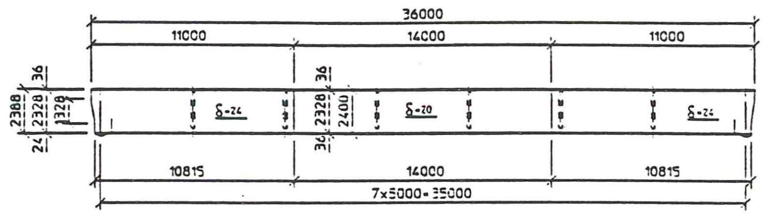


Section with K-bracing

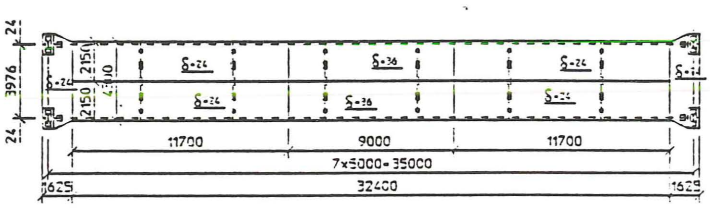


End section

Figure 2.48 : Cross section of the eventual design



Steel webs



Steel bottom flange

Figure 2.49 : Longitudinal section of the eventual design

2.4 CONCLUDING REMARKS

Regarding the developments in the fabrication of bridges in the surrounding countries, composite bridges seem to have a promising future. In the last few decades composite bridges have become more competitive compared with steel and concrete bridges. An inventory of the literature regarding composite bridges and an actual composite design alternative for a concrete approach span have led to the following concluding remarks.

The construction depth of the bridge, is large in comparison with concrete troughs and steel truss bridges with the track or deck on the bottom chord level. This is a general disadvantage for application of composite bridges in the Netherlands. This disadvantage applies to steel bridges with orthotropic decks as well.

The clearance underneath the bridge has almost always to answer severe demands for road traffic or water transport. The construction depth as mentioned above has to be limited to reduce the length of the approach tracks leading to the bridge. This is more disadvantageous for railway bridges than for road bridges because of the slight slope of 1:200 of the approaching railway tracks.

This disadvantage may be important for the Dutch environment, but is less important for surrounding countries with hilly or mountainous grounds. If the bridge spans a valley, going from one hill to another, the total span is generally large without any limitation of the construction depth.

The above mentioned disadvantage of the large construction depth does not apply to some composite bridge types, suitable for smaller spans only. Composite designs with parallel embedded rolled beam sections or the preflex system can span up to 20 meters with a small construction depth. The large deflection under live load of these bridge types often restricts the applicability for railway bridges.

In literature several examples have been found of composite bridges with prestressed concrete slab. Prestressing by steel cables can be applied both in transversal and longitudinal direction. Prestressing by lowering intermediate supports or removing temporarily supports will only lead to longitudinal compressive strains in the concrete slab.

An outstanding feature of the composite bridge is the strong relationship between design and production of the bridge. Altering the fabrication method has consequences for the internal stress distribution, leading to difference in mechanical behaviour.

The composite bridge has been found to be advantageous compared to concrete or steel bridges for several issues as illustrated in the following paragraphs.

When compared to a concrete bridge, the design of a composite alternative will lead to the following interesting advantages:
- the steel girder can be prefabricated under controlled conditions in the shop and transported to the erection site;
- the steel girder can carry the total weight of the composite bridge during erection without any intermediate supports;
- the steel girder allows erection procedures which are not familiar to concrete bridges and offers the opportunity of choosing the most economical erection procedure;
- the weight saving may lead to less investments in piers, abutments and foundation of the bridge;

- the period between tendering and delivery of the bridge may reduce due to both reduction of fabrication and erection time.

In comparison with a steel bridge the composite bridge shows the following advantages:
- the concrete plate deck is easy to build in comparison with an orthotropic steel plate deck; and about a factor 2 cheaper.
- less noise emission of the bridge due to the increased mass directly underneath the traffic, especially advantageous for railway bridges;
- very suitable for ballasted railway bridges.

Fatigue problems will reduce in the steel part of the composite bridge compared to an overall steel bridge for two reasons.
Firstly, the increasing weight of the bridge requires a larger stiffness, leading to smaller stress ranges due to the relatively decreased live loads.
Secondly, directly under the attached rails the number of stress intervals will be enormous; every passing wheel load counts. These stress alterations will be spread by the concrete slab and reach the steel structure strongly reduced.

The knowledge about fatigue of the shear connection and concrete structures is limited. Much attention is paid to this problem lately. It is too premature to conclude that no overall fatigue problem will occur. Research efforts have to answer this question.

As a final remark it can be stated that a composite bridge is a hybrid structure that combines the advantages of two well known construction materials in an elegant and simple concept.

3. CALCULATIONS

3.1 MECHANICAL PRINCIPLES OF COMBINED ACTION

When two materials are combined in one structural element in such a way that no slip occurs between the two materials, optimal use can be made of the advantages of the different mechanical properties of the materials. In order to make it possible to calculate the force distribution in and the load capacity of the composite structural element the basic elastic mechanical principals of the combined action of the two materials are explained in this paragraph.
It is assumed that the stress-strain relationship remains linear, whatever the stress level and the tensile strength of concrete may be neglected.

3.1.1 AXIAL FORCES

When a cross section of a composite steel concrete member (figure 3.1) is considered and an axial force is acting in the centre of gravity of this member the structural stiffness and the stress distribution in the member can be calculated in the following way:

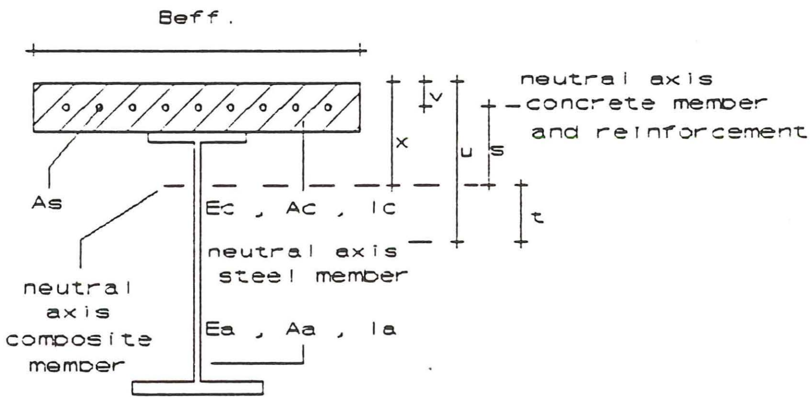


Figure 3.1

steel properties : E_a, A_a
concrete properties : E_c, A_c

$$\sigma = E * \epsilon \rightarrow \sigma_a = \frac{N_a}{A_a} = E_a * \epsilon_a$$

$$\rightarrow \sigma_c = \frac{N_c}{A_c} = E_c * \epsilon_c$$

no slip between steel and concrete : $\epsilon_a = \epsilon_c = \epsilon$

the total force : $N = N_a + N_c$

introducing : $n = \frac{E_a}{E_c}$ (3.1)

$$\rightarrow N = E_a * \left[A_a + \frac{A_c}{n} \right] * \epsilon \tag{3.2}$$

Formula (3.1) indicates the ratio between the modulus of elasticity of steel and concrete. This ratio is often used in calculations of composite structures because E_c can have different values for different load cases. Formula (3.2) indicates that the strain stiffness EA, of a composite member can be calculated as the modulus of elasticity of steel multiplied by the area of the composite member in which the area of the concrete is replaced with an equivalent steel area. This is done by dividing the concrete area by the factor n.

3.1.2 BENDING MOMENTS

When a cross section of a composite steel concrete member (figure 3.1) is considered and a bending moment is acting on this member, the centre of gravity and the moment of inertia can be calculated in the following way:

centre of gravity: $\times = \frac{\Sigma z_i * A_i * E_i}{\Sigma A_i * E_i}$

In the case of structural elements used for steel-concrete composite bridges the centre of gravity usually lies in the steel part of the composite member. This means that in sagging moment regions the concrete part of the member is completely in compression where in hogging moment regions the concrete part is completely in tension.

In sagging moment regions the influence of the reinforcement is neglected and the factor n is used again :

$$\times = \frac{u * A_a * E_a + v * A_c E_c}{A_a * E_c + A_c * E_c} = \frac{A_a * u + \frac{A_c}{n} * v}{a_c + \frac{A_c}{n}} \tag{3.3}$$

The bending stiffness EI_1 of the composite member can now be calculated:

$$E * I = E_a * I_a + E_a * A_a * t^2 + E_c * I_c + E_c * A_c * s^2$$

$$I = I_a + A_a * t^2 + \frac{I_c}{n} + \frac{A_c}{n} * s^2 \tag{3.4}$$

Formula (3.4) shows that the bending stiffness can be calculated in the same manner as the strain stiffness by dividing the concrete area by the factor n. With the resulting equivalent steel area the second moment of inertia can be calculated using normal procedures.

In hogging moment (figure 3.2) regions the concrete is in tension and may be neglected in the calculation. In this case the centre of gravity x and the second moment of inertia I_2 can be calculated using normal procedures taking into account the area of the steel member and the area of the reinforcement.

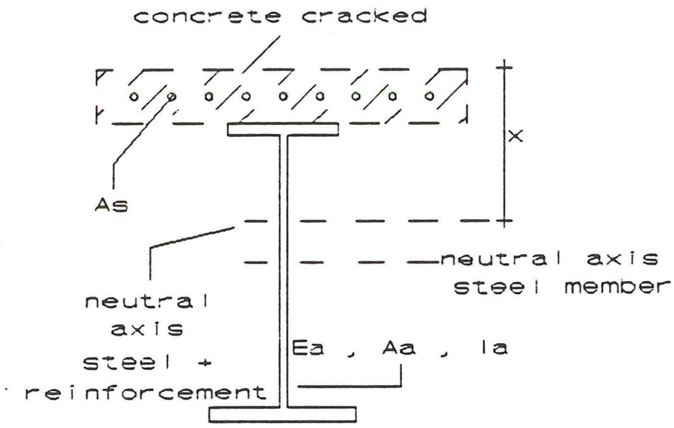


Figure 3.2

3.1.3 SHEAR FORCE

The load carrying capacity for vertical shear forces of the concrete deck of a composite steel concrete bridge will be neglected. Thus the vertical shear forces are to be taken by the steel member only.

3.2 CALCULATION OF COMPOSITE STRUCTURES VERSUS COMPOSITE BRIDGES

The latest draft of the Eurocode 4 presents rules for composite steel concrete structures by means of general rules and rules for buildings. The scope of the code is therefore different from the subject of this report. Certain subjects required for the calculation of composite bridges are not discussed and some parts of the code discuss rules which may not be used for the calculation of composite bridges.

In the Eurocode a classification of cross sections is made. There are 4 classes which can be defined as indicated in Table 3.1.

Table 3.1 : Cross section classification.

Class	Definition of the cross section
1	Cross sections which can form a plastic hinge with rotation capacity required for plastic analysis.
2	Cross sections which can develop there plastic moment resistance, but have limited rotation capacity.
3	Cross sections in which calculated stresses in the extreme compression fibre of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
4	Cross sections in which it is necessary to make explicit allowance for the effect of local buckling when determining their moment resistance of compression resistance.

The classification of the cross section is mainly based on the dimensions and the stability of steel parts (flange and web) of the composite member under compression.

The calculation of the action effects for all four classes can be carried out by means of elastic analysis. Only for class 1 cross sections a plastic hinge method may be used.

The resistance of a cross section classes 1 and 2 can be calculated with the plastic moment method. The resistance of a cross section class 3 is limited to the ultimate stresses of the used materials and the resistance of class 4 cross sections is also limited to the ultimate stresses of the used materials taking account of local buckling of the flange and the web.

Generally, the cross sections of plate or box girders of composite bridges are class 3 or 4 [3.4]. This indicates that the global calculation of the bridge composite system must be done by elastic analysis with redistribution of the bending moments (if appropriate). The resistance of the cross sections is limited to the ultimate stresses of the materials used taking account of local buckling of the steel parts of the cross section.

The action effects to be considered for composite bridges taking account of the general classification of their cross section are :

1. *ultimate limit state:*
   - weight of structural elements
   - erection method
   - weight of non-structural elements
   - traffic loads
   - wind loads
   - shrinkage and creep of concrete
   - temperature

2. *serviceability limit state:*
   - concrete cracking
   - deformations

3.3 EFFECTIVE WIDTH OF THE CONCRETE SLAB

In the calculation of composite structural elements the effective width of the concrete flange should be taken into account. The effective width is smaller than the total width due to the shear lag effect.

Shear lag is caused by in-plane shear force deformations of the concrete slab which is acting as a flange for the composite member. Because of this deformation the stresses near a web of a composite member are higher than the stresses far from a web. By using the effective width of the slab the calculated stress equals the real maximum stresses. This is explained in figure 3.3.

In [3.2] the effective width is specified for beams in buildings. The total effective width b_{eff} of a concrete flange is defined as the sum of effective widths b_e of the portions of the flange on each side of the centre line of the steel web. The effective width of each portion should be taken as $b_e = L_0 / 8$. For simply supported composite members, L_0 is the length of the span, for continuous members. L_0 is given in 3.3. The effective width of each portion should not exceed half the distance to the adjacent web.

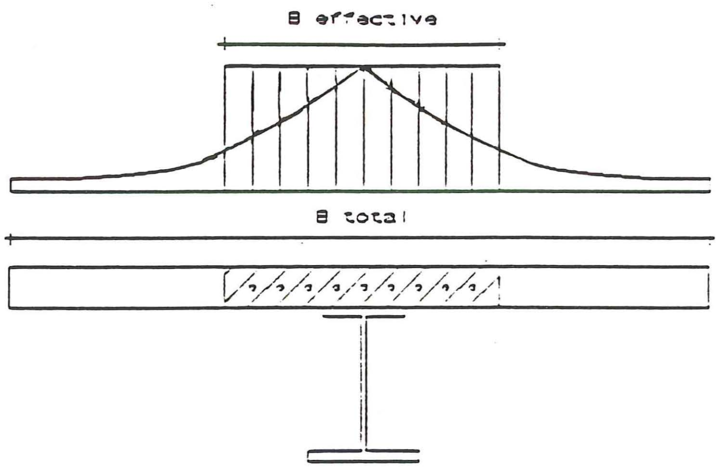


Figure 3.3 : Effective width

Because, in the Eurocode, the effective widths are specified for buildings, the values are compared with those of the draft code of the Netherlands and the British code. The comparison has been made for various spans of a bridge type outlined in Table 3.2

Table 3.2 shows the results for the mid span of simply supported bridges and for the mid-span and the internal supports of continuous bridges.The results show that the Eurocode in relation with the British code is somewhat conservative for the mid span locations of simply supported and continuous bridges with small spans and somewhat progressive for larger spans. For internal supports of continuous bridges the Eurocode is conservative for all spans. The Dutch code is only valid for simply supported bridges and is a little conservative in relation with the Eurocode.

Table 3.2 : Comparison for various spans

Span (m)	Eurocode			British code			Dutch code
	1	2	3	1	2	3	1
10	2.5	1.75	1.25	3.55	1.98	0.89	2.0
15	3.75	2.62	1.87	4.62	3.22	1.35	3.0
20	5.0	3.5	2.5	5.18	4.00	1.74	4.0
25	5.62	4.37	3.12	5.47	4.50	2.1	5.0
30	6.25	5.12	3.75	5.64	4.97	2.42	5.5
35	6.5	5.56	4.37	5.78	5.27	2.65	6.0
40	6.5	6.0	5.0	5.87	5.44	2.8	6.5
45	6.5	6.44	5.31	5.92	5.53	2.97	6.5
50	6.5	6.50	5.62	5.93	5.62	3.14	6.5

1) mid span effective width of a simply supported bridge
2) mid span effective width of a internal span of a continuous bridge
3) effective width of a internal support of a continuous bridge

3.4 GLOBAL ANALYSIS OF COMPOSITE BRIDGES

The global analysis of the action effects on the structural system of a composite bridge is carried out in order to determine the distribution of the bending moments and vertical shear forces along the structural elements.

When the steel sub-structure is unpropped during the erection phase, separate analysis must be made to calculate the effect of the permanent actions applied to the steel members only. These effects must be determined by elastic analysis of the structural steel subsystem using the properties of the steel members only. The resulting stresses should be added to the stresses resulting from the analysis of the composite action of the system.

The effects of actions working on the composite system must also be determined by elastic analysis. In this analysis the properties of the composite members can be calculated using the method of the equivalent steel section, taking account of shear lag by means of an effective width for the concrete slab and taking account of the ratio of the modulus of elasticity of steel and concrete.

For the global analysis of the bridge system a constant effective width may be assumed over the total length of each span. This value may be taken as the value at mid span.

The ratio of the modulus of elasticity of steel and concrete depends on the kind of loading on the bridge (short time loading or permanent loading including creep of concrete) and on the material grade of the concrete.

The global elastic analysis can be carried out with two methods (see fig. 3.4).

In method 1 the flexural stiffness of the cross sections may be calculated assuming the concrete to be uncracked over the total length of the bridge.

In method 2 the flexural stiffness of the sections may be calculated assuming the concrete to be cracked over a length of 15% of the span on each side of an internal support.

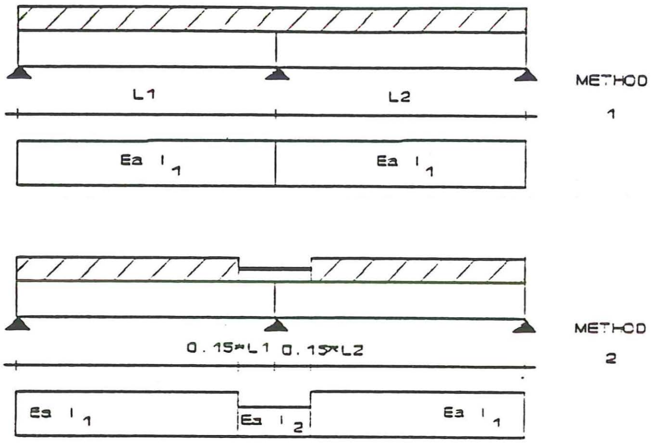


Figure 3.4 : Global analysis

The bending moment distribution given by the elastic global analysis may be redistributed in a way that satisfies equilibrium and takes account of the effects of cracking of concrete, inelastic behaviour of materials and local buckling of structural steelwork. The maximum percentages for redistribution of hogging moments are given in table 3.3.

Table 3.3 : Redistribution of bending moments

Cross section class	3	4
Method 1	20	10
Method 2	10	0

Using the flexural stiffness of the cross sections and the way of analysis explained in this paragraph, the distribution of bending moments and vertical shear forces can be calculated.

For class 4 cross sections the effect of shrinkage, modified by creep, also has to be considered. For simply supported bridges shrinkage of the concrete results in extra deflections of the bridge, a change of the stress distribution in the cross sections and additional forces on the shear connectors.

The free shrinkage strain ϵ_r to be considered for bridges (construction in open air) is approximately 200 * 10^{-6}.

Shrinkage leads to internal normal forces and bending moments on the steel and the concrete part of the composite construction (see figure).

$$N = \frac{E_a * \epsilon_r}{\frac{n}{A_b} + \frac{1}{A_a} + \frac{n * d_b^2}{n * I_a + I_b}} \tag{3.5}$$

These forces working on the neutral axis of the steel section and the concrete flange result in a total bending moment on the composite cross section : $M = N * d_b$

This moment can be divided in a moment on the steel section and a moment on the concrete flange :

$$M_c = \frac{d_b * I_b}{n * I_a + I_b} * N$$

$$M_a = d_b * \frac{n * I_a}{N * I_a + I_b} * N \tag{3.6, 3.7}$$

In continuous bridges a simplified method can be adopted by using the same procedure as specified for simply supported bridges. Each individual span may in this case be considered as a simply supported span. The rotation at the supports of each span can be calculated with the method presented above and the following formulae :

$$\Theta = \frac{M * l}{2 * E_a * I_a} \tag{3.8}$$

The continuity of the system can be reached by adding additional hogging moments which close the rotation gaps. According to the effects of cracking of the concrete should be taken into account when tension stresses in the concrete are higher then $0.15 * f'_{ck}$. The resulting additional moments should be multiplied in this case by a factor 0.6. This method for analysing the shrinkage effects may only be used when the concrete is not prestressed at any way. Furthermore the ratio:

$$\frac{E_b * I_b}{E_b * I_b} \leq 0.2 \tag{3.9}$$

The effects of a difference of temperature between the concrete flange and the steel sub-construction can be calculated in the same manner as described above. In this case the shrinkage strain ϵ_r has to be replaced by the strain resulting from the difference in temperature and n is the ratio for short term loading.

3.5 ANALYSIS OF CROSS SECTIONS OF COMPOSITE BRIDGES

The internal forces resulting from the global analysis of the composite bridge system should be transferred to stresses on the separate materials.

The stresses due to bending moments in composite action in sagging moment regions are:

$$\sigma_{b1} = \frac{M * \times}{N * I_a} \quad N/mm^2 \tag{3.10}$$

$$\sigma_{b2} = \frac{M * (\times - h_b)}{n * I_1} \quad N/mm^2 \tag{3.11}$$

$$\sigma_{s1} = \frac{M * (\times - h_b)}{I_1} \quad N/mm^2 \tag{3.12}$$

$$\sigma_{s2} = \frac{M * (h - \times)}{I_1} \quad N/mm^2 \tag{3.13}$$

The value of n in the equations above depends on the type of loading which is considered. For long term loading the n value should be used in which creep is discounted (see paragraph 2). For the calculation of I_1 the actual effective width according to paragraph 5 should be used. The resulting stresses of effects of shrinkage modified by creep and the effects of temperature can be calculated from the moments and normal forces derived from paragraph 6. If the composite bridge is unpropped during construction, the stresses in the steel sub-construction have to be added to the stresses due to composite actions.

The stresses due to bending moments in hogging moment regions, where the concrete is supposed to be cracked, are:

$$\sigma_r = \frac{M * (\times - h_r)}{I_r} \quad N/mm^2 \tag{3.14}$$

$$\sigma_{s1} = \frac{M * (\times - h_b)}{I_r} \quad N/mm^2 \tag{3.15}$$

$$\sigma_{s2} = \frac{M * (h - \times)}{I_r} \quad N/mm^2 \tag{3.16}$$

For the calculation of I_2 the reinforcement area within the actual effective width according to paragraph 5 should be used. The resulting stresses of effects of shrinkage modified by creep and the effects of temperature can be calculated from the moments and normal forces derived from paragraph 6. If the composite bridge is unpropped during construction, the stresses in the steel sub-construction have to be added to the stresses due to composite actions.
The resulting stresses at any point at the most unfavourable load combination should not exceed the following maximum stresses:

- $0.85 * f_{ck} / \gamma_c$ - for concrete in compression
- f_y / γ_a - for structural steel in tension
- f_y / γ_a - for structural steel in compression taking account of local buckling
- f_{sk} / γ_s - for reinforcement in tension reinforcement in compression may be neglected.

The shear forces resulting from the global analysis are carried by the steel member only. Specific attention must be paid to the combination of compressive stresses and shear stresses near the supports of continuous bridge systems and to the lateral stability of steel flanges under compression.

3.6 SERVICEABILITY LIMIT STATES

In the serviceability limit state a check must be made for deflections of the bridge and the crack-width of concrete in tension.

3.6.1 DEFLECTIONS

The deflections of a bridge should be limited to ensure adequate rigidity of the bridge and to maximise the user comfort. The calculation of the deflections should be based on all actions on the bridge under serviceability states. Effects to be considered are :
- shear lag
- slip
- cracking and tension stiffening of concrete in hogging moment regions
- creep and shrinkage of concrete

The deflections may be calculated by global elastic analysis as explained in paragraph 6 using method 2 when the concrete tension stresses in hogging moment regions exceed 0.15 * f_{ck} if method 1 is used. The effects of creep for long term loading may be taken into account by using the proper ratio n in which creep is included. The deflection effects of shrinkage modified by creep and temperature can be calculated using the method explained in paragraph 6. When the steel sub-structure is unpropped during erection the deflections due to single action have to be added to the deflections of combined action.

In order to provide maximum user comfort, a camber is normally provided. This camber compensates the deflections arising from dead load of the structure and other permanent loads.

3.6.2 CRACKING OF CONCRETE

In composite bridges tensile stresses in the concrete flange can arise from:
- hogging moments in a cross section near a support for continuous bridges.
- concrete shrinkage.
- positive temperature differences between the concrete flange and the steel girder.

Due to these effects transverse cracks in the concrete flange may arise. The width of these cracks should be minimised in order to avoid corrosion of reinforcement- or prestressing steel and to prevent rupturing of waterproof membranes or asphalt. The crack width can be limited by using closely spaced longitudinal reinforcement.

In Eurocode 4 the crack width is limited to 0.3 mm. The present Dutch regulations limit the crack width to 0.2 mm at the minimum concrete cover above the reinforcement. The English regulations give a maximum crack width of 0.2 mm.

3.7 SHEAR CONNECTORS

The combined action of the steel sub-construction and the concrete flange can only be reached when limited slip or no slip occurs between the two structural elements. This is possible when elements are submitted to the interaction zone which form a shear connection between steel and concrete. These elements are called shear connectors. It is called adequate shear connection when the total load carrying capacity of the shear connectors is larger then the actual shear force at the interaction surface. In this case an optimal stress distribution in the composite member can be reached and the strength of the composite member is not limited by the amount and the strength of the shear connectors. In this chapter adequate shear connection is a starting point; inadequate shear connection is not discussed.

There are several types of shear connectors which can be used for transmitting the shear force from the steel member to the concrete flange:
- Bent up reinforcement bars welded directly to the steel top flange.
- Rigid shear connectors. These are small steel bars welded to the top flange of the steel beam. They transfer shear forces by exercising pressure on the concrete in a plane perpendicular to the direction of the shear. This type of connector is applied in combination with hoops to prevent uplifting of the concrete slab.
- Flexible shear connectors such as headed studs. These connectors allow for slip because of the local deformation of the connector. Failure occurs because of local crushing of concrete or shear of steel. The head of the stud prevents uplifting of the concrete slab.
- Friction grip bolts. This system of shear connection can be used in combination with precast concrete slabs. The shear force is transferred by friction.

The most commonly used type of shear connectors is the headed stud. The nominal static strength of these studs can be calculated using the equations of the new Dutch code for bridges.

1. shear-failure of steel : $P = 0.7 * d^2 * \frac{1}{4} * \pi * f_u / \gamma_v$ N (3.17)

2. concrete crushing : $P = 0.32 * d^2 * \sqrt{(f_{ck} * E_c)} / \gamma_v$ N (3.18)

$\gamma_v = 1.25$

For concrete qualities with a cube strength of 25 N/mm^2 or higher the following capacity can be reached:

d (mm)	P (kN)
16	52
19	72
22	97
25	125

The Dutch code also gives regulations for the fatigue strength of headed studs. The static strength of the studs should thereby be multiplied by a factor α_{dyn} :

$$\alpha_{dyn} = \frac{\delta D * \sqrt[5]{n_1}}{D_{max} * 9} \qquad (3.19)$$

The number and the spacing of the shear connectors should be in accordance with the longitudinal shear calculated by elastic theory for the loading considered. The longitudinal shear forces can be calculated from the vertical shear force distribution derived from the global analysis as explained in paragraph 6. Only those loading cases should be considered where combined action of steel and concrete is used.

1. Longitudinal shear stresses (general equation):

$$\tau = \frac{D * S}{b * I} \quad N/mm^2 \tag{3.20}$$

2. Longitudinal force per unit length n_v at the surface between steel and concrete:

$$n_v = \frac{D * S_c}{I_s} \quad N/mm \tag{3.21}$$

   The values of D, S_c and I_s should be determined separately for long term and short term loading by using the ratio factor n.

3. The normal forces resulting from shrinkage modified by creep and differences of temperature are introduced by the studs at the ends of the composite girder. The normal forces can be calculated with the method described in paragraph 6 using the short term n-ratio for temperature loading and the shrinkage n-ratio for shrinkage modified by creep. The introduction of the normal forces is assumed to spread linearly from 2 times the normal force divided by the introduction-length l_i at the support to 0 at a point L_i from the support. L_i is approximately equal to the effective width.

4. FABRICATION AND ERECTION

4.1 INTRODUCTION

4.1.1 OPTIMUM SPAN

About 80% of the bridges made in France the last 10 years, are composite bridges.
Ponts Metalliques, a magazine published by OTUA, Paris, gives insight in the development of spans, for which composite bridges are used.

- These informations concern highway bridges only.[1)]
- No difference is made regarding the bridge type (box girder, multi girder etc.).
- A distribution is set up with intervals of 10 m span.

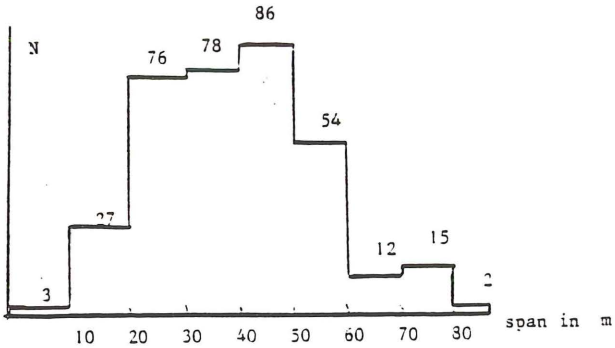


Figure 4.1 : Frequency distribution of spans

Conclusions:
The domain of spans in which composite bridges are competitive is located between 25 m and 60 m.
The optimum span is about 40 m.
Developments, in particular in the concrete industry, makes longer span economically more and more attractive. In Germany there is a clear tendency to apply composite structures in all new bridges, in which concrete slabs replace orthotropic decks.

Example:
The Nantenbach Viaduct, Baden-Würtenberg, now in erection, is such a continuous composite bridge.

The attractiveness is easily perceived, considering:

- the shorter erection time,
- the manufacturing of a concrete deck is about one third of the cost of manufacturing a more labour demanding orthotropic steel deck.

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1) This does not mean that no railway bridges are carried out as composite bridges. In particular the TGV-line Paris to Calais shows some fine specimen.

These costs ratio seems to grow more and more to the advantage of concrete, as labour costs increase relatively more than material costs.

- For railway bridges maintenance of the steel structure can be carried out completely, without interfering with railway traffic (this in contrast with e.g. a railway bridge with open steel grid).

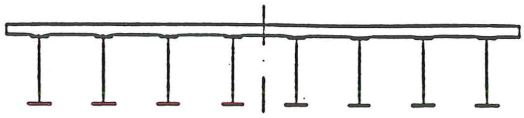

Figure 4.2 : Set of main girders

4.1.2 CONTRACTORS AND TENDERING

A difficult problem in the Netherlands seems to be the fact, that few contractors are equipped to carry out a project comprising a combination of steel **and** concrete in one structural element: the composite bridge. For this reason many contractors are inclined to offer an alternative **or** in concrete **or** in steel.
This situation regarding equipment is not typically Dutch. In England the situation is very much the same. Still, many bridges are made as composite structures. The reasons to apply composite bridges are completely economical. The choice, in France for instance, is **not** between a steel or prestressed composite bridge (a steel bridge is more expensive in the competitive range) but **exclusively** between a pre-stressed concrete or a composite bridge.

In an analysis, carried out by the SNCF, it was found that in tendering for the same project a prestressed design and a composite design, in about 70% of the cases, the composite structure was cheaper. The difference in price was 5 to 10%.

Next an analysis was carried out to see in which cases a composite structure was more advantageous. It was found that a composite structure had preference if:

- The site was easily accessible. Poor accessibility makes concrete pouring cheaper as it can be transported in smaller charges.
- The bridge could be erected using the launching method. Launching is not usual in our country. In France about 90% of the bridges are erected by launching. Differences in level, quite normal in mountainous areas in France, makes this point clear. However, in the TGV-track between Paris and Calais, not mountainous at all, launching was applied as well.
  On the other hand using pontoons and floating cranes may offer a similar advantage in our situation. This aspect needs further investigation.
- Spans are shorter than about 100 m and formwork is reclaimed. This aspect was, by far, the least important.

Conclusion:

- Though additional investigation is necessary, there are reasons to believe that also in the Netherlands this type of structure is competitive.
- In particular the cost aspects need careful consideration, to discover the mechanisms which determine the differences.

4.1.3 GENERAL INFORMATION

The three main structural elements of a composite bridge are:
- the steel structure,
- the concrete superstructure, the slab,
- the shear connectors.[1)]

Regarding concrete structures there are hardly new points of view compared to normal practice. To a large extent the same applies to steel structures. Corrosion protection, fabrication are well known topics, dealt with in the usual way. It is useless to pay much attention to this part. An exception is to be made for the corrosion protection of the upper flange of the main girder, where the steel structure is in contact with concrete (par. 4.5.6.1).
It seems as if there are no real problems. This, however, is not true. In the paragraphs to follow mainly the deviations from common practice are highlighted.

4.2 ERECTION METHODS

4.2.1 GENERAL

The erection methods depend largely on the type of cross section and the possibilities offered by the circumstances.

Site situation:
- a new bridge or the replacement of an existing bridge
- crossing a river, a highway, a valley, a railroad or a combination of these situations
- the concrete deck made in situ or prefabricated.

Types of cross sections:
- a set of main girders, evt. interconnected by a bracing or cross girders (fig. 4.2)
- a box girder - with concrete deck (fig. 4.3)
- with steel top flange and concrete deck (fig. 4.4)

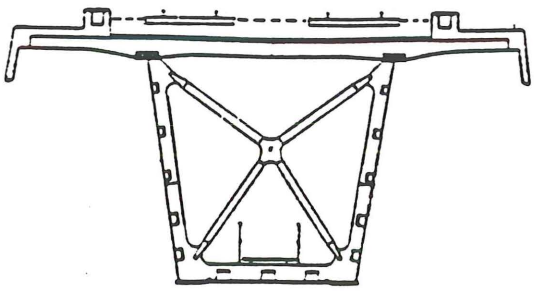

Figure 4.3 : Box girder with concrete deck

1) In some cases the bottom chords are also reinforced with concrete, in particular near intermediate supports. This is a development to which no particular attention is paid, as there is no essential difference.

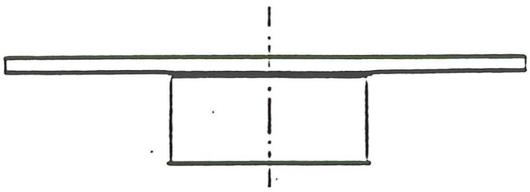

Figure 4.4 : Box girder with steel top flange and concrete deck

Regarding dead load: a composite bridge is heavier than the equivalent steel bridge, but much lighter than the prestressed concrete equivalent.
These consideration have a obvious influence on the erection method.
All these bridges (except the types 5 and 9, page 15 and 16) have a property in common: ***a concrete deck as a top flange***.
This means a great construction depth, a situation very much comparable with the use of orthotropic decks.
In using a multi girder system with girders interconnected by bracings, it is worth while to consider sets of two main girders. The bracing between these two girders automatically takes care of the stability during erection.
It leads to an *even number of main girders*. To avoid transport problems, the sets are preferably assembled on site.
If the main girders are mounted one after the other, temporary lateral supports are necessary.

4.2.2 THE MAIN METHODS OF ERECTION

It is not worth while to investigate all methods of erection. In special cases special methods should be developed. These situations are not within the scope of this report.
A particular aspect of bridge erection in the Netherlands is the tendency to fabricate large construction elements/units or even complete bridges. Quality (manufacturing, corrosion protection,testing) and efficiency (portal cranes, welding equipment) are kept under close control. Erection and transport are directly connected with these developments.
Only the most important erection methods are mentioned:

- Use of cranes for the mounting of the steel structure. The concrete deck is made separately in situ or is prefabricated.
- Launching in longitudinal direction of the steel structure. In cantilevering and launching the same problems show. Therefore they are discussed together. The concrete deck is made:
  * separately in situ
  * prefabricated and mounted after the erection of the steel structures. Sometimes the deck is mounted partially beforehand to protect special areas of the site, for instance railway tracks to be crossed (fig. 4.5).
- Lateral shifting of a complete composite structure to replace an existing bridge (fig. 4.6).
- Use of floating cranes to mount the steel structure or the complete composite structure
- Use of pontoons :
  • to carry the completed bridge
  • to be used as a support during launching of a bridge across river or canal (fig. 4.7).

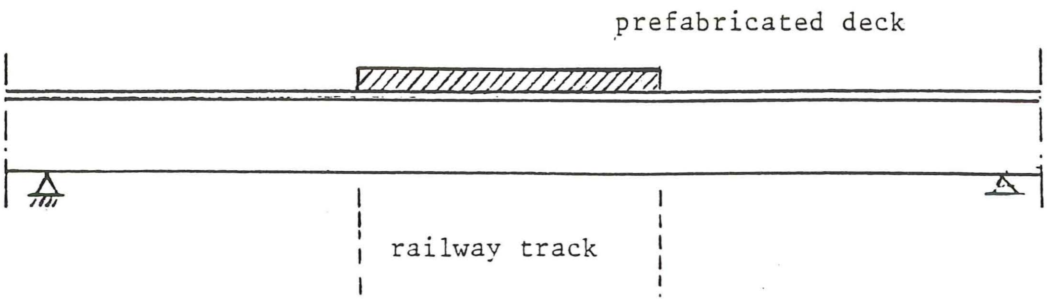

Figure 4.5 : Partial prefabricated deck

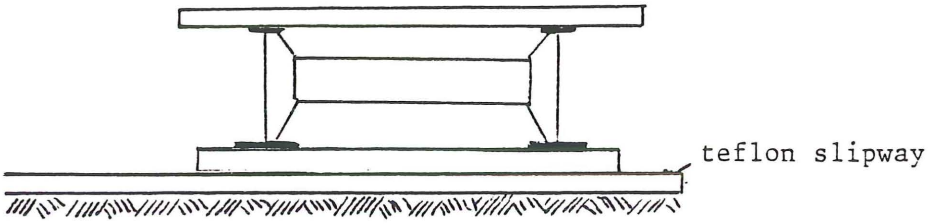

Figure 4.6 : Lateral shifting of a composite structure

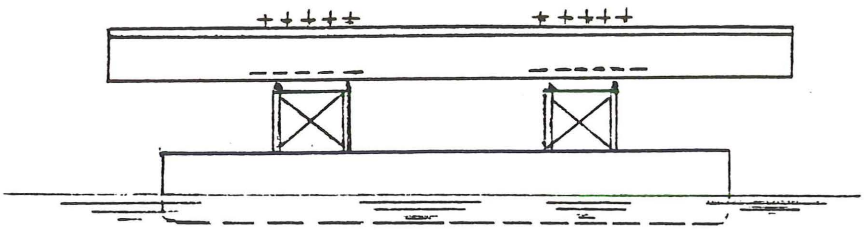

Figure 4.7 : Use of pontoons

4.2.3 THE USE OF CRANES

In many cases accessibility of a site is no problem. The load carrying capacity of one, and if necessary two, cranes is enough to mount sets of steel main girders or box girder elements. The use of mobile cranes usually implies transport of girders by road with lorries. Heavy elements are transported over water and need floating cranes.

Splices are bolted or welded. On site, bolted connections are to be preferred. The outer main girders are often welded for aesthetical reasons. The concrete deck is made afterwards in situ.

4.2.4 LAUNCHING

If the site is not easily accessible or parts of the site should be kept open to traffic, launching of the steel structure is an attractive alternative. In fact it is a sort of dynamic cantilevering. The steel structure is assembled on one of the abutments and moved towards the other side, eventually passing piers or temporary piers on which rollers are installed. Clearly the web passes the rollers, instead of all rollers being attached to the flange at favourably chosen, fixed places.

These rollers support the girder but in particular the web, introducing concentrated loads in every edge point of the web (fig. 4.8). Precautions with respect to local buckling should be considered. To distribute concentrated forces a special strip, included in the strength calculations, may be welded to the lower flange (fig. 4.9).

Welds between web and lower flange should be checked with respect to vertical concentrated roller forces .

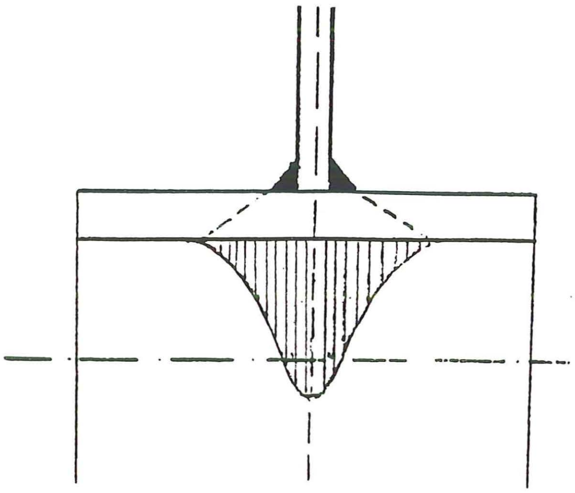

Figure 4.8 : Introduction into the web of the concentrated load

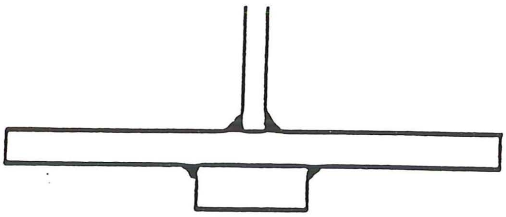

Figure 4.9 : Strip to distribute the concentrated load

If the bridge passes a highway or railroad, it may be useful to avoid working above these areas, to reduce the danger for traffic and labourers as much as possible (fig. 4.5). In that case it is to be considered to provide for a prefabricated local concrete deck, before hand. The consequences with respect to higher concentrated loads on the webs due to the rollers should be taken into account.

For long distances between the piers, a launching nose is used: a light steel structure, mounted in front of the bridge. The moment the nose reaches a pier (or abutment) it starts serving as a support. Extra pushing power is needed to overcome the deflection of this launching nose and main girder. This necessarily introduces horizontal forces on a pier or abutment in longitudinal direction.

The maximum length of the launching nose depends mainly on the maximum moments in the main girder in the extreme position.

To guide the steel structure, the flange fits between the vertical flanges of the rollers (fig. 4.10).

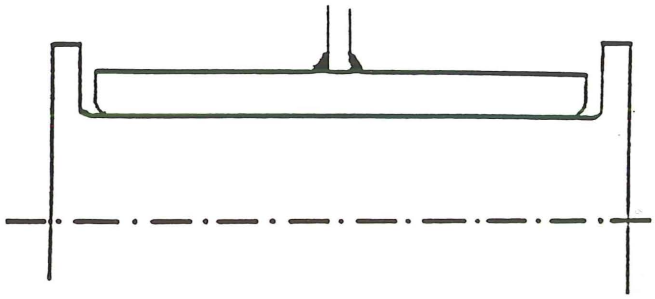

Figure 4.10 : Flange and roller

In case of launching of a horizontally curved steel structure, lateral forces are exerted on these flanges and consequently also on the piers or abutments. In particular in mountainous areas it may happen that these horizontal forces govern the design of tall piers.

4.2.5 LATERAL SHIFTING OF A COMPLETE STRUCTURE

In replacing an existing bridge, it may be very attractive to build a completely new bridge (a composite bridge, supposing the construction depths to match) next to the old one. This can be done independently, i.e. not interfering with traffic or whatsoever. By laterally shifting the set of bridges, or each bridge separately, an excellent solution is obtained. In particular for railway bridges, steel or concrete, this method has been used frequently. In literature this method is mentioned:
The Dundas Street Bridge in Trenton, Ontario: a continuous composite bridge, statically indeterminate, was laterally slid into its final position. (IABSE Symposium, Leningrad, 1991).

4.2.6 FLOATING CRANES

In contrast to launching, the use of floating cranes and pontoons is common practice in The Netherlands with its many navigable rivers and canals. A floating crane lifts the bridge from a pontoon and places it on its supports.

For simply supported bridges there should be no real objections to use this method even for completed, composite bridges. For redundant composite structures including concrete slab, stresses in the concrete deck during lifting and transport should be considered carefully. Because of too much risks, this method should be considered with utmost care for redundant systems.

In case of separate erection of the steel structure, we meet the usual problems during erection. No special attention is paid to this situation.

4.2.7 THE USE OF PONTOONS

In fact there is little difference between pontoons and floating cranes. Pontoons were used at the Nantenbach Bridge across the Main, to mount the mid span of about 200 m. (IABSE Symposium, Leningrad, 1991). In general, the risk of using pontoons to support the end of a composite structure while being shifted across a river should be considered careful. During the launching the bending moments and shear forces change continuously.
Unless all these situations are carefully accounted for in the design (economically a doubtful option), there is risk of cracks fractures in the concrete deck.

4.3 STEEL

4.3.1 STRUCTURAL STEEL

The usual steel used in bridge engineering are Fe E 355 and Fe E 235. If strength is predominant, Fe E 355 has preference because of lower cost to strength ration.
D-quality is necessary to ensure sufficient notch toughness after welding, even under severe conditions. Z-quality is required if vertical stiffeners are welded to the flanges of a girder. Swiss, French and Dutch experience learns, that it is useful to avoid stiffeners against buckling, by taking greater plate thickness, thus saving labour costs.

4.3.2 WEATHERING STEEL

The use of weathering steel has a great number of beneficial environmental aspects. The removal of a coating, e.g., is expensive as severe precautions are to be taken to comply with legal requirements to protect environment against pollution.

Weathering steel is applied in a rural environment. Cities, coastal and industrial areas should be avoided. To be more specific:
- The contents of chloride of the air should not exceed 0.1 mg/100 cm^2/day.[1)]
- For industrial areas the level of 2.0 mg/100 cm^2/day sulphur trioxide should not be exceeded.
- It is advised not to use weathering steel without coating, if the moisture coefficient of the air exceeds 60 % continuously.
- Weathering steel is not advised :
  - for narrow depressed roadway sections
  - between vertical retaining walls
  - in case of narrow shoulders
  - bridges with minimum vertical clearance and deep abutments adjacent to the shoulder.
- A clearance of at least 3.50 m is recommended between the bottom flange and stagnant water.

1) In the USA the 0.1 is replaced by 0,5.

Weathering steel lends it favourable properties regarding corrosion from alloying elements such as copper and even some additional sulphur. These elements form a patina protecting the material against corrosion. An aggressive environment leads to decay of this patina and consequently to proceeding corrosion.

In England an extra shell of 1 mm plate thickness is applied to cope with this problem [1)]. If environmental conditions are no obstacle, the extra plate thickness seems the better and more economical choice. In particular in Scotland weathering steel is used frequently. In America about 50% of the bridges with average spans are built as composite bridges, in particular highway crossings. In this 50%, about 10% is carried out using weathering steel.

4.3.3 HIGH STRENGTH STEEL

In England for some important viaducts Qst 460 (i.e. steel with a yield stress of 460 N/mm^2) is applied. No further details are available.

4.3.4 HEADED STUDS

The material properties for headed studs are given in NEN 6788.

4.4 CONCRETE FORMWORK

4.4.1 CONCRETE QUALITY

According to Dutch Codes (NEN 6730), the minimum characteristic cube strength must be 15 N/mm^2 (grade B15).[2)]
All related data (modulus of elasticity, creep factor etc.), are presented in the applicable code. The final choice of the concrete grade to be used, depends on the following parameters:

- The use of flexible or rigid connectors.
  According to [4.8], headed studs may be considered as flexible, provided:
  - the diameter does not exceed 22 mm
  - the overall length is more than 4 x the diameter
  - the concrete grade is less than B30. [3)]

---

1) The economical balance is then:
   - extra material, 2x1 mm per 1 m^2 plate area $\approx$ 15 kg
     steel costs about dfl. 1.75 per kg
   - painting of 2x1 m^2 plate, to be repeated every 10 year
     costs about dfl. 60.00 per m^2

2) NEN 6788 however supposes a minimum grade B35.

3) NEN 6788, indicates, that headed studs should be considered as rigid connectors, as full interaction is required. The formulas, given in this code indicate also, that for B30 and lower grades, the concrete governs the strength of the connections. If B35 or higher grades are used, the code and literature are in agreement.

- The relation between the connector strength and concrete strength, (see [1]) and NEN 6788).
- The relation between the concrete grade, creep and shrinkage, (see [1]) and NEN 6788).
- The environment may cause additional requirements for the concrete mixture which directly affects the grade.

In the investigation up till now no use of light weight concrete was encountered. British information was:
- if applied, then scarcely
- the Ministry of Transport does not prohibit the use
- no example is available.

4.4.2 CONCRETE CONTROL

The procedure to be followed and the precautions to be taken in the construction of a composite bridge are, in general, a combination of those required for the erection of steel structures and for reinforced concrete structures.

The proposed method and sequence of construction (propped or unpropped) for the superstructure are essential inputs in the design process and should be clearly indicated and described in the final design plans and site instructions.

Consideration should be given to the speed and sequence of concreting, in order to prevent damage to partly matured concrete. This damage might be the result of limited composite action, under the subsequent pouring operations. Due to increasing dead load, deformation of the steel structure increases as well.

During the pouring of the concrete slab, each concrete charge should be checked and a clear book keeping should be maintained regarding the deposit of the contents of each charge.

4.4.3 FORMWORK

Formwork can be distinguished in:
- formwork to be reclaimed:
  * the conventional timber formwork, to be reclaimed after solidification of the slab,
  * moveable formwork
- formwork to be integrated in the slab and eventually participating in the load carrying capacity.

Timber formwork offers no specific problems. The inevitable gap between timber and steel flange can be closed with a resin. The results were excellent (Marsh Mill Viaduct, England).

A movable formwork can be made as shown in figure 4.11.
An alternative is shown in figure 4.12, Viaduc de la Haut Colme, France.

---

1) NEN 6788 indicates, that headed studs should be considered as rigid connectors, as full interaction is required. The formulas given in this code indicate also, that for B30 and lower grades, the concrete governs the strength of the connections. If B35 or higher grades are used, the code and literature are in agreement.

Types of formwork to become part of the structure are:
- corrugated plates
- Omnia planks
- glass fibre reinforced cement
- the GRP-system (Glass fibre Reinforced Plastic).

In England the use of corrugated plates is prohibited as the Ministry of Transport supposes this construction to have negative effects on fatigue behaviour. In the USA the corrugated plate is used as formwork to carry the reinforcement and concrete, without participating in the load carrying capacity. The plate is "flattened" by filling the deeper parts with poly-styrene. The reinforcement is located on top of the corrugated plate. (see figure 4.13)

Experience in France, where profiled plates have been used, showed that due to tolerances it is not practical to locate prefabricated reinforcement grids.

Omnia planks are used extensively in England (fig. 4.14). The planks are supported by the girder flanges. Between plank and flange a bituminous strip takes care of an equally distributed support. Leaking of water is avoided.

Clearly these planks act in transverse direction. The concrete slab is calculated considering the planks as being completely incorporated. The gap between two planks is filled with silicon sealant again in order to avoid leaking.[1]

Glass fibre reinforced cement or plastic as a support for the reinforcement and poured concrete are accepted as good solutions.

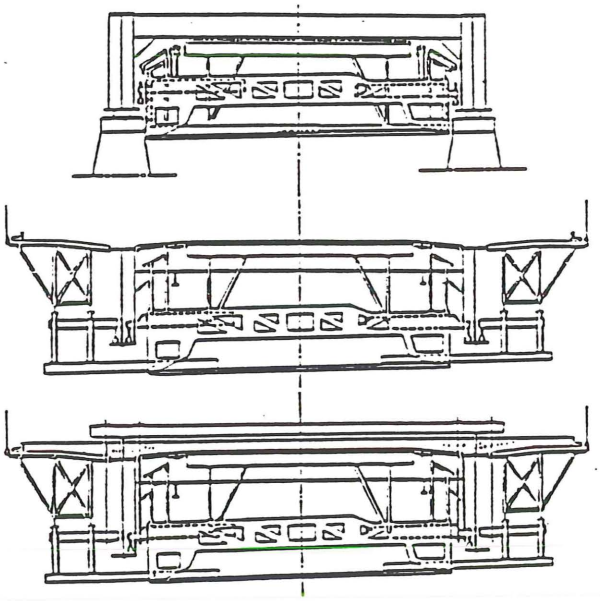

Figure 4.11 : Movable formwork

1) The fact that also in longitudinal direction stresses are to be transmitted is considered to be of no importance. Still, rather awkward stress combinations may occur in these points.

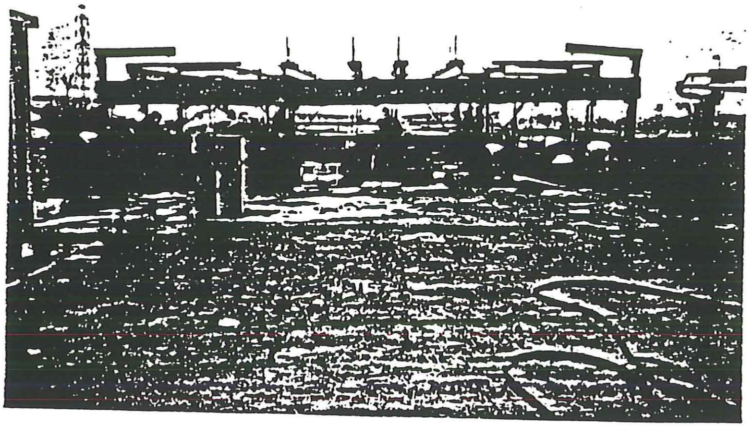

Figure 4.12 : Viaduc de la Haute Colme, France

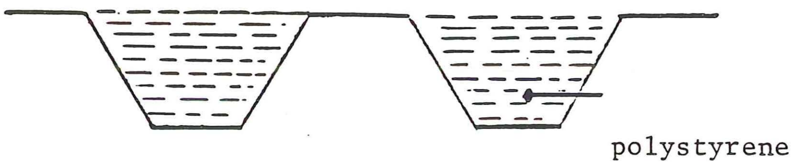

Figure 4.13 : Polystyrene fillings

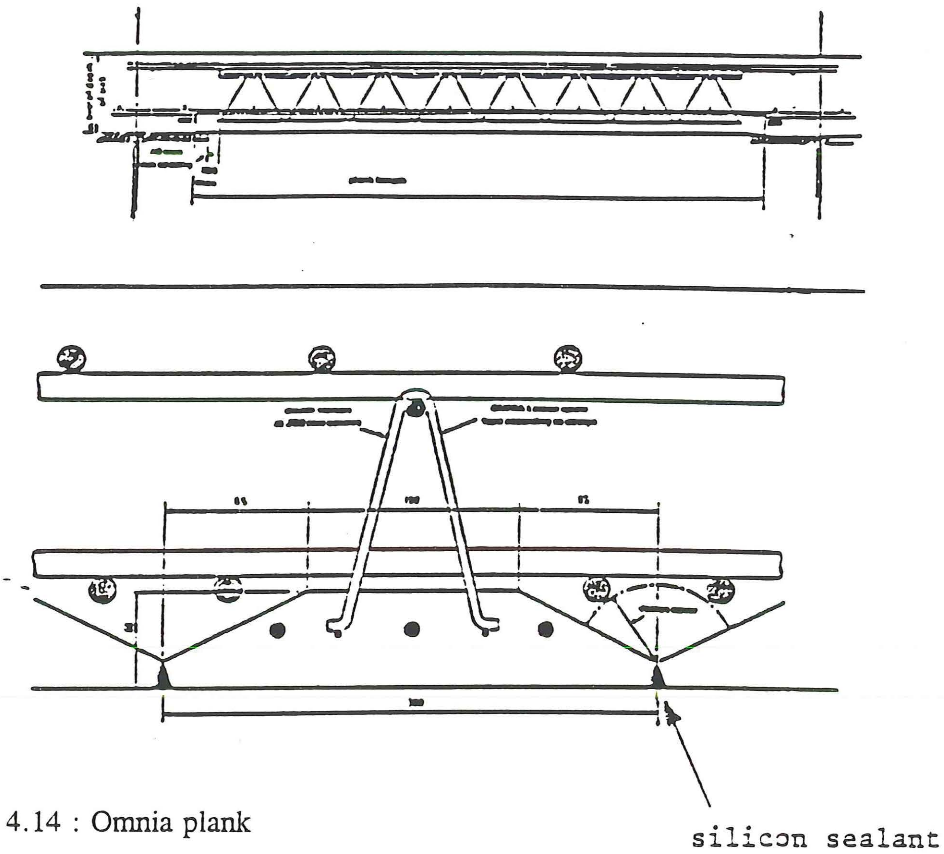

Figure 4.14 : Omnia plank

4.4.4 PREFAB SLABS

Precast concrete slabs (Omnia planks) are used as permanent formwork. The next step is a completely prefabricated slab and thus saving formwork. For this type of structure the following aspects should be considered:

- The slab should be supported by two steel flanges only, in order to keep the mechanical situation statically determinate.[1)]
- The width of a prefab slab should be relatively small, in order to follow the flanges of the steel girders smoothly.
- In the slab "pockets" should be saved for shear connectors. These pockets should be large enough to be filled, in situ, with non shrinking grout.
- The transverse joints have to be filled to make a connection for the stress transfer.
- Special attention should be paid to the joint between the top flange of the steel girder and the prefab slab, to prevent corrosion. The gap between slab and flange must be protected completely against corrosion. A full coating is not enough. A neglected damage of the coating, e.g. damage due to mounting of the concrete slab, starts the corrosion process. Afterwards no repair is possible. A mortar bed or an equivalent intermediary is necessary.

4.5 SHEAR CONNECTORS

4.5.1 TYPES

In the past several types of shear connectors were developed. Older types of shear connectors are shown in fig. 4.15, angles, and 4.16, horseshoe connectors.
In the end mainly one type prevailed: the headed stud.
In the Dutch code NEN 6788, regulations are presented for this type only.
Other types are allowed, provided it is shown that these connectors lead to a safe and durable construction.
The average output per man per day is about 400 studs. This productivity is an economical advantage not yet surpassed by any other connector type.
Moreover, due to symmetry, the mechanical behaviour of a stud is independent of the direction of the shear force.
New developments, which are still under discussion are:

- the perfobond strip (fig. 4.17), in which "concrete shear connectors" are introduced as a calculation method.
- Headed studs with thicker footing, i.e.: the shaft 16 mm, the footing 22 mm. These connectors are supposed to behave like 22 mm connectors, but are mainly 16 mm connectors (fig. 4.18).

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1) A redundant system is economically not very attractive because of additional reinforcement (see also par. 4.2.5.)

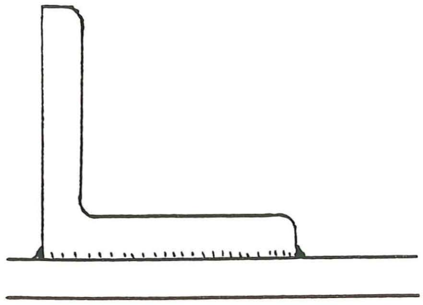

Figure 4.15 : Angles

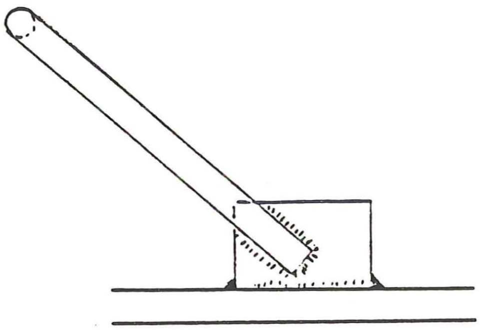

Figure 4.16 : Horse shoe

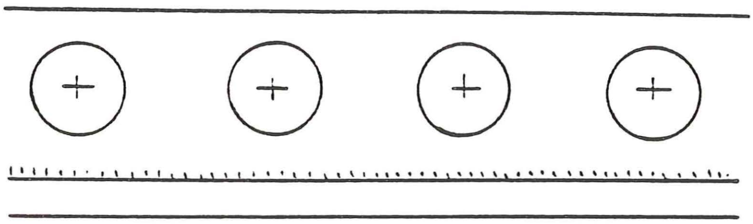

Figure 4.17 : Perfo bond

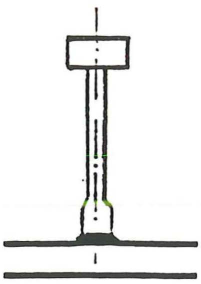

Figure 4.18 : Headed stud with a thicker footing

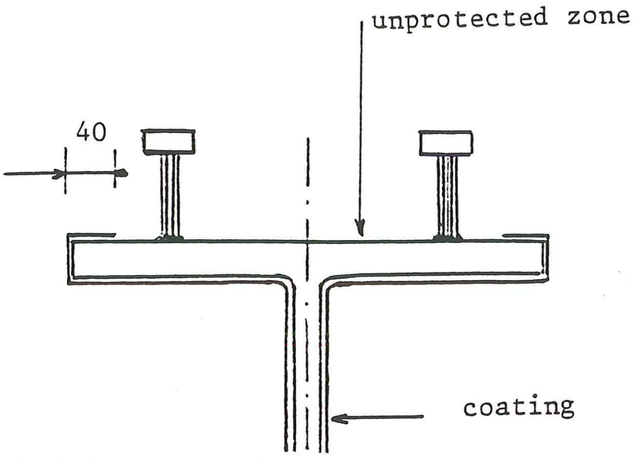

Figure 4.19 : Studs in unprotected zone

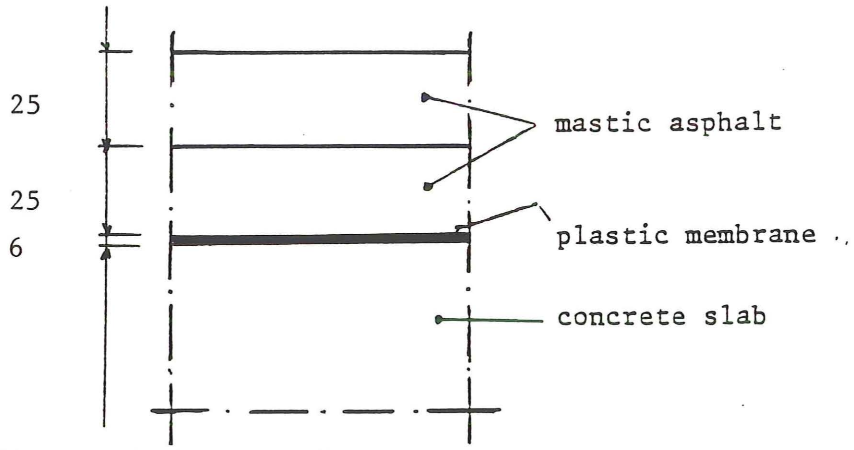

Figure 4.20 : Water proofing systems

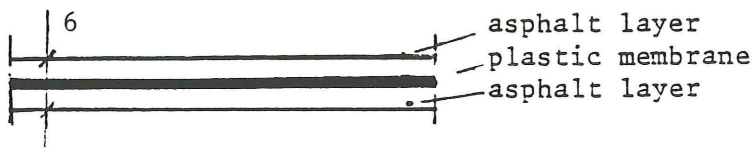

Figure 4.21 : Plastic membranes covered with bitumen

4.5.2 WELDING

The welding of studs takes place by means of a sort of pistol.
The welding parameters are inclined change during the process. As a result the hardness in the heat affected zone may become too high. This is why these parameters should be kept under close control.
Studs are usually welded to the top flange only ; not to splice plates.
Advantages welding in the workshop are: no moisture, best circumstances, energy sources at hand.
Disadvantages are: poor accessibility of the steel structure.
The best is to adopt an erection method to reducing this disadvantage.

A low current causes porous welds: high current (1900 A, ∅ 22, during 1 second) gives a greater weld pool.
- Evaporation of material protect the weld against nitrogen from the air.
- Current should be administered symmetrically, to avoid a one-sided arch, reduction of material evaporation, less protection and so the generation of a porous weld.
- If the gap between stud and flange during welding is too small, then first solidification occurs near the edges. A lack of liquid weld material causes cracks in the centre of the weld.
- A ceramic ring serves as a mould.
- An asymmetric melting, causes defects; the gap between shaft and ring is not properly filled.
- Practical points in stud welding:
  - the surface must be dry and clean
  - the ceramic ring must be dry
  - prevent blowing
  - regular control of parameters
  - make a test before starting.

4.5.3 TESTING

First of all visual inspection is carried out, directly after welding.
Next about 2% of the studs (the code gives the precise number) are tested in bending in a way, prescribed in the code as well (e.g. BS 5400, part 5 and 7). Procedure is:
- each stud is hit several times with a hammer.
- 2% of the studs are bent over an angle of about 50 degrees.
- if no failures occur, it is supposed that the work has been carried out according to good workmanship.[1)]

4.5.6 CORROSION

For a long time it was an accepted idea, that maintenance costs for concrete structures were much lower than for steel structures. However, more and more the idea grew, that this supposition was incorrect. Now it is believed that there is no difference. There is no argument to think that the same does not apply to composite structures.

Corrosion protection is important, mainly in three places:
- The steel structure.
- The use of weathering steel was already mentioned. The coating systems for the steel structure do not differ from usual practice.
- The reinforcement of the concrete slab.
- The transition zone between concrete and top flange of the girders.

The last two aspects are discussed more thoroughly; the first point, concerning the steel structure, offers no particular problems.

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1) Quality insurance regarding this item are not yet integrated in the Dutch Regulations. Bending and hammering of studs should be carried out about one day after welding. Hydrogen brittleness does not show immediately. This is why the testing should be carried out some hours or a day after welding. To make clear that the testing has been carried out, the bent studs are left as they are.

4.5.4.1 THE TRANSITION ZONE

Only 40 mm of the flange edge is protected with a coating; the central part remains unprotected. After 30 years of experience no problems have shown yet. [1)]

The concrete, poured in situ, is in direct contact with the top flange of the girders. Only a strip of about 40 mm width is protected by a coating.

The connectors are welded to this unprotected part of the flange (fig. 4.19).
The transition: steel to concrete slab, is considered to be the most vulnerable part. Steel is supposed to be protected by the alkaline effect of concrete. The concrete is not always water tight. It could be argued that the alkaline protection disappears with time due to capillary water. If corrosion occurs the rust damages the concrete slab, which then, inevitably, must be replaced. There is no alternative.

Doubts regard the remaining 70 years of its existence (supposing the life time to be 100 years). The two important questions are then:
- Is it necessary to worry about this problem. The answer is simple: nobody can tell.
- How do we ensure a 100 years life time. No practical alternatives to improve the situation have been proposed, but protecting the whole flange.

4.5.4.2 THE WATER PROOFING SYSTEM

Concrete has the unfortunate property of crack development, due to tension, shrinkage and creep. In fact a calculation model is based on this property. Essential for the protection of the reinforcement of the concrete slab is a water proofing system. Water, polluted by chemical elements like sulphur trioxide and salt, causes danger of corrosion of the reinforcement of a concrete slab. Frost is a second danger, however mainly for the concrete itself.

Conclusion: "No concrete slab should go without a waterproofing system".

Several water proofing systems have been developed. Such a system might be built up of (fig. 4.20):
- * a mastic layer or,
  * a primer and then a plastic membrane covered on both side with bitumen.
- on top of that a wearing surface:
  * in highway bridges : 2 layers of melted asphalt, thick 25 mm each,
  * in a railway bridge: 1 layer of melted asphalt, thick 30 mm, or,
- a second system used in railway bridges is a primer and then three plastic membranes, again covered on both side with bitumen (fig. 4.21).

A third system is the use of a liquid resin, solidifying after application.
The thickness can be kept under close control and can be adjusted if necessary.
The hardness guarantees sufficient resistance against damage. As in the case of mastic, the advantage is the complete absence of overlaps.

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1) Defects are easily detected as pulverized concrete and rust betray themselves, by leaving the remains of the dilution, in which small particles of rust and concrete.

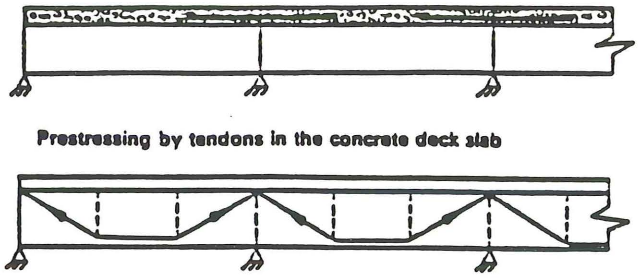

Figure 4.22 : Prestressing by external tendons

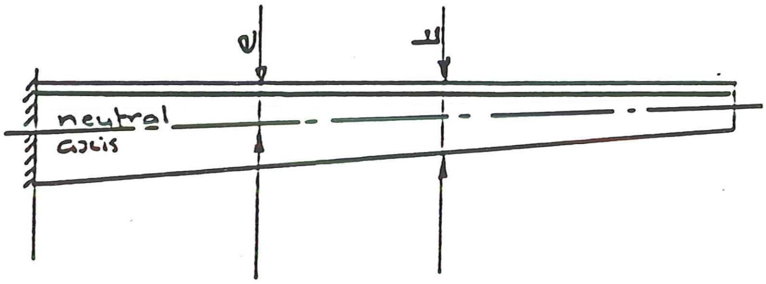

Figure 4.23 : Straight tendons and variable eccentricity

Comment:
- Mastic can be an excellent cover but requires a careful application in order to prevent thin places. The advantage is its continuity. The effect can be considerably improved by requiring, that the thickness should be at least equal to three times the grain diameter.
- The plastic membrane settles the water proofing problem, if applied according to good workmanship. It has the advantage of ensuring overall equal thickness, but the disadvantage of overlaps. If these overlaps show defects; only the path of the water is longer.
- The asphalt layers distribute the traffic loads (highway bridges) or protect the plastic membrane against damage due to penetrating ballast.
- The primer serves as a intermediary between concrete and plastic membrane. A certain primer belongs to a certain membrane. A mix-up easily leads to a failure.
- The layer thicknesses are obtained from experience only and consequently need testing.
- The cohesion between concrete and waterproofing system must be tested. No regulations have been given in our bridge engineering codes, up till now.
- The elasticity of the asphalt layers is improved by adding rubber particles. They act like little springs by adding Eventhaan to the asphalt layer, a sort of network is created in the layer which improves the resistance against deformation.

It is generally believed, that manufacturing of a water proofing system can be carried out without technical problems and to complete satisfaction.

Future developments:
- reduction of dead load up till 50% by using polyurethane.
- shorter time of application
- for environmental reasons: the development of epoxy products to replace the tar component.

4.6 PRESTRESSING

4.6.1 INTRODUCTION

Longitudinal internal prestressing at an intermediate support is common practice "on the continent" but rare in England and the United States.
One of the reasons is the German Standard (1952), stating that, to ensure durability, it is better to prevent cracking than to use a membrane to keep out the water.[1)]
The width of the cracks can be minimized by an appropriate sequence of construction of the slab, i.e. mid span before supports.
Moreover, additional closely spaced longitudinal reinforcement helps to keep crack width under control. In France external prestressing is accepted practice.

4.6.2 METHODS OF PRESTRESSING

The next methods are used:
- longitudinal prestressing:
  * enforced support displacement
  * tendons in slab or external
- transverse prestressing:
  * in situ
  * prefab

Lowering of an intermediate support:
Longitudinal prestressing of the concrete slab by means of lowering of an intermediate support is often the cheapest method. However, the loss of the initial prestress due to shrinkage and creep is far greater than by the use of tendons.[2)]

Furthermore, the costs of jacking increases rapidly with the span and the weight of the bridge.

Internal tendons:
The use of *internal* tendons is relatively expensive. Attention must be paid to the concrete pouring sequence:
- first the slab at the intermediate supports is made and prestressed after solidification,
- next the rest of the slab is made.[3)]

Compared to prestressed concrete bridges the loss in prestress is much higher for composite bridges, due to the axial and flexural stiffness of the steel sections.
The solution: a separate slab, prestressed, allowed to creep and then fixed to the steel structure would be much better but is often impracticable.

---

1) Still no composite structure in Germany is made without a waterproofing system.
2) A loss of 50% was found for an existing bridge.
3) This sequence is in contrast with practice for non-prestressed bridges.

External tendons:
In France prestressing by *external* tendons is common practice (fig. 4.22).
Improvement of the quality of tendons with a polyethylene housing and more advanced calculation methods has resulted in interesting achievements. A great advantage of this method is the possibility to replace the tendons and to adjust the prestress.

It is to be noted that the external tendons are supported every 10 to 15 m.

Prestressing in transverse direction is common for large bridges with cantilevering slabs. Varying thickness of the slab, applied in France and Switzerland, enables transverse prestress with almost straight tendons; the variable eccentricity matches with the variable bending moment (fig. 4.23).

For transverse prestress in prefab slabs, it is necessary that the eccentricity is the same for all tendons to prevent undesirable curvature to occur. This can be a problem for the housing of internal longitudinal tendons, as misalignment of these housing may occur.

Alternative for prestressing:
In particular in Switzerland and The United Kingdom there is a tendency to keep cracking under control by using closely spaced longitudinal reinforcement and a water proofing system.

Example: The Marsh Mill Viaduct, Plymouth, England.

5. COST INFORMATION FOR COMPOSITE BRIDGES

5.1 Introduction

Available cost data for composite bridges in the Netherlands and references referring to costs for composite highway or railway bridges are presented. Cost data has been collected for two highway bridges and two railway bridges. Caution is exercised when comparing costs between concrete and composite designs for the same bridge because of differences in the sophistication and completion of the composite alternative. As a result, only global construction costs for substructure and superstructure are compared. For each proposed composite alternative the design is reviewed. Cost comparisons between different bridges are more difficult to make and have thus not been attempted. This is due to the following differences and difficulties:

- Span lengths, width, loadings, site conditions and other design constraints. Ideally, designs and cost calculations should be conducted for "typical" short, medium and long span highway and railroad bridges.
- Many cost parameters are missing for individual bridges.
- Material, fabrication and contractor costs can vary widely, reflecting economic factors that are not appropriate for this study.

References referring to composite bridge costs are reviewed.

5.2 AVAILABLE COST DATA

5.2.1 INTRODUCTION

In Table 5.1 cost figures (in percentages) are given for four bridges: two highway bridges and two railroad bridges. Note that the composite alternative prices are given for the superstructure, substructure and the total bridge: the total price of the composite option is assumed equal to 100%. The differences in cost between the composite and concrete options are given in percentages. For example: The total cost of the concrete bridge "C" (including interest payments) is 8% more expensive than the composite alternative, equally, the concrete superstructure of bridge "C" (including interest payments) is 9% more expensive than the composite superstructure alternative.

Table 5.1: Summary of costs for concrete and composite bridge options

Bridge	Description:	Proposed composite alternative:			As-builtconcrete alternative:		
		Sub.	Super	Total	Sub.	Super	Total
"A"	Railway bridge	--	100**	--	--	+1	--
"B"	Highway bridge: Approach spans						
	(alternate # 1)	--	--	100	--	--	−14
	(alternate # 2)	19	81	100	+3	−14	−11
"C"	Highway bridge:				+2	+ 2	+4
	Main spans	23	77	100	+4*	+9*	+13*
"D"	Railway bridge: (near Rotterdam)	--	100	--	--	−5***	--

* Including interest payments
** The price for this alternative is based upon cross-section # 2 (see fig. 5.7).
*** Not built

5.2.2 BRIDGE "A"

This bridge consists of a series of simple spans 35 m in length and carries two parallel railway tracks. The resulting deck width is 12 m. The bridge is straight and the site provides excellent access by water, rail and road. No overall height restrictions were placed upon the design.

The composite alternative consists of a ballasted flat slab and a steel box caisson with interior K bracing, see figures 5.1 and 5.2. Unfortunately substructure costs for both alternatives were not available, see table 5.1. Superstructure costs for both concrete and composite options are similar.

An examination of the proposed design for the composite alternative suggests the following with respect to cost :

- Flat slab construction reduced reinforcement placement and concreting costs.
- Ballasting allows for easy rail maintenance when automatic track laying equipment is used. It also tends to lower vibration and sound problems, reduces the exposure of individual element to wheel loads (thus lowering the number of contributing cycles for fatigue considerations), but increases the weight of the bridge.
- Costs may be reduced further if continuous span construction is used due to a elimination of construction joints, bearings and a more rational use of available material. Continuous construction increases reinforcement costs and also provides for a stiffer structure. Long term durability is increased due to the elimination of expansion and construction joints if hogging moment cracking widths are controlled at service load limits.
- Costs may be reduced if the steel superstructure is shop fabricated and welded in larger sections, if barge erection remains possible. The elimination of field joints, especially welding, can reduce costs increase quality and structural durability.

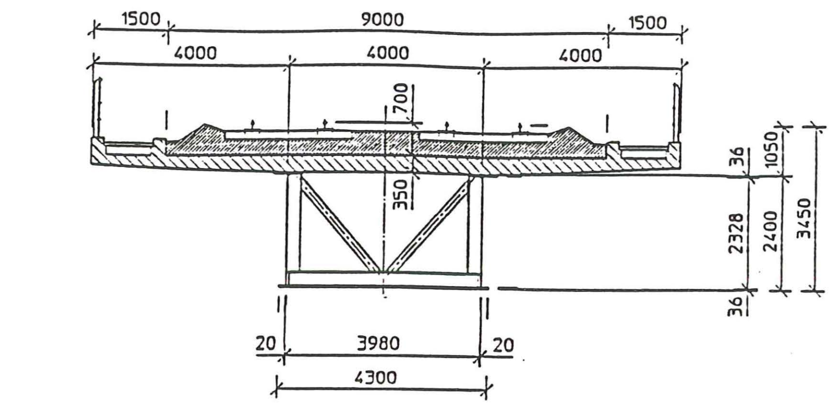

Figure 5.1 : Typical cross-sectional geometries for the composite alternative, bridge "A" (Railway bridge)

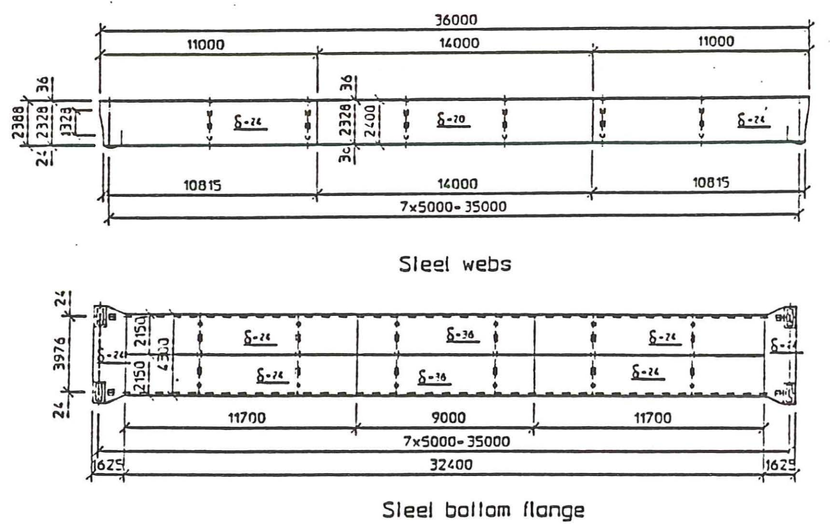

Figure 5.2 : Typical longitudinal view of the composite alternative, Bridge "A" (Railway Bridge)

5.2.3 BRIDGE "B", APPROACH SPANS

Alternative #1

This alternative consists of a series of approach spans between 42 and 61 m in length and carries six lanes of traffic with an extra approach lane on one side. The resulting deck width is 34.4 m. The bridge is slight curved at one end, the remainder being straight. The site provides excellent access by water, rail and road. No overall height restrictions were placed upon the design.

The composite alternative is based upon a preliminary design only and consists of a corbelled (haunched) concrete deck concreted on precast concrete units which span between adjacent steel girders, see figures 5.3 and 5.4. A total of 11 parallel main girders are used. Girders are fabricated in 20 m long units which are erected using temporary and permanent supports.

Cost estimates for the approach spans of bridge B in table 5.1 are based upon estimates made by RWS and independent contractors. An examination of the composite alternative indicates that some cost savings may be possible by modifying the design, as well as fabrication and erection procedures. There are indications that some contractors have made modifications to the original design when preparing their own bids. Some modifications which may have the most important cost implications include the following:

- Flat slab construction was not used. It would appear that individual wheel loads control slab thickness and necessitated a corbel near the top flange of each main girder. This complicates reinforcement placement and concreting. The precast concrete units themselves do not contribute structurally and represent about 7% of the costs of the approach spans.
- Steel weight can be saved, shear connectors can be eliminated and a haunch in the slab provided by a concrete top flange to the steel girder.
- Costs may be reduced if continuous span construction is used due to a elimination of construction joints, bearings and a more rational use of available material. Continuous construction increase reinforcement costs but also provides for a stiffer structure. The elimination of field joints, especially welding, can reduce costs increase quality and structural durability.
- Costs may be reduced if the steel superstructure is shop fabricated and welded in larger sections, if barge erection remains possible. The elimination of field joints, especially welding, can reduce costs increase quality and structural durability.

Alternative #2

An independent design of the approach spans was performed. Drawings showing the general layout of this option are not available. Note, however, that the overall difference in price between alternative #1 and alternative #2 is similar.

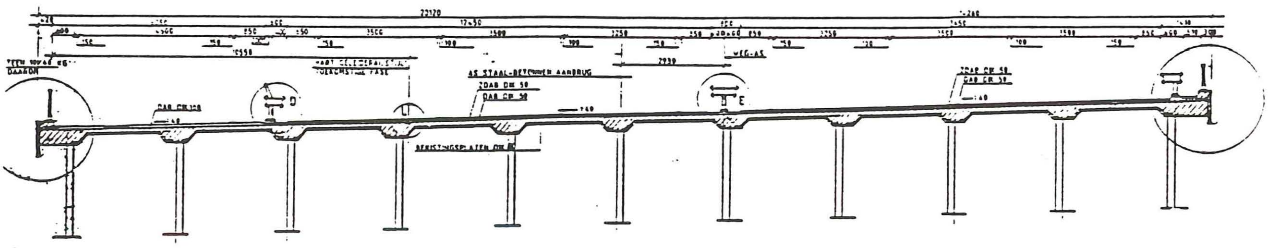

Figure 5.3 : Typical cross-sectional geometries for the composite alternative, Bridge "B" Approach spans for alternave #1 (Highway Bridge)

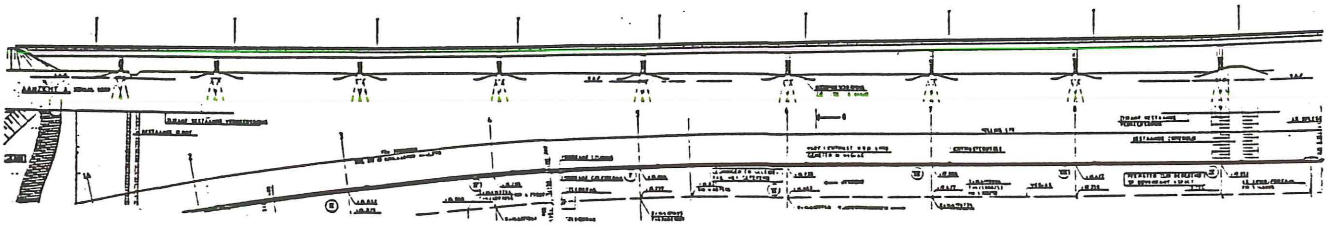

Figure 5.4 : Typical longitudinal view of the composite alternative, Bridge "B" Approcah spans for alternative #1 (Highway Bridge)

5.2.4 BRIDGE "C", MAIN SPAN

The general situation of the composite alternative is shown in figures 5.5 and 5.6. This design is similar to previous cable-stayed composite bridges built outside of the Netherlands.

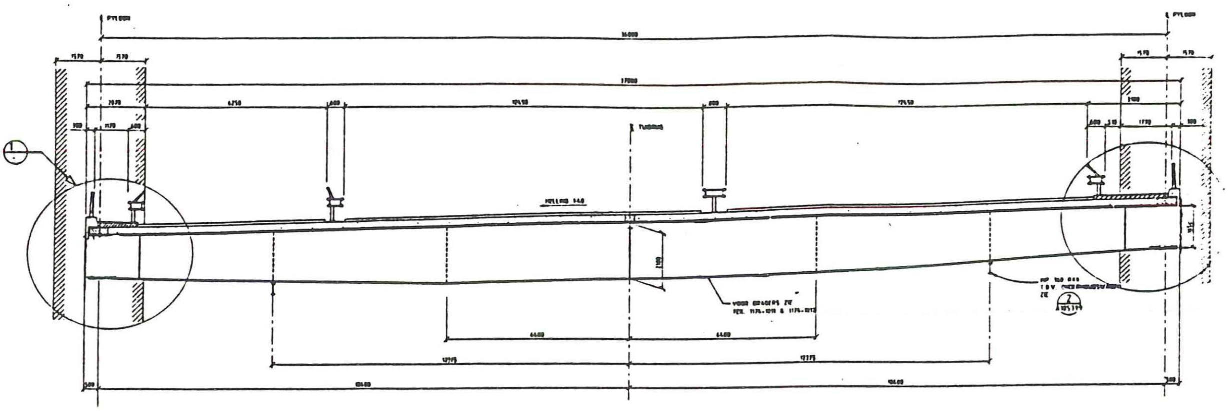

Figure 5.5 : Typical cross-sectional geometries for the composite alternative, Bridge "C" Main span (Highway bridge)

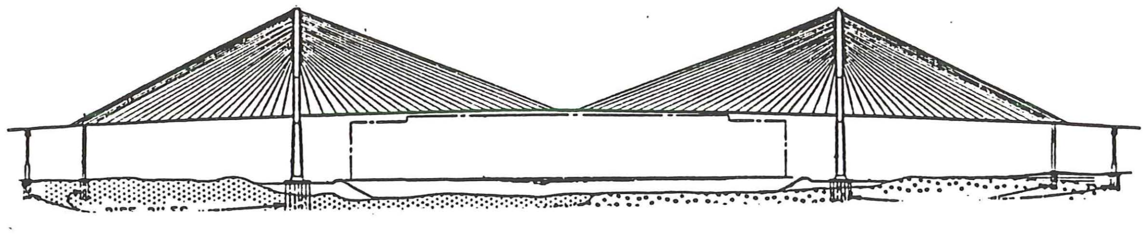

Figure 5.6 : Typical longitudinal view for the composite alternative, Bridge "C" Main span (Highway bridge)

5.2.5 BRIDGE "D"

This bridge is described in Chapter 2.1.1, shown in figure 2.10, is stated to be the only example of a long-span composite bridge in Holland. Overall construction costs for this one-track railway bridge with a span length of 58.4 meters are said to be 5% higher than for the concrete alternative. The composite alternative was chosen (1974) because "a reduction of dead load was necessary because of the admissible foundation pressure" [2.37].

5.2.6 SUMMARY

From the cost data and composite designs presented in paragraphs 2.1 to 2.4 the following conclusions are made:

- The construction costs of composite alternatives are roughly the same as for the concrete alternative for both highway and railroad bridges. From table 5.1 total costs vary from a reduction of 8% to an increase of 14%.
- Construction costs for alternatives designed in the Netherlands may be lower than indicated if a thorough final design was completed.
- Costing estimate for composite bridges should improve (decreasing costs) as experience with composite construction is acquired.

Experience in neighbouring countries confirms that composite bridge alternatives have competitive initial construction costs for typical highway and railway bridges. Only in cases where special design criteria are present is the initial cost price substantially less for a particular option.

Not enough data is available to make clear comparisons for life cycle costs (maintenance, repair, renovation and removal) for individual bridges. Experience in many countries suggests that maintenance repair, renovation (such as widening) and demolition costs for composite alternatives are less than for a concrete bridge (Tschemmernegg, 1992).

5.3 REVIEW OF COST RELATED REFERENCES

5.3.1 INTRODUCTION

A review of references giving detail costing information for composite bridges was made. Emphasis was placed upon cost information relevant to the Netherlands. Many sources of information were found, however, most could not be made public. The remaining references are reviewed in the following paragraphs.

5.3.2 REFERENCE REVIEWS

As part of a thesis work cost comparisons were made for one span of a railroad bridge [Zimmerman, 1993]. The location of this bridge is the same as for bridge "A" in paragraph 5.2. The objective of this work is to determine the relative cost advantages (or disadvantage) of several composite cross-sectional types. These cross-sectional types are shown in figure 5.7.

In order to determine relative costs, preliminary designs were first performed for each cross-sectional types. The preliminary design is made based upon assumptions on the most appropriate ("best") fabrication, erection and construction procedure and shortest construction time. The basic design criteria for all calculations are listed as follows:
- Span length, 35 meters.
- Number of rail lines, 2. All rails are to be ballasted.
- Total deck width, 4.25 meters.
- Loads, standard Dutch rail loading (VOSB 1963).
- Design code, VOSB 1963 and VOSB 1990 codes.
- Special conditions; no limitations are placed on cross-sectional height, good water transportation is assumed and site conditions are considered to be of average difficulty.

A detailed summary of all the cross-sectional options for which preliminary designs were performed is listed in table 5.2.

Unit costs upon which a cost ranking of each cross-sectional option is based are listed in table 5.3. In this reference cross-sectional costs are first compared from the point of view of raw material quantities. Fabrication costs are then examined from the point of view of weld quantity. Transportation costs are compared based upon total bridge weight. Construction and erection costs are compared based upon bridge stiffness. Lastly, painting and maintenance costs are looked at according to the painted area. Cost components are weighted to enable an overall cost comparison to be made. These weightings are empirical in nature.

Those used in this reference are listed as follows:
- Raw Materials: Concrete 5%, steel 10%.
- Steel Fabrication: 30%.
- Transportation, erection and construction: 25%
- Maintenance: 35%.

Table 5.2 Summary of cross-sectional alternatives investigated.

Cross-sectional type	Alternative	Comments:
1		**Flat slab with two plate girders, bracing between bottom flanges**
	a	Plate girders with no web stiffeners: web thickness is 20 mm
	b	Plate girders with 1 web stiffener: web thickness is 12 mm
	c	Plate girders with 2 web stiffeners: web thickness is 10 mm
2		**Flat slab with one square box girder** *Note*: alternative web thickness are the same as for cross-section #1
4		**Haunched slab with two parallel trusses, bracing is provided between bottom flanges**
5		**Flat slab with one trapezoidal box girder** *Note*: alternative web thickness are the same as for cross-section #1
6		**Flat slab with four plate girders, placed directly under the tracks**
	a	Plate girders with no web stiffeners: web thickness is 19 mm
	b	Plate girders with 1 web stiffener : web thickness is 11 mm
	c	Plate girders with 2 web stiffeners : web thickness is 10 mm
7		**Flat slab with V shaped plate girders placed directly under the tracks**
8		**Double bridge: flat slab with one square box girder** *Note*: alternative web thickness are the same as for cross-section #6
9		**U-shaped haunched slab with rolled girders**
10		**Flat slab with parallel girders**
	a	6 plate girders (no web stiffeners and a 13 mm thick web) with a 320 mm thick slab
	b	8 plate girders (no web stiffeners and a 13 mm thick web) with a 270 mm thick slab
	c	6 plate girders (1 web stiffeners and a 8 mm thick web) with a 320 mm thick slab
	d	8 plate girders (1 web stiffeners and a 8 mm thick web) with a 270 mm thick slab
	e	6 rolled girders with a 320 mm thick slab
	f	8 rolled girders with a 270 mm thick slab

Table 5.3 Initial construction cost components for composite bridges.

Cost component	Comment
Design	Initial and final designs normally contribute 2 to 5% of a bridges total cost Composite bridge designs may initially require more man-hours and expert advice during conception.**
Raw materials:	
Steel	Steel tonnage normally contains only plate and/or hot rolled sections. Steel is priced at fl 1,25/kg*.
Concrete	Standard B25 or higher grades for cast-in-place slabs and the sub-structure. Concrete is priced at fl. 250,=/m3 *.
Reinforcement	Standard transverse and hogging moment reinforcement consisting of large numbers of straight components. Shear reinforcement is normally not used. Reinforcement price is included in concrete price.*
Prestressing	When used, straight tendons (not draped) are normally placed transversely or longitudinally in the slab.**
Connectors	Normal headed studs. Connector price is included in fabrication costs.*
Wearing surfaces	Standard.**
Painting	Standard.**
Site preparation	This includes access roads, temporary housing for labour and materials, foundation costs (exploratory drilling, pile installation and testing). These costs are unique to each bridge site.**
Steel fabrication	It is assumed that the fabricator shop fabricates, assembles and paints large steel units. In this case the following unit prices are used: Rolled steel beams (fabrication only): fl 2,--/kg Plate girders (cutting and fitting): fl 1,25/kg Welding: fl 5,--/kg The total painted area is used to compare painting costs.* The length of weld for each alternative per meter of bridge length is used to compare other fabrication costs.*
Precast concrete elements	When used these consist of 2 to 5 meter lengths of deck. Concrete and precast construction methods are standard but price may include transportation, erection, grouting between units, prestressing and measures needed to ensure composite action after installation.**
Transportation	Fabricated steel and concrete elements must be transported from the fabrication yard to the construction site. The total weight of each alternative per meter bridge length is used to compare transportation costs.*
Erection and construction	This includes the steel and concrete superstructures. Steel erection may be land based (cranes, launching, etc.) or water based, barges or pontoons. Slab erection may be in combination with the steel (erection of the composite span) or independent. If independent, erection may be cast-in-place with permanent or slip formwork. Precast elements may be lifted by crane or pushed into place. Actual prices may be effected by unforeseen adverse site and weather conditions. The stiffness (moment of inertia) of each alternative is used to compare erection costs.*
Secondary construction elements	Rails, ballast and rails or wearing surfaces must be installed. Crash barriers, pedestrian protection and lighting must be installed. Pipelines and other services may be installed.**
Total construction time	An important factor that must be included in cost estimates is time. Fabrication, transportation and erection procedures should be chosen not only as a function of material and man-hours. Interest payments, traffic disruptions, closures of waterways and other time related factors may have a major impact on the overall project cost.**

* Values used for these comparisons
** Not included in comparisons

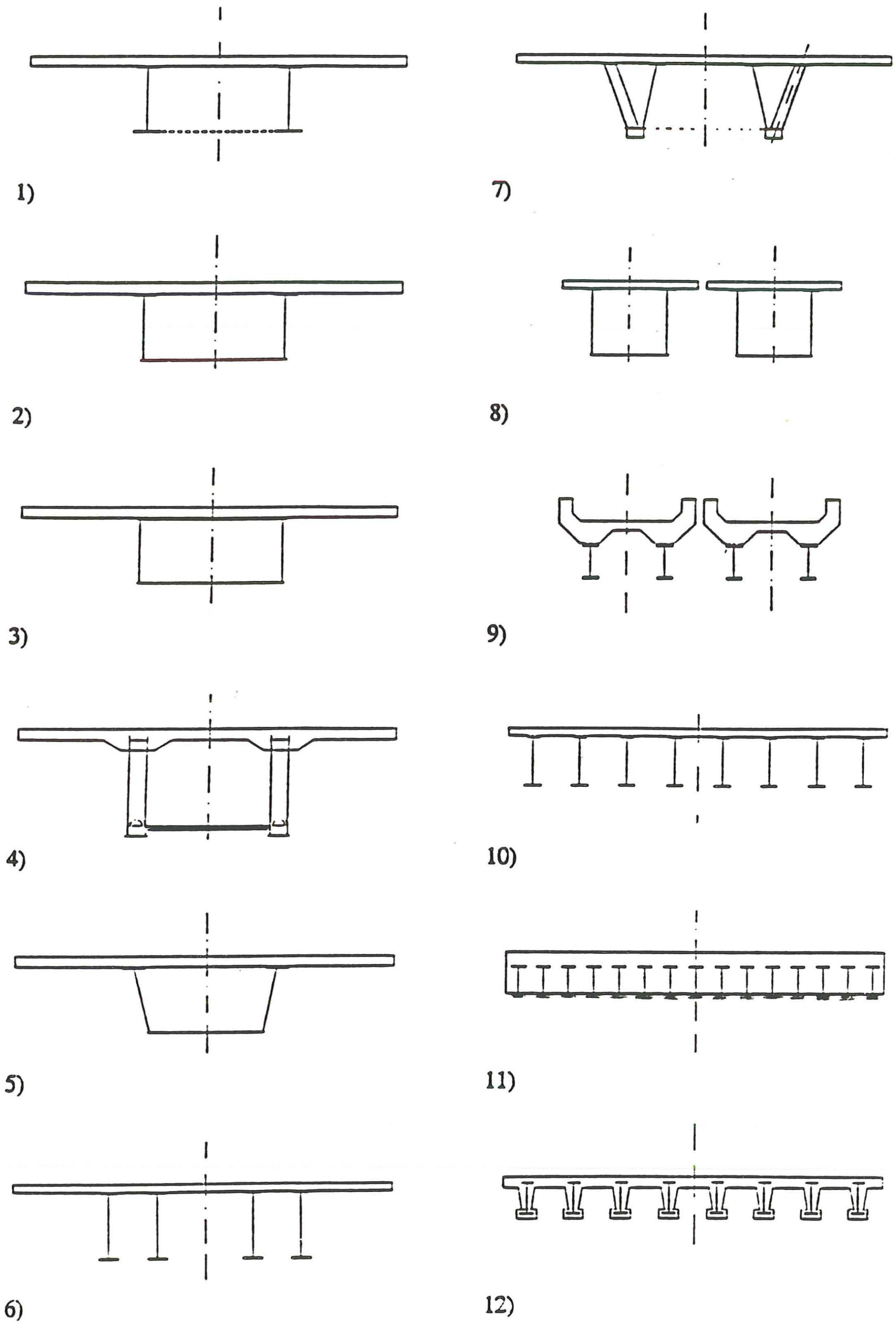

Figure 5.7 : Composite cross-section studied

Cost components are weighted to enable comparisons to be made between cross-sectional alternatives. The weighting factors used in this reference are listed below:

- Raw materials:
  - concrete 5%,
  - steel 10%
- Steel fabrication: 30%
- Transportation, erection and construction: 25%
- Maintenance: 35%

The conclusions of this study are presented in table 5.4. The cross-sections included in this table correspond to those listed and described in table 5.2. In this table weighted component costs and total costs are given. These values can be directly compared between cross-sectional alternatives. Important results from this comparisons are stated as follows:

- The least cost cross-sectional alternatives are #1, 2, 5, 9 and 10 "e" and "f". These are flat slabs with box girders, rolled girders with a U shaped deck and multiple rolled girders with a flat concrete slab.
- The highest cost cross-sectional alternatives are #7 and 10 "a" and "b". These are V-shaped plate girders and multiple plate girders on a flat slab.
- The weighting factors used for the cost comparison were varied be 5% to examine their effect upon cost conclusions. The result of this study indicates that the relative cross-sectional alternatives costs are not significantly effected.

Table 5.4 Cost comparison conclusions

Cross-sectional type	Design alter-native(s)	Materials: Concrete	Steel	Fabrication	Transportation, Erection, and Construction	Maintenance	Total
1		0.090	0.058	0.045	0.054	0.081	0.059
2		0.090	0.058	0.045	0.057	0.081	0.060
4		0.090	0.050	0.147	0.060	0.079	0.090
5		0.090	0.058	0.045	0.052	0.081	0.058
6		0.070	0.070	0.090	0.098	0.070	0.086
7		0.070	0.065	0.180	0.092	0.069	0.110
8		0.050	0.070	0.090	0.087	0.056	0.078
9		0.116	0.118	0.003	0.059	0.116	0.062
10	a	0.090	0.084	0.141	0.113	0.089	0.113
	b	0.075	0.090	0.188	0.151	0.080	0.138
	e	0.090	0.136	0.012	0.083	0.104	0.071
	f	0.075	0.144	0.016	0.095	0.096	0.075

Composite highway bridge const information has been published for short spans [Arbed, 1992]. This information has been reproduced in figure 5.8. It is stated that for a composite bridge with a total length of 550 m long the overall costs (superstructure and piers) are most competitive for span lengths between 20 and 33 m. For this comparison average pier heights of 6 and 15 meters were used. The composite cross-section consists of a flat slab, poured using metal decking and supported by hot-rolled beams.

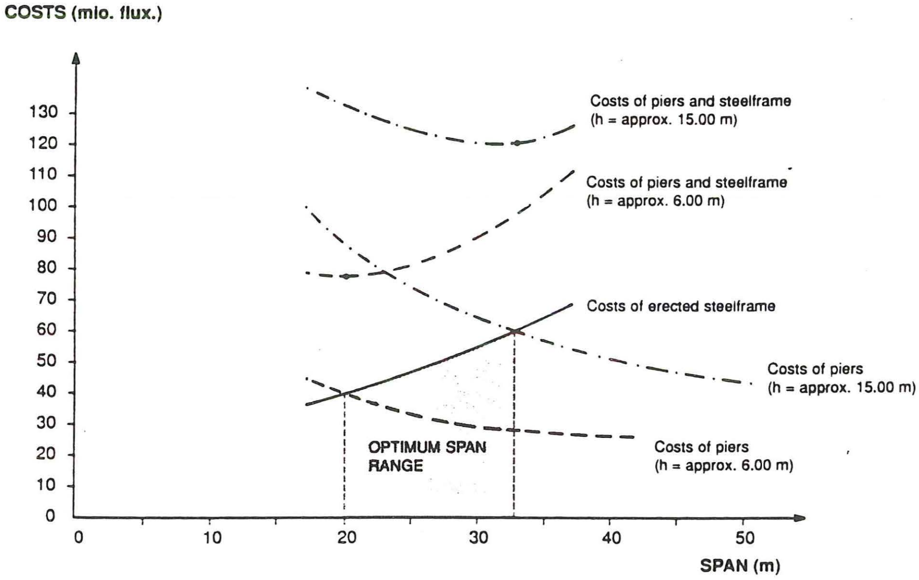

Figure 5.8 : Arbed cost estimations for multi-girder composite bridges as a function of span length.

Cost comparisons have recently been completed for a short span highway bridge in the United States [Rubeiz and Rogers, 1992]. Bids were let for the replacement of an existing bridge consisting of simple spans of about 17 m each. Prices for seven bids are given for the following bridge types:

- Precast concrete. The superstructure consists of eight precast prestressed concrete box beams with a concrete wearing surface.
- Inverset system. This is a patented precast precompressed concrete / steel composite structural system. A modular concrete deck unit is shop-cast upside down in forms suspended from the steel beams. Each unit contains two beams. Before the concrete is cast, the beams are inverted and supported near or beyond the end of the forms. When the units are turned over the entire composite section becomes available to resist dead loads.
- Conventional composite. This alternative consists of hot rolled beams and a composite cast in place concrete slab.

The results of this comparison indicate the following:

- The inverset system has the highest initial costs. These could be offset in some cases due to a substantial decrease in construction time. For this bridge construction time could be reduced by 10 days from 55 to 45 days.
- Precast concrete and conventional composite bridges have about the same initial construction costs.

The conventional composite bridge was chosen because "the steel superstructure is easier to inspect and to maintain".

Private discussion with French designers and practicioners were held. Important points from these discussions are summarised as follows:

10 to 15 years ago the situation in France with respect to composite bridges was similar to that in the Netherlands today. To change this situation the French steel industry exerted considerable pressure at a high governmental level to change the way in which the design process was conducted. This pressure lead to new procedures, in which steel (including composite) and concrete alternatives must be allowed for most bridges. In was observed that this resulted in lower costs for all bridge types (steel, composite and concrete), due to increased competition. For comparison purposes a simplified flow chart showing the new design and construction process in France for state-owned highway bridges is schematically shown in figure 5.9.

Parallel to this initiative by the French steel industry, a major information campaign was undertaken to educate designers so that they could economically design using steel. An example of the resulting publications is given in reference (Ponts mixtes..., 1985).

In France today there is fierce competition between concrete and composite highway bridges for short span lengths (up to 40 m). In general, composite highway bridges are most economic for span lengths between 40 and 120 meters (medium span lengths). For long span lengths, designs tend to reflect special site situations and thus it is difficult to generalise about prices advantages.

80% of medium span composite highway bridges in France consist of two parallel plate girders with lateral bracing at or near the bottom flanges. This is similar in geometry to cross-sectional type 1 in figure 5.7. The steel girders are nearly always continuous. Typical deck slab thicknesses are between 25 cm and 30 cm. When the bridge carries more than two traffic lanes the deck is usually prestressed transversely to minimum concrete thickness.

The majority of new, short and medium span, composite railway bridges are ballasted. Typical cross-sections consist of either a flat or U shapes concrete slab, placed upon a steel caisson. This is similar to the steel part of cross-sectional type 8 and the concrete part of cross-sectional type 9 in figure 5.7. Steel caissons are chosen because they are the least expensive method providing sufficient torsional stiffness.

Some France unit costs were obtained are listed in table 5.5. All costs have been converted to Dutch Guilders. Typical French requirements and practices for composite bridges are listed in table 5.6. Lastly, a cross-sectional alternative with considerable potential for two-lane short spans in flat terrain was identified. This alternative enables the overall height to be considerably reduces, thus reducing the length of approach spans, and is shown in figure 5.10. Steel fabrication is simple and consist of many part that may be mass produced.

Table 5.5 Some unit costs in France for composite bridges

Item	Cost	Comment
Steel	fl. 1070,-- to fl. 1170,--	
Steel fabricator	6 to 7 man-hours/ton	Hour rates varies depending upon plate thickness, weld quantity and type
Concrete	fl. 130,--/m^3	
Concrete pumping	fl. 10,--/m^3	

Table 5.6 Typical French requirements and practices for composite bridges

Item	Comment
Static system	Nearly always continuous
Concrete slab	Transverse prestressing to reduce slab thickness if necessary Longitudinal prestressing avoided if possible Negative moment cracking is limited by differential support movements or modifications to the concrete schedule
Material requirements	Steel must come form an approved supplier. Approval covers the following items: - Quality assurance - Weldability (Equivalent carbon content) - Thickness and straightness - Minimum impact energy: t < 30 mm 40J at -20oC 30 mm < t < 80 mm 40J at -20oC 80 mm < t < 120 mm 27J at -50oC
Reinforcement	Minimum yield stress of 400 N/mm2 and a minimum concrete cover of 3 cm are required. Note: maximum crack widths are not given in France, however, a minimum reinforcement percentage and maximum reinforcement stress are specified.
Concrete	Free use may be made of grades B30 and B40 concrete. Restrictions are placed upon the use of B60 concrete. No design fatigue requirements for the concrete slab are specified. The following points must also be observed: - A limitation is placed upon low ate shrinkage - The water/cement ration is limited to 0.45 - Limits and requirements are place upon plastifiers - Concreting cycles are limited to 48 hours. - Concrete strength must be greater than 15 N/m^2 before formwork can be removed.

Inition of project by the state

↓

Appointment of a "Maître d'Ouvrage"
- Private design office
- CETRA

Two designs are normally made: one in steel (or composite) and one in concrete. In some cases, such as long span structures, this rule may be broken.

A list of allowable changes to the design documents is drawn up.

↓

Official offer is announced and bids are requested
Each bidder must perform full structural calculations based upon the offer and list of allowable changes from the official design. The bidder is responsible for the full structural integrity of the design.

↓

Designating of the bid winner
The Maître d'Ouvrage checks the designs submitted by the bidders.

The state and Maître d'Ouvrage decide upon a winner by establishing the bidder who has the best price for the best quality (ratio of price to quality)

Figure 5.9 : Flow chart showing the new design and construction process in France for state-owned highway bridges

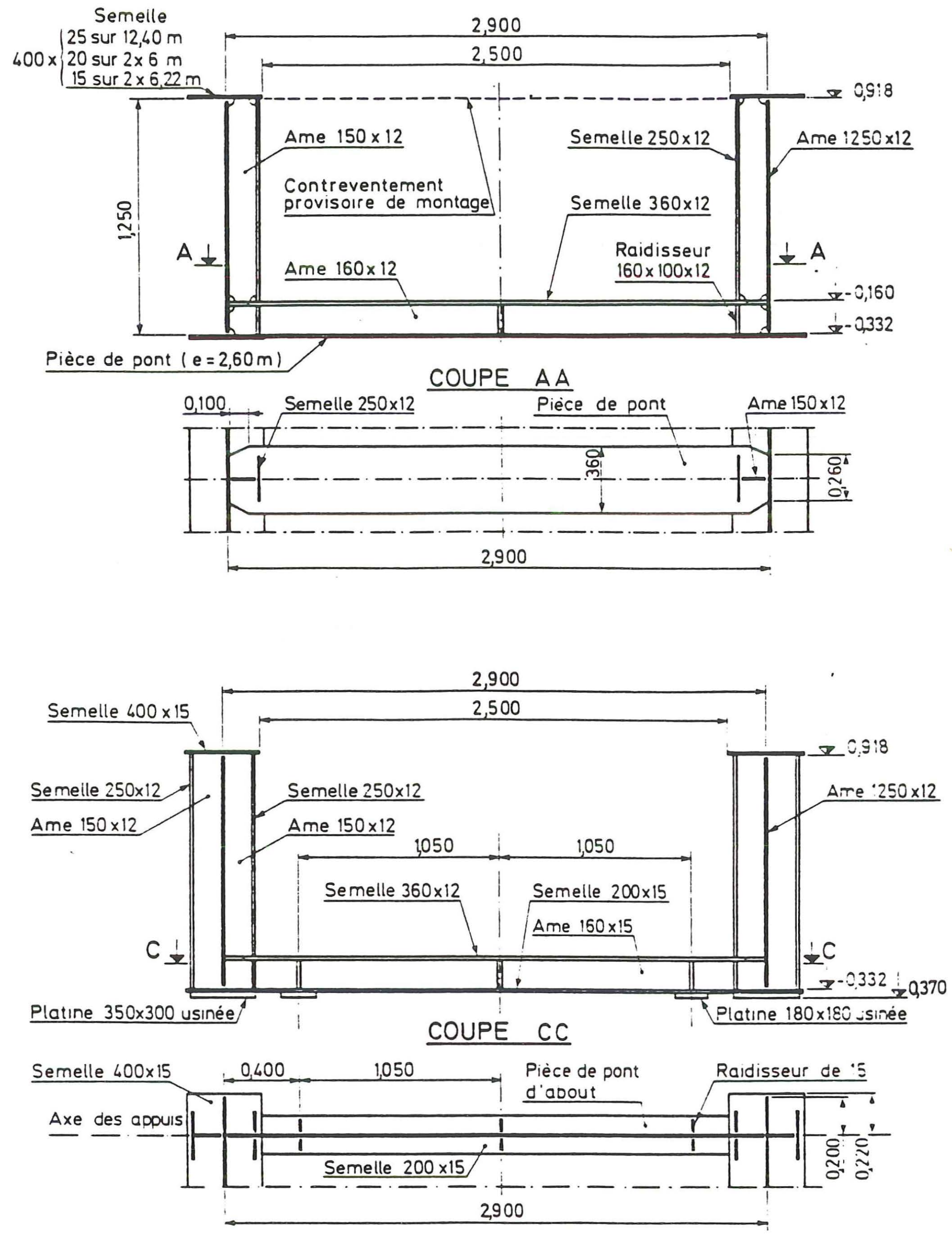

Figure 5.10 : Example cross-section for a low height French composite bridge alternative

5.4 CONCLUSIONS

Two methods of comparing costs for composite bridges have been presented. First, comparison of average contractor bids for concrete and composite alternatives in the Netherlands. Second, costs comparisons based upon available literature has been reviewed. It would appear that composite construction is a competitive form of construction when compared with concrete. This is equally true in the Netherlands and in many other countries.

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