bulletin 9

guide to good practice

fib CEB-FIP

Guidance for good bridge design

Part 1
Introduction
Part 2
Design and construction aspects

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Guidance for good bridge design

Guide to good practice

prepared by ***fib*** Task Group 1.2 *Bridges*

(former FIP Special Commission on Bridges)

July 2000

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category	minimum approval procedure required prior to publication
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State-of-Art report	approved by a Commission
Manual or Guide (to good practice)	approved by the Steering Committee of *fib* or its Publication Board
Recommendation	approved by the Council of *fib*
Model Code	approved by the General Assembly of *fib*

Any publication not having met the above requirements will be clearly identified as preliminary draft.

A first draft of the guide had been reviewed in July 1998 by the former FIP Editorial Board. Recommended for publication subject to some amendments, these were introduced in the meantime by *fib* Task Group 1.2 *Bridges*, which started as a former FIP Special Commission with the same title. After the 1998 merger of CEB and FIP into *fib*, this guide is now published in the new *fib* series of bulletins.

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Foreword

The Fédération Internationale de la Précontrainte - FIP - decided to set up a special commission devoted to design and construction of bridges. Its work began in June, 1993, with a limited number of experienced engineers to prepare a document on bridge design.

Our intention, initially, was to issue recommendations to designers in order to help them improve the quality of their projects in all aspects including aesthetics. But we very soon discovered the most important help we could give was not in this field at all.

We discovered that many problems concerning bridge design, particularly those related to creativity and innovation, to aesthetics and elegance and to many other elements of the global performance, as well as the quality of the detailed design itself, could come from the selection of the designer according to inappropriate criteria and also from the contract conditions. Clearly, a good design must be paid for at its real cost ; economising on the design cost can be extremely counterproductive for the owner when considering the final whole-life cost of the project. In addition, we considered it very important to address the designer's responsibilities and relations with other participants in large projects and finally design philosophy itself. As the preparation of a complete document on bridge design, including these major aspects as well as more technical ones, would have taken a very long time, we decided to work by successive steps. We concluded that the most urgent need was to address these major professional problems, which are not covered by existing documents or the activities of international associations.

The first part of this report is our tentative answer to these problems. It is addressed to designers, of course, but even more to Owners and Project Managers as a guide to the efficient selection of designers and contractors, and to the preparation of fair contracts providing high quality at reasonable cost.

The second part, more technical and mainly addressed to bridge designers, is devoted to a systematic analysis of structural and constructional bridge concepts: erection techniques and their influence on design, organisation of cross-sections...

Together they form the first volume of what was to become the **FIP** Guidance for a good bridge design. As with the merger between FIP and CEB, the FIP Bridge Commission has been integrated in a new Commission with a wider scope: Commission 1 - Structures; the document has been finalized through the activities of the new ***fib*** Task-Group 1.2. The first volume of the guidance to good bridge design is published in the name of the new association, the Fédération Internationale du Béton, ***fib***.

This document may be considered as being based on European experience only, since all authors are from Western Europe and have taken many of their examples from this part of the world for practical reasons. However the problems discussed in the different chapters are international, and the best technical solutions are the same, or almost, all over the world. If necessary, and if this guidance to good bridge design is considered useful, it may be complemented and amended in the future.

Finally, it should be understood that guidance like that included in this report has to take very clear and strong positions, some of which may be challenged by those who have different opinions. We have tried our best to give a balanced presentation of all the problems and systems which exist in different countries, but at the same time we could not weaken our analyses and have had to criticise some methods or tendencies which lead to low quality and poor designs. We have attempted to present clearly our ideas and proposals which we consider are keys for producing well-designed bridges.

Bonnelles / Genève, June 2000

Michel VIRLOGEUX
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Chairman of ***fib*** commission 1 "Structures"

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PART I

1 Objective of a good design

1.1 Introduction

For Marcus Vitruvius Pollio, a military engineer and architect who served under Julius Caesar, the art of building is characterised by three main principles: *firmitas* (strength / safety), *utilitas* (suitability / serviceability or functionality) and *venustas* (elegance / beauty) [Germann 1979 – 1.1]

After more than twenty centuries these principles are still valid although the search for economy is more important today, perhaps to too great a degree. According to David Billington [1983 - 1.2] the fundamental objectives of a good design are Efficiency (firmitas / utilitas), Economy and Elegance (venustas); these constitute the leading ideals of what he calls "Structural Art" a new form of art born with the industrial revolution.

This clearly shows that, to achieve a successful bridge design, engineers must not only optimise the safety, serviceability and economy of the structure to be built, but also its aesthetics. It must be emphasised that designing a bridge for purely economic and technical purposes and then attempting to improve the appearance by minor shaping or ornamentation is a mistake. On the other hand, selecting shapes for pure fantasy and appearance without any structural logic, and then trying to amend the design to balance loads and effects is a pure nonsense.

A beautiful design can only be achieved if the aesthetics is developed from the beginning as an essential part of the global structural concept, expressing a logical and simple flow of forces.

Figure 1.1: "Pont du Gard" Roman viaduct

1.2 Efficiency

1.2.1 Functionality

Efficiency begins by satisfying the functional needs of the bridge which is to be built to create new connections. These needs must be carefully analysed, with appropriate weighting given when they conflict.

Safety of the different types of user must be considered and means provided to protect them as much as is reasonably possible against accidents, aggression and even in some cases against severe climatic conditions.

However, the new construction should not alter the operation of other infrastructure ; it must respect the constraints of the roads, railway lines and navigation channels which pass below or above; and the life and activity going on in the vicinity. *Utilitas* for the new construction, without altering the *utilitas* of the existing transport systems, industries and habitations.

1.2.2 Design

Bridge design is essentially a solution to an engineering problem where safety and serviceability play decisive roles.

Bridges must efficiently support the estimated loads and provide adequate factors of safety against instability or material failure, without excessive deflection or vibration which might lead to degradation of the structure cause, alarm to the users or interfere with serviceability.

Figure 1.2. Salginatobel Bridge – The art of Robert Maillard

Structural forms must be selected for this purpose, and thus be appropriate for an efficient transfer of loads, from the points where they are applied to the supports and foundations; for an efficient use of materials and of their strength, deformability and other mechanical characteristics; and for an efficient use of the chosen erection techniques and equipment.

In addition the design, including the selection of materials, must be such that it does not suffer significant loss of strength within its projected life due to corrosion or other long term effects.

Finally, engineers must be conscious that safety comes much more from the coherence of the global structural concept, i.e. the efficiency of the organisation of the flow of forces, and good construction than from fulfilling code requirements which by themselves cannot assure the required safety. Codes codify safety; good designs provide it.

1.3 Economy

Cost is an important factor; but too much importance has been given to it at certain times, and this tendency is again increasing. In addition economy cannot be judged on the basis of construction cost alone but on the whole-life cost, that is the total cost of construction, maintenance and use.

To achieve an economic solution it is important to choose the most appropriate structural form and erection system taking into account all the limitations imposed and opportunities offered by the nature of the soil, the local topography and environment, and the functional constraints and requirements. As the cost of a bridge is the sum of materials, manpower and construction equipment, it should be borne in mind that the best economic solution in one location may not be the same as that at another location.

The costs of maintenance and operation can be reduced by giving particular attention to providing easy access to all parts of the bridge and especially to the most vulnerable parts (inspectability); to providing easy replacement of bearings, joints and other equipment without interruption of traffic (maintainability); and to providing easy - or at least as easy as is reasonably possible - strengthening, upgrading or modifying of the structure (adaptability).

Among all structural materials, prestressed concrete, when properly used, is certainly the best for durability and economy in a very great number of countries and situations.

1.4 Elegance

As already stated, cost is important but is not the only decisive factor. What we build should not only demonstrate the designer's skill and creativity, but also respect the environment and the public desire for elegant structures. Engineers must be aware that their constructions will form part of their surroundings for decades; therefore they should not affect them adversely and where possible they should be a positive improvement. At least, this should be true for the best creations of the new Structural Art.

Figure 1.3 Garabit Viaduct – Masterpiece of technique, beauty and elegance [1]

This clearly means that engineers have the responsibility and the duty to produce beautiful bridges at a reasonable cost. Beauty is a significant demand for man's happiness.

Bridge aesthetics have two different aspects: the integration of the structure into its surroundings, either urban area or natural site, and the intrinsic bridge architecture itself.

1.4.1 Integration into its surroundings

Integration of a bridge into its surroundings depends very much on the site itself.

a. Natural site

Integration is much simpler into a natural site, especially when it is beautiful and has a strong or clear character.

Firstly, careful attention must be given to the bridge location taking into account the road alignment and profile. When the site is highly variable, there must be a clear logic for the location selected. The level of the bridge in the site is of major importance for its appearance. The quality of the line which the road forms - in plan and in elevation, resulting in a curve in three dimensional space - is decisive for the bridge aesthetics. Of course, the selection of these parameters must be carefully made in collaboration with those from other disciplines, mainly road engineers. Then the designer must select the structural type, the location of supports and all the other main parameters having regard to the site topography, the landscape and the soil conditions.

Arches and bridges with inclined supports which produce arch effects are well adapted to V-shaped valleys with steep slopes when the rock is strong and near the surface. Cable-stayed and suspension bridges with long approaches are very well adapted to crossing a navigation channel, a sea channel or a navigable river in a flat plain. Repetitive spans give an impression of tranquillity in a quiet place when the spans are well proportioned to pier height.

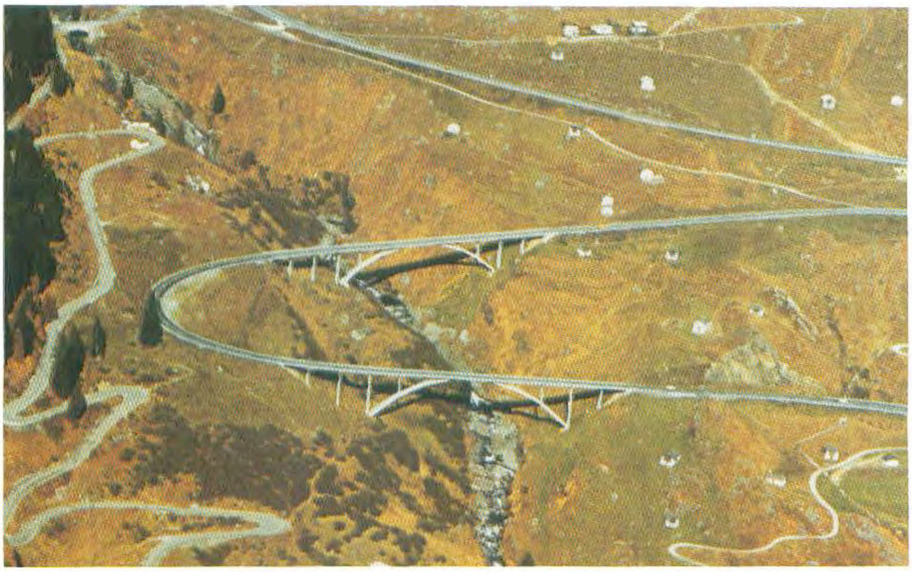

Figure 1.4 Cascella and Nanin arch bridges on the St Bernardino Pass [17]

Bridge size is of major importance. Large bridges, particularly with very long spans, have a majesty all their own which makes them very attractive when they are properly designed.

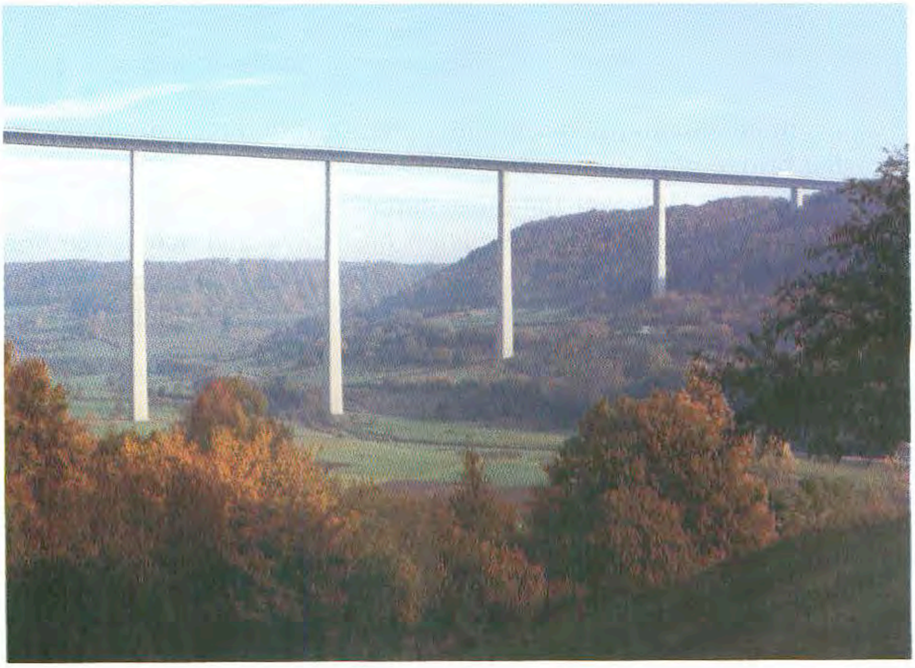

Figure 1.5:Kochertal Bridge [5]

b. Urban bridges

At the opposite end of the spectrum are bridges built inside a town, with either modern or traditional architecture. Local sensibilities can be decisive in the selection of the design, and no rules can be given.

In some towns, even those rich in historical monuments, modern bridges will be preferred for their simplicity, i.e. their straight lines which are not in opposition to the traditional architecture, just as many art collectors mix ancient and modern furniture and paintings in their houses. In other places modern designs would be considered totally unacceptable and would provoke strong opposition; new bridges are then designed with extremely traditional shapes, mainly dating back to the 19th century.

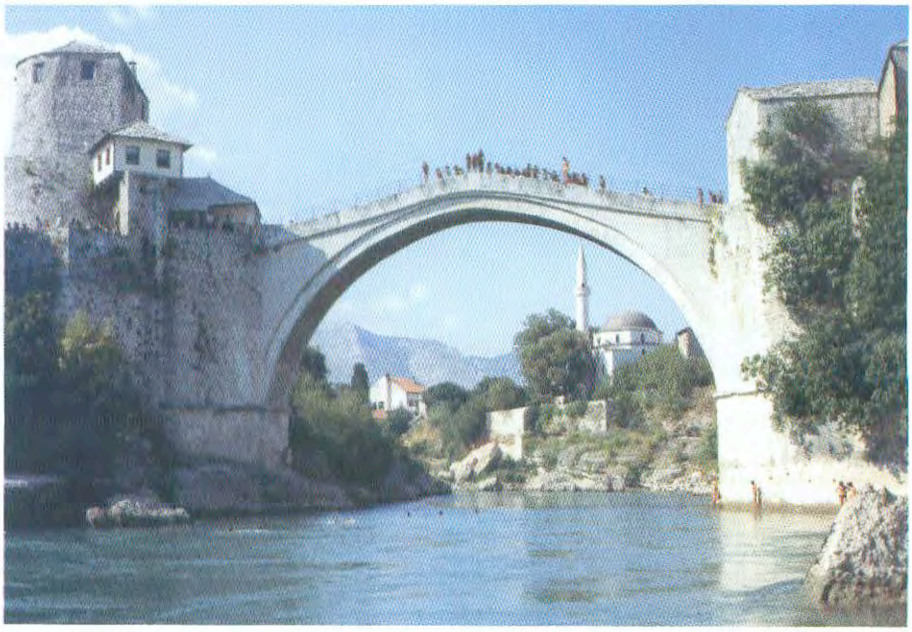

Figure 1.6: Bridge across the Neretva in Mostar – Yugoslavia [5]

In all situations, careful attention must be given to details, to the connection of the bridge with the existing infrastructure, to the quality of materials and even to decoration. However the most important aspect is the way in which the structural solution caters for the circulation on, below and across the bridge of pedestrians, cyclists, cars and buses. Pedestrians and cyclists should travel safely and be protected as much as possible from accidents and aggressions. Utilitas is very important here.

Figure 1.7: Mirabeau Bridge – Paris [1]

c. Suburban, industrial and other areas.

Between these extremes are bridges located in suburban and industrial areas, where many transport systems exist and cross each other. Often the main problem here is to give a unity to the bridge in a place where none really exists.

1.4.2 Aesthetics

Bridge aesthetics do not differ very much from traditional architecture, and clear recommendations have been given by the classical authors [Leohnardt 1984 – 1.3], [Menn 1986– 1.4], [Transportation Research Board 1991 – 1.5]. However for bridges, the main aspects governing aesthetics are the structural concept and the simplicity and efficiency of the flow of forces.

a. Proportions

Good proportions must be achieved between the various elements of the structure, height and thickness of piers, depth of the deck and length of the spans, height of piers and span length, mass and voids, light and shadow. Good proportions are a key to achieving an architectural success.

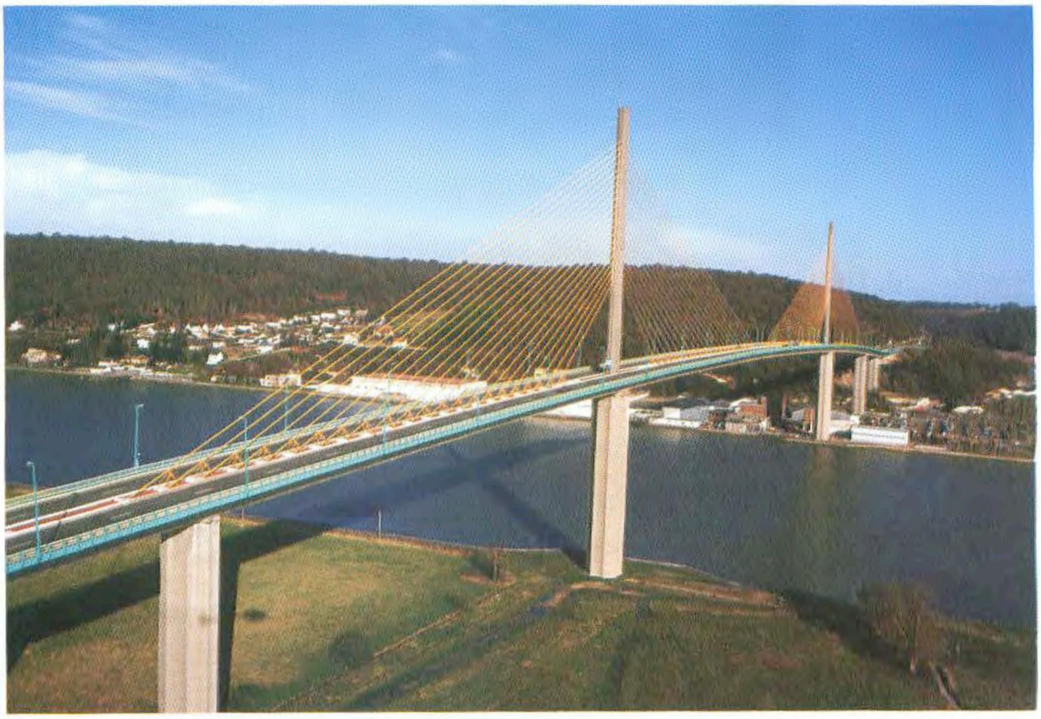

Figure 1.8: Brotonne Bridge, the right choice of proportions [4]

b. Transparency

Transparency is also a major goal: the view underneath the bridge must remain open even from oblique angles. Transparency is influenced by the number and shape of the piers; the relation between the width of piers and the span length plays a major role.

c. Slenderness

Heavy structures are inelegant. Thus the slenderness of the bridge deck - beam depth to span length - and of the piers when they are high, is an important criterion affecting the bridge appearance. However, excessive slenderness must be avoided not only due to the discomfort that can arise from the dynamic behaviour, but also because excessively slender members may give the appearance of being too weak for their task.

Figure 1.9: Hennebique Bridge, elegance and slenderness [2]

d. Unity and harmony

Unity and harmony between the different bridge elements are needed to achieve a pleasing appearance. The best results come not only from simplicity in the design and clarity of the structural concept, but also from the coherence between the different elements and their shapes. The bridge must be homogenous, harmonious and easily understood by observers.

Figure 1.10: King Tai Bridge - China [1]

e. Details

Careful attention must be given to details and secondary elements because they are of great importance visually as well as functionally. For bridges which are mainly seen from long distances, dimensions and proportions are most important; there is little justification for investing in fine details such as the treatment of concrete surfaces. The opposite is true for urban bridges and for all bridges which are seen by many people close up.

f. Colour

Colours can play a significant role in bridge aesthetics. When used properly, colours can accentuate or subdue the visual impact of structural elements. Warm colours tend to accentuate the presence and size of forms, whereas cool colours diminish the visual importance of the elements to which they are applied.

g. Materials and equipments

Aesthetics are widely influenced by the quality of the selected materials. The quality of concrete is of primary importance, not only its colour but also its degree of compaction which governs the surface appearance and the durability of this appearance. The quality of ducts in cable-stayed bridges, and cables in suspension bridges are also important. Low-quality materials immediately degrade the bridge image.

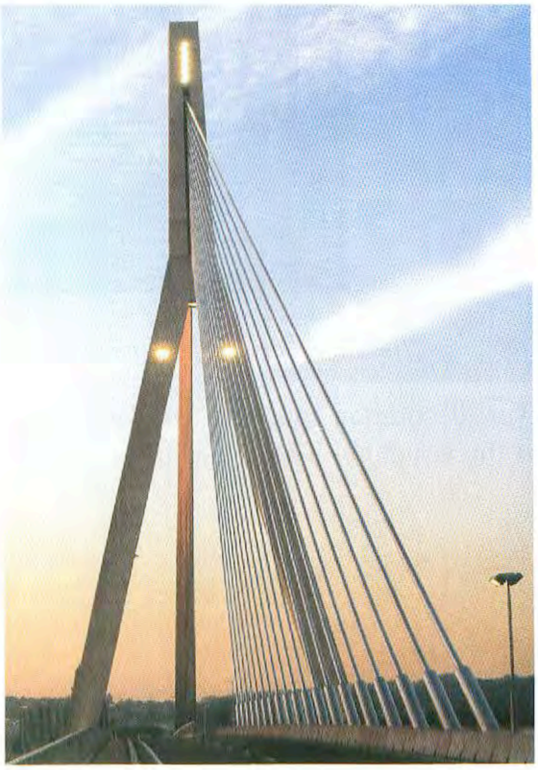

Figure 1.11: Wandre Bridge, pureness of details given by stainless steel cable ducts - Belgium [2]

Bridge fittings and equipment must also have a good quality appropriate to their purposes e.q.: cornices, barriers and hand-rails. Poorly designed, or built with low-quality materials, these can seriously degrade the bridge aesthetics. Bridge fittings and equipment should be considered from the beginning of the design; the later addition of new equipment is very often a disaster.

h. Decoration

Decoration can add to the appearance of a bridge, mainly in urban areas, if it is in agreement with the structural concept and helps to express it. Many examples could be given of decoration which has improved a bridge and made it particularly attractive.

Figure 1.12: Alexandre III Bridge, Paris A beautiful example of bridge decoration in urban areas [1]

However, decoration must be avoided when it is totally artificial, bearing no relation to the structure. There is a fine line between decoration and a ridiculous addition.

1.4.3 Conclusion

Elegance results from many sensible decisions made by the designer, but the selection of the structural concept, i.e. the organisation of the flow of forces, is the major consideration which governs all others.

Figure 1.13: Normandie Bridge – When bridge design leads to structural art [4]

2 Co-ordination with other aspects of the project, other than structural

2.1 General concepts, elements of the route

The planning of a route for a highway or a railway should be considered a true art as the projected route must fulfil many criteria related to user comfort, speed of use, security, harmonious integration into the surroundings, etc. A good route is often the result of long experience acquired over time and of the lessons learned during previous projects.

In the past, routes were planned by public works engineers who were also responsible for engineering the structures. The layout of the route was, therefore, largely influenced by the obstacles to be crossed, which resulted in the use of numerous transition curves to place the route perpendicular to obstacles in order to cross them using the shortest spans. Other considerations, usually of a more economic nature, encouraged the optimisation of the route in elevation in order to balance the cut and fill, which often led to longitudinal sagging curves, a configuration which is particularly detrimental to the aesthetics of a structure.

At the present time, the problem is much more complex, especially due to the intense urban activity which cuts across transportation routes, to the multiplicity of underground public service networks and, most particularly, to the changes in philosophy and priorities in the criteria which determine the route. User comfort, speed of use, security, traffic flow, the harmony and diversity of the route, economic considerations, maintenance and operational requirements are, in addition to environmental protection and urban or natural integration, the principal criteria which determine a route. Each discipline has become more and more specialised, and the study of a project necessitates a multidisciplinary team made up of specialists from each field. This team must work together in perfect symbiosis from the beginning of the project, as the contribution of each member is influenced by the constraints and limitations arising from the other disciplines.

The systematic planning of a route for a highway taking account of all the constraints lends itself perfectly well to a computerised treatment which permits direct visualisation by a three-dimensional representation of the route. This approach also permits the location and optimisation of traffic signal controls as a function of user visibility and, in particular, permits the successful treatment of delicate transition zones such as tunnel entrances and exits. In effect the planner can produce a visualisation of the highway from any point along its route which allows him to check out all the parameters which influence the scheme. Energy consumption calculations as a function of the route may also be carried out at the same time, and can be used as one of the criteria to choose between variants.

2.2 Where do bridges fit in?

Engineering structures form an integral part of the route and must, therefore, fulfil the same criteria of user comfort and security. Bridges must be designed in an optimal manner to ensure their usefulness. They must respect the flowing nature of the route geometry, which implies that they often have to accomodate curvature in plan, skewed abutments and vertical transition radii. Respecting the layout of the route in this way is often beneficial to the aesthetics of the structure and facilitates its integration into the surroundings.

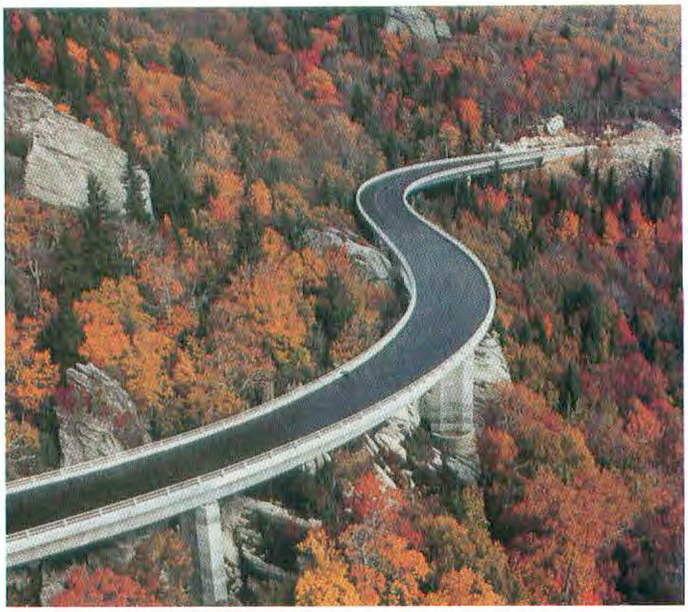

Figure 2.1: Linn Cove Viaduct: Harmony between route and structure [8]

However, the structure which crosses an obstacle must not become a slave to the highway geometry. The bridge engineer must be an integral member of the team of specialists cited above and must participate in the project from the first pencil sketch of the route. Certain basic ideas presented in Section 2.4.1 should be respected and taken into account when determining the geometry of the route. In all cases, the bridge designer must react strongly to certain abuses of power which could shift the project toward solutions which are contrary to the structural logic of the bridge.

To cite an example, the planning of a motorway in Switzerland foresaw several crossings of a large river. The study of the route was carried out according to current practice, despite certain difficulties to solve aspects concerning land use and ecological and forest protection. However, in an industrial site, the highway geometry required the crossing of the river at an angle of 12°, transforming an ordinary crossing with a span of approximately 55m into a truly awesome crossing of more than 270m! The bridge is situated very close to existing ground level, and the local authorities responsible for the river strictly limited the possibilities of pier placement in the river bed. The result was obviously very costly and, due to its lack of logic, the structure is not well integrated in a valley landscape which is already very cluttered.

In this example, an enlightened intervention by a competent bridge designer would certainly have permitted a slight modification of the route in order to avoid these problems and may even have led to decreased cost.

A second, more common, example shows the danger of maintaining certain rigid ideas or concepts without specific analyses for each case. This example concerns the construction of a cantilever bridge, situated in a deep valley, which required very high piers of approximately 130m. In this situation, the wind load was very critical, especially during the construction phases of the structure. In order to ensure better management and maintenance of such structures, but also in order to limit the risk that the bridge must be closed in cases of serious highway accidents, most authorities impose a solution of twin bridges which are totally independent of each other. It is clear that in most common cases this is a reasonable and judicious solution. However, it is also clear that in the case of such high cantilever bridges, this solution could be reasonably abandoned in favour of a single box girder with a single pier, which would provide greater structural rigidity against wind. In the present case, this dispensation was not possible, and special costly fittings, such as temporary cable stays and transverse girders between the piers were necessary. It goes without saying that the aesthetics and overall visual balance of the structure were strongly affected, which is regrettable for a bridge which should have been a masterpiece, given its situation and importance. Here again, the intervention of a competent bridge designer would have been able to convince the decision makers to relax some of their requirements.

Figure 2.2: Nantua Bridge – France, before and after construction of the second bridge [6]

It is clear that bridge engineering is not an independent discipline at the service of highway planners but that it must be included in the team at the beginning of the project in order to influence basic decisions.

Figure 2.3: Aesthetical study for the Cheviré Bridge – single or double piers [7]

2.3 The different disciplines involved

Depending on the scope of the project, the complete study of the route and the conceptual design of the structures require a significant team which should be composed of the specialists listed below. This non-exhaustive list should be adjusted for each specific project. In the case of construction or replacement of a single structure, only a limited number of specialists would be involved.

The persons involved in the construction:

- representatives of the owner
- engineers for the planning and management of transportation systems
- engineers specialised in highway construction
- bridge engineers
- contractors
- geo-technical engineers
- personnel responsible for structural maintenance and management
- architects and urban planners
- electrical and mechanical engineers
- engineers specialised in hydraulics and hydrology (in the case of a waterway crossing or coastal structure)
- firms specialised in environmental studies and analyses of environmental impacts (construction and final phases)
- acoustic engineers (for urban sites)
- landscape architects

External specialists:

- police services (in the case of traffic restrictions or limitations)
- sociologists (in the case of crossing densely populated areas or of connections between neighbourhoods)
- forestry engineers
- wildlife specialists (for special installations for route crossings)

The contribution from these different specialists is very variable. Their decision making powers and their ability to impose their wishes must also be carefully weighed, which, in general, is regulated by the owner after consultation with the principal specialists.

2.4 Interaction with the bridge designer

The interaction between the bridge designer and the other specialists varies between disciplines. The dialogue with the geo-technical engineers, road construction engineers and electrical and mechanical engineers should be continuous and close during both the project phase and the construction of the structure.

On the other hand, while discussions may seem less relevant with planners and those specialists who establish the route, they are none the less important. Good interaction with the bridge designer should take place in a group environment from the early stages of the route study in order to avoid certain errors from a structural point of view being made.

These potential errors concern not only the location of the route but also its longitudinal transverse sections. A brief description of some basic criteria which should be respected and that the bridge designer should stress during the discussions on the choice of the route is given in the following sections.

2.4.1 Plan view of the route

Extreme situations, such as those which occurred in Switzerland and which were described in Section 2.2, should be avoided by modifying the geometry of a section of the route. A skew which is too great leads to structural complications as well as having a negative aesthetic effect which may be reduced by a modification of the route. However, the smooth-flowing line and visual harmony of the route should not be sacrificed in order to cross an obstacle at a right angle. A compromise between the visual harmony of the route, user comfort and a reasonable and aesthetic structure should be found which does not unduly reduce the design speed of the new road (figure 2.4).

Figure 2.4: Compromise between visual harmony of the route, user comfort, equilibrium and aesthetics of the structure. Daillart bridge - Switzerland

Structural simplicity can be achieved by limiting the number of directions of the structural lines, which respects a principle of order and accentuates the harmony and calm of the bridge helping its integration into the surroundings. Therefore, for a skewed bridge, the lines of the transverse elements should be parallel to the obstacle being crossed. For rivers, this principle implies that the piers should be parallel to the current, which gives them a favourable hydraulic profile and limits scouring. In the case of small skews, it is possible to design a straight bridge with central piers by integrating the abutments into the river banks.

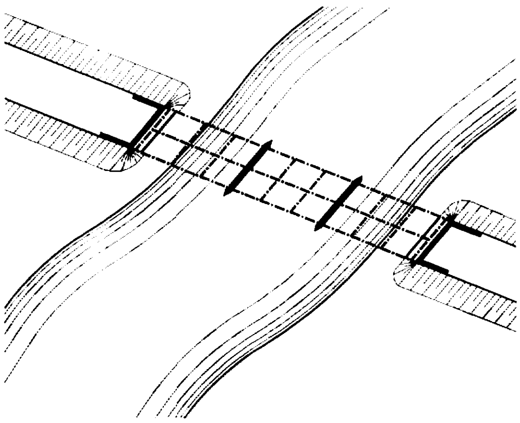

Figure 2.5: Skew bridge over a river, piers and abutments parallel to river direction [9]

If the bridge is situated on a curve it is necessary, as much as possible, to arrange that the transverse elements be perpendicular to the roadway axis. The layout of the piers should follow the same principle (figure 2.6).

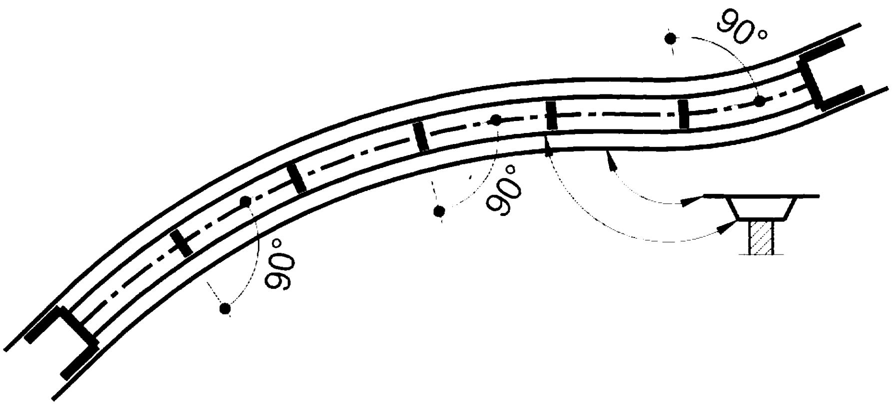

Figure 2.6: Bridge in a curve, direction of bridge lines and arrangement of elements

In all cases, abrupt changes in direction or radius of curvature should be avoided. The edges of the structure should be continuous and follow the roadway axis to avoid giving any perception of rupture or interruption of the route.

These considerations are not only applicable to the layout of the structural elements. All the external and visible structural fittings, such as security railings, technical installations for meteorological measurements, etc. must follow the same rules. For urban sites, it may also be necessary to equip the bridge with acoustic barriers. The studies of the actual wall efficiency must be carried out with its collaboration with an acoustical engineer whilst and the installation must follow the general principles of the structure described above. Special consideration in conjunction with architects and urban planners must be given to these elements in order to avoid their being an eye-sore.

Figure 2.7: Bad choice for acoustic barriers often brings down the bridge aesthetics but can also be transformed in a success like in the bridge over the Nesenbach valley in Stuttgart where rounded arches carry not only adjustable flaps/wings to optimise the acoustic protection but also a bicycle pathway – Germany [11]

2.4.2 Influence of the longitudinal section

As for the plan view, certain general principles must be observed by the route planners concerning the longitudinal section of the highway. The real impact of a bridge in its surroundings is essentially measured with respect to its geometry in elevation, which is defined by the roadway surface. Without endangering the quality of the route alignment , this should follow certain elementary principles. For example, it is not recommended to choose a completely flat and horizontal geometry for both aesthetic reasons and for rainfall run-off. Also, an abrupt variation in vertical curvature is not aesthetically pleasing.

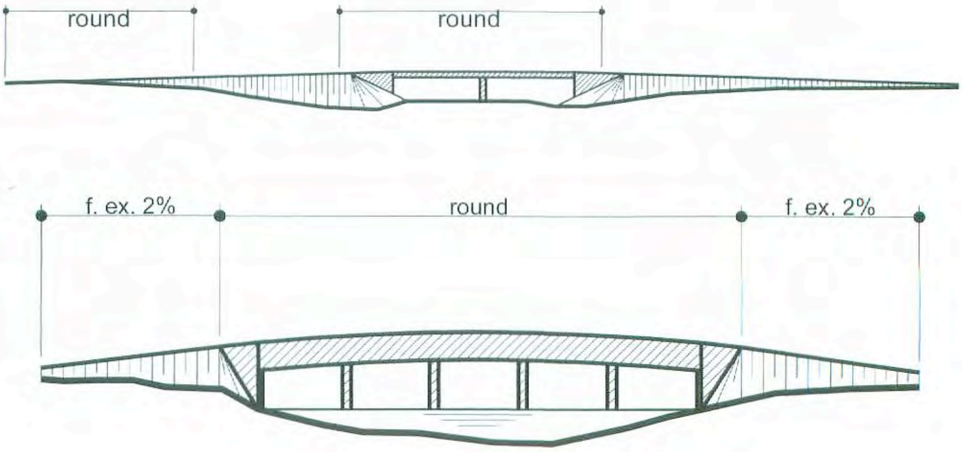

Figure 2.8: Vertical radius of curvature – a good help to lighten the bridge profile, especially for very low bridges in flat countries but beware of the "bump" effect

Generally speaking, a convex vertical radius of curvature contributes to a certain lightening of structural lines and is a logical choice with respect to the crossing function of the bridge. This curvature should be extended to the bridge approaches in order to avoid discontinuity. In all cases, the "bump" effect, which strongly reduces user visibility, should be limited (figure 2.8).

When there is a large altitude difference from one side of the valley to the other or if the route imposes a concave curvature on the structure, the general configuration of the structure should be studied in order to limit a "sag" effect. From an observation point either in the valley or at the level of the route, seeing a structure more or less hung in space between two extreme points is not favourable for the acceptance of the bridge. In order to limit that impression, the concavity of the bridge should be reduced, especially as a function of length, by modifying the layout and spacing of the piers. A progressive shortening of the spans toward the abutments gives an impression of slenderness to the structure and refines its general lines. This last arrangement leads to variable spans for the structure, which is not always an ideal solution from an economic point of view. It is, however, necessary to remember that the structure is going to modify the site for at least several decades, and, without doubt, some sacrifices can be justified in order to achieve better integration with the natural environment. It should be noted that this type of geometrical remark essentially concerns bridges, either highway or railway, subject to normal traffic and that the philosophy governing the integration of an overhead pedestrian or cycle crossing is not necessarily the same.

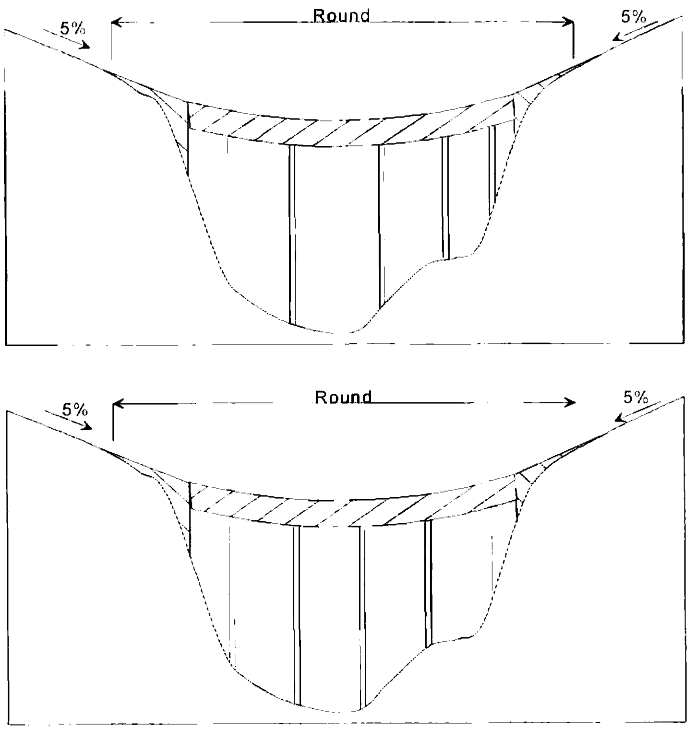

Figure 2.9: Downwards curvature, a variable spacing of the piers limits the sag effect

2.4.3 Transverse section

The clearance necessary for traffic determines the transverse section of the structure. However, certain arrangements must be discussed at the beginning of the project with the entire group of specialists. Firstly, the collection and disposal of rain water needs to be considered. An inverted V profile may be a favourable choice for the transversal section. Nevertheless, such a choice imposes a double network for water flow. This presents no problems in a wide bridge with four lanes. For a two-lane bridge, such a solution is possible but costly, if the structure is designed for two-way traffic. In the case of one-way traffic, an inverted V between the two lanes is unacceptable from the point of view of user comfort and safety.

The disposal of water collection from the road surface must be decided upon in conjunction with environmental specialists to determine if decantation or degreasing basins are needed, as well as agreeing the location for discharging the water into the local drainage system.

From the point of view of the management and maintenance of the structure, it is necessary to decide, in conjunction with the responsible persons, whether a double bridge is required or if a single bridge will suffice. The technical and aesthetic problems linked to this choice must be considered together. The second example presented above illustrates this point. A double bridge has the clear disadvantage of having twice as many piers. A very wide bridge very often means a heavy profile, which certainly is a negative feature in the case of limited available space above the ground. On the other hand, if the height of the bridge is great, such a choice becomes aesthetically and provides improved wind resistance due to its greater stiffness.

A transverse section with variable plan geometry along the length of the bridge is poor aesthetically and economically and should be avoided. The same applies to road junctions, which should remain separate from bridges. These elements are very difficult to integrate due to the many different structural lines which they possess, and which form a fortiori a visual chaos from which it is difficult to bring order.

It is the responsibility of the bridge designer to make sure that these basic elements are respected or at least to inform the project owner and the working group of the consequences of their choices. His intervention does not, however, stop there. It is obvious that during the design of a bridge, the bridge designer is at the centre of the group and the specialists become satellites which intervene in an independent manner according to the needs of or with the agreement of the bridge designer. As an example, the interaction between the bridge designer and the specialists in this phase particularly concerns the following areas:

a. Local Conditions

The designer should make a reconnaissance of the site of the works in order to assess the adequacy of the existing access roads or trails for the transportation of equipment and materials; and the availability of construction materials in favourable conditions for bridge construction and according to the construction permits that will be available.

b. Nature Preservation

This aspect has been quite rightly growing in importance in recent years.

The co-operation of environmental impact specialists, who from the early discussions should participate in the conceptualisation process, is of great importance, sometimes even indispensable, in finding the most appropriate solution which avoids, or, at least, minimises the negative impacts of the project in general and the bridge structures in particular.

The environmental specialists has an important role in the execution of a project.

c. Geotechnics

Knowledge of the geo-technical properties of the site is of prime importance, not only for the conception and design of the foundations, but also for the choice of the best suited construction method.

The co-operation of a geo-technical engineer and/or a specialist in foundation engineering, or both, is highly advisable, and indispensable in difficult foundation situations.

d. Hydraulic Constraints

The knowledge of the flood regime of rivers, or of tides and currents at sea is indispensable for the conception and layout of the bridges and their foundations. Specialists in fluvial or maritime hydraulics are required in order to advise the bridge designer of the options available and their consequences.

e. Wind Action

In some bridge types of medium to long spans aerodynamic instability problems can occur which may create the need for co-operation with specialists in this field.

f. Seismic Action

In zones prone to seismic activity strong points should be provided in order to enhance the bridge's behaviour to the action of earthquakes. Ductility of the structure should be ensured in order to prevent sudden and progressive collapse under extreme loading and deformation, in excess of design values, which can occur due to seismic action.

The use of energy dissipating or damping devices considerably reduces the susceptibility of the structures to the seismic action, thus avoiding a direct confrontation with the forces of Nature.

g. Impact of the Works

Particularly for structures in urban zones, the conception of the bridges and their construction methods should take into account the inconvenience that may result to the community due to the execution of the works.

Solutions that minimise the negative impacts on community life should be duly analysed and implemented whenever possible or necessary.

h. Public Services Networks

In co-operation with the public services companies (water, gas, electricity, telecommunications, etc.) affected by the construction, the designer should consider the necessary spaces required for the installation or relocation of the appropriate services, with an assured easy access for maintenance and possible replacement.

i. Drainage

In some cases, there might be the need for the construction of decantation chambers (for example where oil or gas spills are likely to occur). Where drainage networks are involved, these will need to be connected to the public network.

j. Sound Pollution

In urban zones this is an aspect which must be considered, which may lead to the necessity of installing acoustic barriers.

Any solution must be duly analysed, not only acoustically but also to avoid any undesirable visual intrusion.

Figure 2.10. Morbihan Bridge : to be noted : the Pedestrian walkway along the arch curve. A very good example where Bridge designer can help the acceptance and the attractivity of the bridge for pedestrian. Crossing the bridge becomes a game – France [4]

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k. Accidental impacts

The designer should take due account in the conception and design of the structural elements of the effects resulting from possible impacts of light or heavy vehicles.

The potential hazard from ship or barge collision with the piers shall be considered as well, and suitable protection provided.

l. Clearances

The bridges should have clearances large enough to allow the passage of the largest and tallest of the legal vehicles with reasonable margins or to allow the passage of shipping, as appropriate, in accordance with the standards and/or specifications supplied by the client.

2.5 Conclusion

It has been demonstrated that the design of a bridge generally requires the contribution of several technical professionals, each with their own expert knowledge, in order to achieve the best possible solution to the various aspects that the bridge should satisfy. It is of fundamental importance that the Project Manager for a bridge project always keeps in mind all the aspects of the project, their interconnection and any situation where they conflict now or may conflict in the future. He should be involved in all major decisions taken about the global project, which could influence the conception or design of the bridge.

3 Responsibilities, design and project management

The process of selecting the designer of the project and the contractor in charge of erection can vary widely from one country to another. Consequently it is very difficult to give a global view of the corresponding problems without a short description of some of the existing systems.

3.1 The main participants in bridge construction

The main participants in bridge construction are the Owner, the Project Manager, the Designer and the Contractor.

The Owner pays for the construction and has the full right to make any decision on condition that he duly pays for the ordered work and the pain taken. For public works, procedures aim at avoiding purely personal positions from the man - or men - in charge, and at selecting the designer and contractor on clear and honest bases. Unfortunately, there is an increasing tendency - due to the search for consensus and to the fear of legal challenge or criticism by the media - to select only by the lowest price and to adopt very common solutions. Only very strong owners, who have some technical competence or who call for good advice, can promote innovative and audacious designs and constructions, and are able to select designers and contractors for their qualifications and creativity rather than for cost.

The Project Manager is selected by the owner to steer the project during design and construction. In some countries the project manager is a civil servant for public construction; in others, the project manager is a private (or supposed private) consultant.

The Designer - who must be an engineer - has the responsibility for the bridge design, or at least for the conceptual and detailed design which is the basis of the tender. In some countries the selected contractor is in charge of the execution design, under his own responsibility, the designer - or an independent checker - being in charge of the control of this final design. In countries where the main responsibilities are given to private consultants, the same office is often in charge of the project management, design and site supervision ; but in many countries, especially when the project manager is a civil servant, his team is in charge of site supervision, with external help when necessary, together with quantity surveyors.

The Contractor is awarded a construction contract after submitting a successful tender. He is in charge of erection with the help of sub-contractors and of specialised contractors (e.g. for foundations, prestressing, and cables in cable-supported bridges). Depending on the local system and responsibilities, the contractor produces only shop drawings or the final design using his own design office. He may employ a consultant to undertake this design work, but the contractor still retains the responsibility under the contract for this work.

Many other minor parties take part in design and erection including experts, specialists and testing laboratories.

The distribution of responsibilities between these participants can be different from one country to another, according to tradition and practice.

Several systems can be efficient, on condition that effective responsibilities are given to those who are really qualified to carry them, and that a clear and honest competition is organised between contractors bidding for the construction.

In all cases, the major condition for good design and construction, at reasonable cost, is that either the owner is technically qualified or he entrusts the project management to a qualified project manager, or engineer, who will have the real responsibility under the control of the owner. A direct selection of the design, the designer, or the contractor by non-professional authorities can often lead to a less than desirable outcome – for example, a mediocre design which has no real structural value and which, in some situations, can be completely out-of-keeping; or inadequately cost construction works which can lead to poor quality and financial loss for the constructor; or in extreme cases corruption. The designation of a qualified project manager or engineer, either public servant or private, who has the confidence of the owner and works under his control, is the best guarantee of quality and a straightforward project.

Figure 3.1. « Truc de la Fare » Overpass– France [9]

3.2 Stage of the Design Process

The various stages of a bridge design project are described in detail in Chapters 4 and 5, particularly in Paragraph 4.2. It is necessary, however, to establish the definitions of these stages at this time in order to avoid any confusion which could arise from the use of words or phrases which may, historically, have different meanings in different countries.

The distinction between the stages is the level of design or the degree of detail that may be required at any particular time. It is not possible, however, to define precisely the scope of the work to be undertaken at each stage, due to the differing requirements of

- The alternative types of contract (e.g. traditional Engineer's Design, Design & Build, etc.).
- The Client (or the sponsor for the project) at any particular time during the Contract.
- The type of structure.
- The working practices of the country for which the design is being undertaken.

The following definitions therefore refer to what will usually be the main stages of the design process, and indicate the possible range of work that could be undertaken in each.

3.2.1 Conceptual Design Stage

This is the stage during which the basic form of the bridge is determined, having given due consideration to such matters as:

- The intended function of the structure.
- The position of the supports.
- The distribution of the spans.
- The choice of construction materials.
- The method of erection.
- The type of foundation

The main objective of this stage of the design is to ensure that the concept of the bridge is the most appropriate to satisfy the needs and the objectives of the client among the different possible alternatives. The design produced at this stage is not supported by detailed calculations, and will usually be based upon a simple analysis only. The design will be sufficient for the purposes of identifying the principal load paths and determining the overall dimensions of the primary members, the latter being illustrated on preliminary General Arrangement drawings.

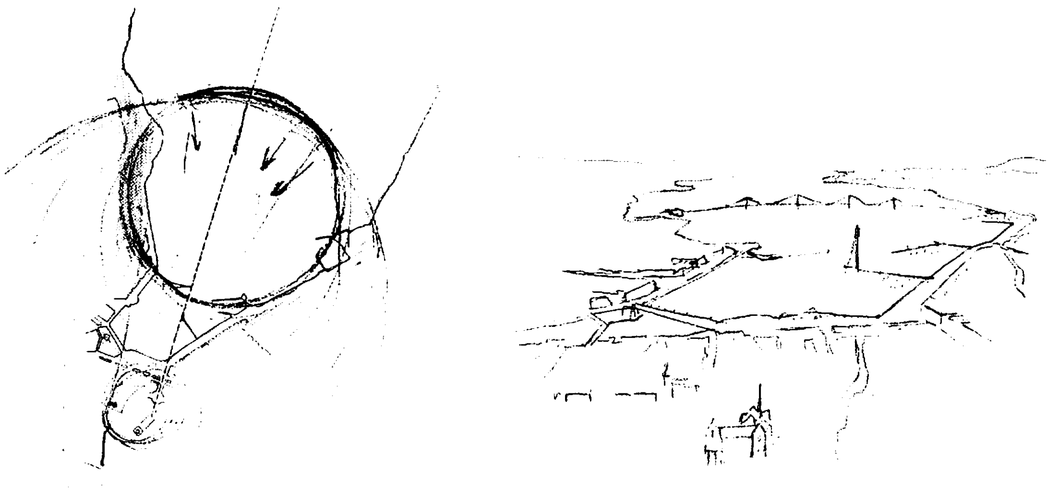


Figure 3.2 First sketches of the Geneva Lake crossing – Switzerland [3]

3.2.2 Preliminary Design Stage

Once the concept of the bridge has been established, it is necessary to progress to the Preliminary Design Stage (also known as the Outline Design Stage). The purpose of this stage is to verify the principal dimensions of the structure and to determine the requirement for prestress and reinforcement at the most critical sections. The design is supported by a more detailed structural analysis, and will be illustrated on drawings that are of sufficient detail to enable preliminary quantities to be determined. This level of design and detail will, in some situations, be sufficient for Tender purposes; but it is generally better to slightly develop the analyses and to give most details of prestressing.

3.2.3 Detailed Design Stage

As the name implies, this is the stage during which the design is undertaken in detail. The level of design undertaken and the degree of detail produced, however, will depend upon the requirements of the project, as discussed previously.

It is usual for the design of the structure to be completed in this stage, and to be fully supported by comprehensive calculations that include a detailed structural analysis. The drawings may then be prepared to show all the details of the structure, including all dimensions and finishes, as well as full details of the prestressing and all reinforcement. The drawings will be of sufficient detail to fully illustrate the engineer's design of the structure and will enable accurate quantities to be determined. Dependent upon the type of contract, these may then form the basis of the Tender Documentation.

3.2.4 Execution Design Stage

This is the final stage of the design process and will usually be undertaken subsequent to the appointment of the main contractor for the Works. During this stage, any remaining items of the design are completed and any additional documents that are necessary for the fabrication and erection of the structure are prepared. The fabrication (or 'shop') drawings for the various elements of the structure will usually be prepared by the Contractor and approved by the designer. Additional documents may include bending schedules for the reinforcement, if not produced at the detailed design stage.

This may also be the stage at which any documentation required in connection with subsequent inspection and maintenance activities will be prepared. Although these matters will have been considered at an earlier stage of the design process, it is unlikely that the formal documentation (guidance notes, maintenance manuals, etc.) could be completed before this time.

As the construction progresses, it is uncommon for situations to arise which require changes to be made to the original design or details. Theses changes may be required for a variety of reasons, but will often be associated with unforeseen ground conditions, a change in the supplier of a component of the structure or a change to the sequence of construction. It is important that any changes are fully documented, and that the drawings are revised to an 'as-constructed' status as soon as possible.

Figure 3.3. Firth of Forth Bridge ; When intuitive design leads to an historic monument – UK

3.3 The different allocations of design and erection responsibilities

Different allocations of design and erection responsibilities are illustrated here by describing – as examples - some of the existing systems which have generally resulted from very old traditions. No system is perfect and each one has clear advantages when actual competencies are distributed between administrations (civil services), private design offices and contracting companies in such a way that responsibilities are really given to the most qualified taking into account the public interest. A system can only be judged by its results : a system is good when it produces good and safe designs, and favours innovation and creativity resulting in beautiful and durable bridges at reasonable costs.

3.3.1 The British system

In the British system, the consultant leads design and construction. He is in charge of project management, design, preparation of tender documents, analysis of bids presented by contractors, site supervision to control the quality of construction and, with the help of quantity surveyors, the payment of the contractor. The civil servant who represents the owner has only limited responsibilities : he selects the consultant in charge of design and project management and he controls finances. The contractor is only in charge of practical erection, as specified by the design ; he produces shop drawings only to detail what is necessary for construction.

Clearly, all technical decisions are in the hands of the consultant. The situation is so extreme with the recent reduction in the responsibilities of the civil service that, for very large projects, the government nominates a private consultant to act as its agent.

Traditionally, there has been no external control of the designer's work, but recently an independent checker is appointed to check both drawings and computations. This system has been extremely successful and helped encourage very strong and creative engineers. It was slightly altered in the 19th century with the emergence of the main railway lines, when private companies entrusted famous engineers with the construction of their bridges or even the whole railway. The engineer then acted exactly as a civil servant for a public owner, driving design and construction with the authority which came from his competence and responsibilities.

The tradition of strong private consultants has been maintained in the 20th century, for example with suspension bridges, but with the increasing influence of management, financial and legal aspects, consultants have been obliged to limit their risks. They support the construction responsibility, but their financial capacities are very limited compared with the cost of the structures. The temptation is then high to protect the office by conservative designs, reducing innovation and technical progress.

3.3.2 The French system

The system which exists - or at least existed - in France is very different. There is a strong tradition of direct involvement of civil servants which dates from the 17th and 18th centuries when the government selected and educated civil servants to directly design and manage the erection of bridges and roads. A special service of engineers was constituted in 1716 (the Corps des Ingénieurs des Ponts et Chaussées) after the collapse of the bridge at Moulin over the river Allier, which had been designed by a famous architect, Jules Hardouin Mansart.

In the 19th century contractors proposed new solutions with the development of steel construction and later of reinforced concrete, when civil servants failed in using and developing these new techniques. The resulting system gave the main responsibilities to civil servants and to contractors, leaving only a very limited role to private consultants.

Basically, a conceptual or detailed design is produced by civil servants or by private consultants working for the owner. When tendering, contractors can propose limited - and sometimes more wide-ranging alternatives. In almost all cases the contractor has to produce an execution design with all corresponding computations, which is controlled by the initial designer or by another consultant, public or private, working for the owner.

But the economic competition between contractors limited their financial capacities and almost all of them reduced their design offices which sometimes completely disappeared. More recently, the increasing influence of politicians in practical decision making and their greater involvement with technical matters has reduced the technical capacities of civil services resulting in a serious weakening of the system which can work effectively only when the civil service is technically qualified and has the confidence of the authorities.

3.3.3 The American system

All other systems lie between these two extremes. The American one is very close in most states to the British system. Some competition has been introduced for example by stating that there must be steel and concrete alternatives. Unfortunately, the decision is generally taken on the basis of cost only, without consideration of quality, durability or aesthetics. Another means of introducing competition is the possibility for the selected contractor to propose an alternative solution after being awarded the contract and to share the savings with the client.

The result can lead to great efficiency, with very low construction costs, but sometimes with limited durability and questionable aesthetics.

Some states, e.g. California, prefer maintaining a strong civil service which can design or at least influence design in order to achieve a higher quality, albeit at a higher cost. In the same way, the Port of New York gave its engineer, Ottman Amman, the responsibility for the design and construction of its bridges (George Washington, Bayonne, Verrazano), demonstrating like the English railway companies in the 19th century or the French engineers in the 18th, that qualified, innovative, gifted and visionary civil servants are a key to the achievement of quality within their domain.

3.3.4 The German system

In Germany the system has many common points with the French one. The civil service is much stronger than in the British or American system, and large contracting companies can widely influence design and construction. However, design has been dominated by some strong consultants who often had the authority of being at the same time professors in the technical universities. For example, Fritz Leonhardt and others could influence construction not only by their designs, but also indirectly by the education of young engineers and imitation by their colleagues.

The control of projects is systematically organised with independent checkers (Prüfungs-ingenieur).

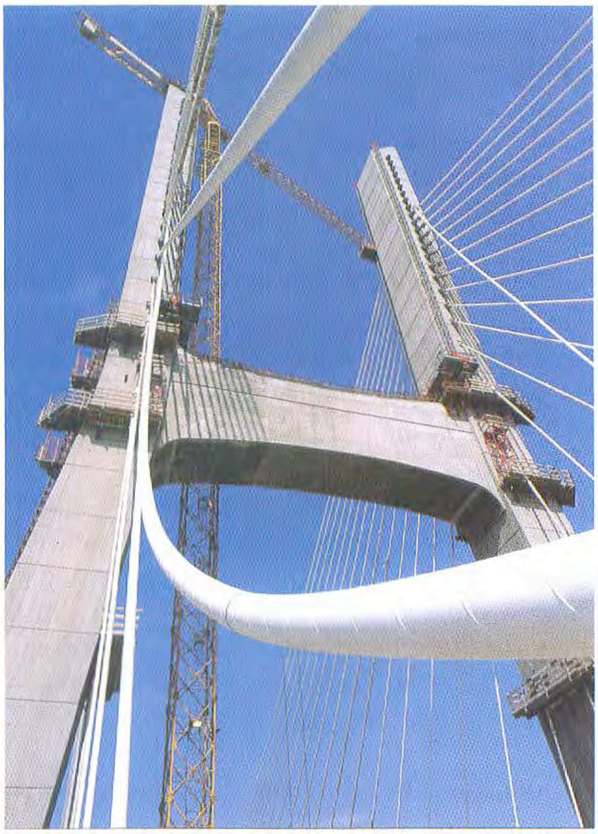


Figure 3.4. Vasco de Gama Bridge over the Tagus River in Lisbon ; a structure built according a concession agreement – Portugal [18]

3.3.5 The Swiss system

Finally the Swiss system has a strong tradition of independence of private consultants in charge of all aspects of design and construction, even more than in the British system. However, because of the political division into cantons, none of them had a very wide influence even when they achieved world-wide fame.

3.3.6 Design and build contracts and Concessions Agreements

The classical systems existing in different countries for distributing responsibilities between owners, designers and contractors are changing with the development of Design-and-Build contracts and, even more, with design-build-and-operate contracts within concession or not. Although this chapter mainly reviews traditional systems, Section 3.9 addresses this recent evolution, which becomes more and more important, and deals specifically with design-and-build contracts and concessions agreements (figure 3.4).

3.4 Legal responsibilities, insurance, competence and initiative.

Experience clearly shows that the best result comes with :

- a strong and decisive owner;
- a qualified and experienced project manager who receives total support from the owner and who selects a creative, qualified and experienced designer;
- mutual confidence between project manager and designer; and
- both discuss and work with a qualified and responsible contractor to aim for high quality, duly paid for under by the contract.

This is, of course, a situation which cannot be obtained for all works. At least, the system must aim at giving to the most qualified the greater responsibilities and to balance freedom in design with responsibilities.

However, legal aspects and insurance conditions can widely influence the system. The actual responsibility in design and construction is divided between owner, project manager, designer and contractor, but the exact distribution depends on the legal system in the country. Evidently the system will be more efficient, and morally acceptable, when legal responsibilities match competence and initiative.

In the traditional British and similar systems, contractors are only responsible for execution; designers normally carry all responsibilities for project management and design. They then have to be financially protected by an insurance, the cost of which is high compared with their fee. In order to limit the risks of accidents or claims calling for the support of their insurance company - involving later increases in insurance cost, or a direct control of designs by the insurance company - designers tend to be conservative.

In other countries like Germany or France, the main responsibility is given more to contractors than to designers, due to the relative costs of design and construction. Only contractors are likely to be financially solvent if there is a major problem such as a collapse. Courts generally distribute responsibilities after an accident between all construction participants not only in accordance with their liabilities, but also in accordance with their solvency. To balance this, the contractor has the responsibility to produce, directly or through an external office, an execution design which allows him to efficiently control the project. The contractor also has some freedom to propose amendments and improvements to construction details in addition to alternatives which he is allowed to propose when tendering, and which have to be analysed and rejected or accepted in the contract.

However, because of the increasing pressure from public opinion after accidents or when construction costs increase in unacceptable proportions, and also because of the increasing influence of judges and lawyers in Society, there is a tendency from owners and from consultants working for them to place all responsibilities on contractors so that they are protected in all situations. This is totally unfair if no freedom is given in design to contractors ; there cannot be responsibility without the freedom to influence the design. In some recent international competitions the tender documents have attempted to place all responsibilities on the selected contractors, who were to produce an execution design, but all the details in the design were mandatory so that the contractors could not change anything and therefore could not actually be held responsible. This was an evident abuse and, in the case of an accident, the contractors could have gone to Court and proved that they did not have the freedom to fulfil properly any design responsibility.

Of course, contractors are also protected by an insurance, but its cost is smaller compared with the erection cost than that for a designer compared with his fee. In addition, since the execution design for which the contractor is responsible is more and more frequently given to an external design office, this office also takes out an insurance. This leads, sometimes, to a distortion of the system. With the increasing pressure of economic competition, contractors tend to bid below the actual construction cost, aiming to claim later additional payments from the client. However, since it is more and more difficult to present successful claims to clients who are prepared to pay them, some contractors take any pretext to involve their external design office and to attempt to recover their losses from their designer's insurance. This is nothing less than a fraud, and it is morally unacceptable to put all the responsibility on the sub-contractor, frequently under-paid, who produces the most creative work.

It is very clear that owners and project managers take a large part of the responsibility when they let design or erection contracts much below the realistic price ; they are supposed to know what this price should be and to be aware of the effect of their decisions on quality and safety. The responsibility of designers and contractors is limited by the pressure of the competition exerted by the owner and his representatives.

Finally, the contracting conditions have a direct influence on where the responsibility lies for incomplete or incorrect designs. When a design is developed for an owner which is later shown as incorrect in some respects, the consequences for the contractor – and the design office which might have worked for him – are clearly the responsibility of the owner. It is owner's responsibility to later seek compensations from the project designer for any errors made by the latter. Public owners have a duty to pay a fair price for the construction (and the design) – including the consequences of problems coming from their side and not to reduce the price paid to the contractor (or designer) below that suggested by the principles of natural justice, as very nicely said by the French engineer Vauban in a famous letter to Louvois, the war minister of Louis XIV.

As a clear conclusion, much depends on the quality and judgement of the owner who has to prepare, directly or indirectly, fair contracts let at reasonable prices. Good contracts and reasonable prices can prevent the destructive effects of legal and contractual aspects when they dominate design and construction, as in some cases in the United States and more and more in Europe.

One way forward would be the instigation of a global insurance for a project, including both design and construction, directly taken out by the owner, or by the contractor through his contract obligations at a clearly stated cost. This would prevent placing the higher risks on the designer whose fees represent a very small part of the final cost. Insurance companies will very soon be in a position to adjust the insurance rates between contractors, thereby influencing the economic competitiveness of the high risk or dishonest ones. They could also adjust their rates to the project itself and to the qualification of owners, project managers and designers, based on previous experience, and thus encourage owners to take a more professional approach to the selection of all the participants.

Insurance companies may be interested in receiving some consultancy advice from qualified and experienced engineers in order to help them evaluate their risks and the corresponding rates; formal certification systems such as quality assurance cannot really help them in this evaluation.

3.5 Selection of the designer and of the design

When the owner is selecting the project manager and later the designer, or when the project manager selects the designer, he must be conscious that he is taking one of the most important decisions on the project. The project manager and the designer will work for the owner, design the bridge and have a major part in the relationships with the contractor and sub-contractors. The fee for the project management, the design and later for the technical control is very low compared with the construction cost (between 5 and 10 %, depending on the precise work to be undertaken), and it is absolutely clear that reducing this fee by 20 or 30 % can be an extremely poor decision if quality is also reduced in any aspect, e.g. architectural elegance, technical performance or durability. It can also be totally counterproductive, financially, if a poor design leads to a more expensive solution (30 % more in design is balanced by less than 1 % in construction cost !) or later to a claim by the contractor which could correspond to several times the designer's fee.

The project manager and the designer must be considered by the owner as his own men, who must be, in return, totally devoted to the owner's interest. They must be totally faithful to the owner to be trusted and supported by him. Complete confidence must exist between them.

The prime conditions for developing this confidence are a clear definition of everybody's responsibilities, fair contracts for the project manager and the designer, and reasonable fees. High fees cannot guarantee that projects are good, but low fees directly lead to poor projects and to poor relations between owner, project manager and designer.

3.6 Relationship with architects

Just after the war the main goal was re-building the bridges which had been destroyed; time and money were then the main concerns. This situation continued through the golden sixties when the expanding western countries built their networks of motorways. It must be said that many of the bridges built in those times were not elegant, and some heavy, boring and even ugly structures were built without any taste by poor engineers.

But good engineers were already concerned with aesthetics, and elegance became a major goal for them in the seventies. Architects began to be associated with some projects, and this became more and more frequent in the eighties. This association is not universal : some good engineers are able to detail shapes very elegantly ; others prefer working in collaboration with an architect who can understand their structural intention and help in expressing it.

To achieve such a goal, the architect must be associated with the designer from the beginning of the project, taking part in the site analysis and in the selection of the structural concept. Fritz Leonhardt often gives an excellent example with the Düsseldorf bridges : in order to fulfil the navigational requirements and for technical reasons, including the designer's structural goals, the bridges had to be cable-stayed. The city architect, F. Tamms, considered the possible conflict with the cathedral towers and recommended a unique pylon, on the opposite bank. This tower consisted of two vertical columns, and he recommended eliminating any bracing for transparency, leaving, of course, to the engineer the responsibility of making the design work structurally. The result is the Düsseldorf Kniebrücke, a really splendid structure, perfectly adapted to the site.

Any system has its drawbacks, and the association of architects in the design has led to some very poor bridges when the engineers had no creativity. Many bridges, for example, are standard structures with the addition of an artificial decoration ; such decorations have no meaning at all, and can be ridiculous in the worst cases. Sometimes also, when the designer is not strong enough or not adequately supported by the project manager or the owner, the system can fail and the architect can introduce artificial elements which weaken and alter the structural concept itself.

Recent years have seen a more dangerous evolution which indirectly results from the ecological movements. As a response to poor construction in the sixties and seventies, in which bridges had practically no part, society reacted against any new construction and mainly against new motorways and railway lines. Some politicians and authorities then decided to give bridge designs to architects, following the unfounded idea that architects, who are supposed to design for beauty, will design beautiful and, hence, acceptable bridges.

The result has been a disaster in many aspects : illogical shapes lead to inefficient structures and to high costs, and many times to ugly structures. It is very clear that some bad experiences will bring an end to such an approach, but it will take some years, and all parties involved must work to stop such an aberration.

A clear analysis of responsibilities provides an appropriate answer: only engineers, who master the structural aspects and have the experience of construction techniques and contracts, can be responsible for safety and cost. It is then absolutely illogical and highly dangerous to give architects the responsibility for design. On the other hand architects can contribute much to bridge design, and engineers who do not know must learn how to work with architects or how to design elegantly.

When an architect is associated with a project, he must be involved from the beginning and have the opportunity to work with the team and have his views listened to. A full and confident co-operation with the engineer must exist; but the final design responsibility must remain with the latter. The role of the architect in the design team is clearly to take part in the evaluation of the site and in the selection of the structural system with the engineer, but his main contribution consists in expressing the engineer's structural intentions by adapting shapes and details, and colours also when they can help the design to produce a delicate balance of light and shadow on the bridge elements or strong contrasts, as desired.

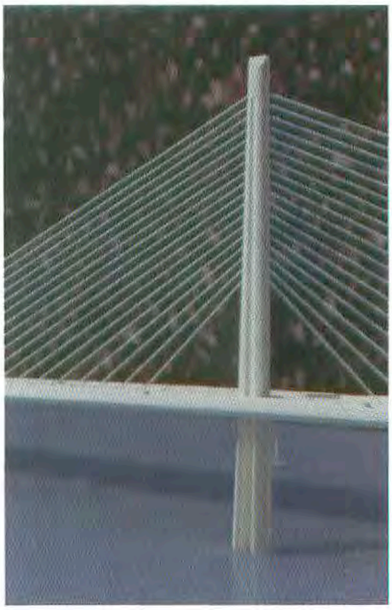


Figure 3.5. A Good collaboration with architects can help to complete and perfect shapes and details: Back span anchorage study for a proposal to the Stonecutters bridge - Pylon shape study for the crossing of the Geneva Lake – Switzerland [3],[21]

3.7 Awarding construction contracts

The selection of the contractor is extremely important. The selection process varies according to the procedure being followed and the design system:

- when the designer prepares a detailed design, the contractors only give a price for clearly determined works, with possible alternatives proposed by the successful contractor to reduce the cost, as in some cases in the United States;
- when the designer only prepares a preliminary design, the contractors can sometimes propose minor or major alternatives.

Whatever the procedure, construction will be successful in all aspects and for all participants only if the construction contract is clear and fair, the structure to be built is precisely defined, agreements exist on construction techniques and equipments, risks are reduced as much as possible for all parties involved, and prices are reasonable meaning that they are not exceedingly high nor too low. Low prices for construction, as well as for designers, cannot lead to good quality and could lead to large contractual problems and claims. When a contractor is losing a lot of money he may be tempted to take any occasion to present a successful claim. Such a situation can be more counterproductive for the owner than a reasonable price from the beginning.

For all these reasons the project manager, with the help of the design team, must evaluate very carefully :

- the tendered prices, to detect abnormal ones, either too low or too high ;
- the proposed construction techniques and equipment, to check whether the constructional problems have been correctly analysed and if the appropriate equipment has been selected ;
- the proposed manpower, both in number and experience ;
- the proposed alternatives, to check the proposed quantities and their conformance with code requirements, and also quality requirements many of which only come from engineering experience and cannot be adequately specified in codes of practice.

When the construction is to be based on a detailed design, and when no aspect of the project is expected to change in any detail, the cost can be given as a global fixed price. This is not recommended when the contract is based on a preliminary design only (excluding, of course, contracts based on a conceptual design, a situation that no engineer can seriously countenance) : when the quantities can change slightly due to the results of execution analyses and designs, and there is no reason to charge the corresponding cost to the contractor.

This point can be debated when the design is an alternative proposed by the contractor, but it is recommended to check precisely the dimensions before signing the contract, on the basis of a detailed analysis including corresponding computations, and to pay for the actual quantities. If the cost is represented as a global fixed price, the project manager and the engineer in charge of the construction will have no easy means of instructing improvements to the project which are not directly prescribed by code requirements, but are only "règles de l'art" which have no precise and legal definition.

With the development of concessions (design, build and operate contracts) and of private financing, there is an increasing tendency to ask for global and fixed prices. This demand is strengthened by the bad experience that some banks have had in recent projects where the costs have increased dramatically. However, it is an illusion to think that a global price could protect owners from important claims which are based on contract changes more than on an increase in quantities. The large problems and financial disasters come about when the project has to change after the signing of the construction contract. The real protection for owners comes from a good project, which will not change except in details ; a good project manager who, with his team of engineers, is capable of exercising proper control of the construction, both technically and financially ; and a contract at a realistic price with the contractor, which duly pays for the works of the required quality.

3.8 Sub-contracting

What is true for designers and contractors is also true for subcontractors and suppliers : they can only work properly and produce high quality if they are paid a reasonable price. Unfortunately, contractors often play off one subcontractor against another to obtain lower prices, sometimes beyond the acceptable limit. It is the responsibility, and in the interest, of the owner and the project manager to prepare a contract in such a way that this tendency is limited, and to agree very early on in the contract the names of the subcontractors to prevent later pressures.

3.9 Design and build contracts

This section applies to design-and-build contracts and design-build-and-operate contracts awarded either by a private owner or a public authority.

3.9.1 In design-and-build contracts, the contractor takes responsibility for both the design and the construction. This situation raises new problems which must be adressed:

- the definition of the requirements by the owner or the public authority;
- the evaluation and award criteria; these can be very sensitive as the proposed projects may be very different;
- the system to be used for technical control of the project during both the design and construction stages; and
- the distribution of risks between the different parties.

3.9.2 The owner, or the authority awarding the contract, must prepare extremely clear documentation defining:

- the functional requirements, and
- the technical requirements regarding performance, safety, durability, inspectability and maintalnability.

Imprecise specifications may lead to distortion in the competition between tenderers, to the as-built structure not having the required quality, or to legal disputes.

3.9.3 The evaluation and award process must be clear and as transparent as possible. The owner, or the authority awarding the contract, must ensure that the proposed projects are safe and that they fulfil the specified requirements.

However, the comparison of very different proposals cannot generally be made without a measure of human judgement, if only when comparing their aesthetic qualities.

3.9.4 In a design and build contract the contractor is responsible for both the design and construction of the project; however, there must be a system of technical control which will assure the owner that the functional and technical requirements for the project are being attained.

This control may be performed directly by the owner (or responsible authority), or on behalf of the owner by a design office working under the owner's direction. Such control must not lead to changes in the design and construction conditions expressed in the contract; therefore specifications must be very clear and precise and the contract documents modified – before being signed – to reflect the specific requirements of the selected solution.

3.9.5 Finally the contract documents must be very clear regarding not only who is talking the risk for construction, but also who is taking the risk for the soil conditions, seismic loading, wind loading and other such issues.

3.10 Concession Agreements.

The comments made in Section 3.9 also apply to bridges built under a concession agreement. In addition other issues need to be considered.

3.10.1 The control of design and construction might be carried out directly by the public authority granting the concession, or by a design office appointed to act on its behalf.

There is a risk in this situation that the public authority will try to change the design and construction conditions set out in the contract, in order to obtain a level of quality and/or safety higher than specified and thus assume, perhaps inadvertently, some of the concessionaire's responsibilities.

Another risk is that each partner – the public authority, the banks financing the project, the concessionaire and the contractor – organises its own technical control, leading to a duplication of effort, a confusion of responsibilities and a potentially long, drawn out process for reconciling differences between the parties.

3.10.2 Therefore the procedure usually adopted in a concession agreement is that design and construction are controlled by an independent checker.

This solution also has some drawbacks when the independent checker is to be paid directly by the contractor. It is far better for the independent checker to be appointed and paid for by the concessionaire. Even so such a solution does not solve all the problems, particularly when the contractor's organisation is also a party to the concession agreement.

3.10.3 The way the control of design and construction is organised is so important and can have such an influence on costs and delays that it must be clearly set out in the contract documentation.

4 Definition and costs of design

4.1 Introduction

The awarding of contracts for public service works should theoretically be carried out on the basis of comparative studies of the services offered by different consulting firms using criteria such as competence, experience, equipment, references and costs. Unfortunately, the market economy and current unrestrained competition modify the relative importance of these criteria in many cases so that only one criterion is considered: that of cost. This tendency is encouraged, even standardised, by the coming into force of the European Union Directives relating to "the co-ordination of Procedure for the Award of Public Service Contracts (Council Directive 92/50/EEC of 18 June 1992)". These regulations obligate the authorities to choose a consultant based on the best price.

It is necessary to fight this tendency actively and to present to clients the problem and the important consequences which such an attitude may bring about. The selected consultants should provide the client with the best services demanded by professional ethics. They should research the best solutions, determine the most economical, and advise the client efficiently. To achieve this, it is of prime importance to work in a climate of total reciprocal confidence between the client and the selected consultants so that each team members is pulling together and that a certain synergy is formed for the research of economical and efficient concepts. This desirable relationship cannot develop correctly in the case of direct and immutable limitation of consultants' fees which are calculated based on the total amount of the consulting costs before construction. Consequently, those who award contracts should search for quality in the studies, being conscious of the fact that minimum study costs do not correspond to minimal construction costs.

4.2 Progressiveness in design - design process

The services of the engineer evolve with the progress of the project and may be separated into different stages. The objectives of each work stage are summarised in Table 1, with a brief indication of the services carried out.

The list of services presented in the following table 1 is not exhaustive and may vary greatly because of consulting regulations in different countries. However, the information in this table gives a complete example of the services of an engineering firm independent of a contractor and responsible for the entirety of the studies, the design and the local supervision of the construction. In other cases, the engineering firm might not carry out certain of these services. It should be noted that the client should be continually informed of the development of the project and that major decisions are always to be taken after consultation and agreement with the client.

Work stages:

Preparatory work

Objectives	· problem analysis· . definition of the mission · research of existing data
Existing data	. client's requirements · local conditions
Designer services	· catalogue of constraints · research of existing data · analysis of technical possibilities · possible hiring of specialised professions preparation of their contracts
Decisions by the client	· have a preliminary study carried out · designate the specialised professionals

Preliminary study

Objectives	· presentation of the complete range of solutions · choice of studied variants
Existing data	· results of the preparatory stage · definition of the mission
Designer services	· flow chart of the studies · variant studies · brief estimate of variant costs definition of assessment criteria for the variants · contact with the specialised professionals
Decisions by the client	· continue the study of the chosen variants until the preliminary project or terminate the studies or start the preliminary study again. · approve the organisation of the operation

Preliminary project

Objectives	· presentation of the preliminary project · organisation of the operation
Existing data	· results of the preliminary study · choice of variants ·
Designer services	· supervision of the studies · preliminary calculations and preliminary design of the retained solutions · sketches, perspective drawings, models · propositions for special investigations · graphic representation of the preliminary project · call for preliminary tenders for the special installations · estimation of the construction costs based on preliminary survey · choice of the final variant · writing of a technical report

Decisions by the client	· approve the preliminary project · continue the study of the chosen variant until the final project or terminate the studies or start the project again with new bases

Final project

Objectives	· presentation of the final project in order to decide if the works should indeed be carried out and to prepare the invitation to tender
Existing data	· preliminary project · results of specialised analyses · additional requirements of the client
Designer services	· supervision of the studies · conception and design of the structure · drawings, graphic representations, models · discussions with the client and the various parties affected by the bridge construction · writing of a technical report
Decisions by the client	· approve the final project · proceed to an invitation to tender or start the project again with new bases or possibly terminate the studies

Invitation to tender and comparison of offers

Objectives	· propositions for awarding of contract
Existing data	· final project · general cost estimate
Designer services	· supervision of the work · preparation of tender documents · draft contracts · writing down special conditions · organisation of information meetings and site visits · invitation to tender · evaluation of the offers based on quality, deadlines and prices · preparation of propositions for awarding of contract · comparison of foreseeable costs with respect to the general cost estimate
Decisions by the client	· award the contract and have the works carried out or proceed to a new invitation to tender with new bases or decide to cancel the project

Executive project

(These services are often carried out by a firm affiliated with a contractor)

Objectives	· providing of documents needed for the execution of the project
Existing data	· contracts with contractors and suppliers · drawings and documents established by the specialised professionals
Designer services	· supervision of the studies · detailed study: final calculations and design necessary to guarantee the security and use of the structure during both construction and service · draft contracts · preparation of construction details · siting plans · construction drawings, framework, reinforcing and detail plans · co-ordination of the studies carried out by the specialised professionals and contractors · check and approval of manufacturing and workshop drawings · choice of construction materials in collaboration with the client

General supervision of construction

(These services are often the responsibility of the client but may be included in the engineering mission (e.g. Switzerland)

Objectives	· creation of the conditions necessary to carry out the construction
Existing data	· executive project · work schedule · general cost estimate · propositions for awarding of contracts and the decision of the contracting authorities · the contracting authorities' specifications
Designer services	· general supervision of construction · finalisation of the contractors' contracts · monitoring of deadlines, construction costs and financing until the final hand-over · definition and co-ordination of the missions and the responsibilities of the authors of the project, the local supervision of the construction and the other participants · analysis of the problems and preparation for decisions to be made during construction · periodic on site work checks · supervision of the control operations and the measures necessary to guarantee the security and the functioning of the structure · preparation and presentation of the final accounts, comparison with the general cost estimate · choices of measures to be carried out in order to eliminate observed defects

File of the completed structure

Objectives	· documentation on the completed structure
Existing data	· executive project · directions for the supervision of construction concerning modifications
Designer services	· updating of main execution drawings · gathering of documents of the specialised professionals · assembling of documents necessary for operation and maintenance · handing over the file to the client

Supervision of the guaranteed repair of any construction defects and final verification

Objectives	· elimination of the observed defects on the structure
Existing data	· verification reports
Designer services	· gathering, preparation and updating of the list of visible defects · notification of defects · summoning of contractors and suppliers involved · organisation and report of the final verification · proposition for a possible extension of the guarantee period · propositions for the lifting of the guarantees

Services which assist the client

(These services are due when the supervision of the construction is guaranteed by the client)

Objectives	· assistance to the client for the major decision
Existing data	· executive project and the directions given by the author of the project · work schedule · general cost estimate · the contractors' contracts · the specifications for the general supervision of construction · list of control operations
Designer services	· assistance to the client on request

Table 4.1: Definition of engineering services as a function of the stages of the study

4.3 Problem of the definition of construction costs

Most of the regulations concerning civil engineering services and the calculation of fees are based on a percentage remuneration of the final cost of the project. Therefore, in order to be able to establish a precise fee offer, the engineer must theoretically know the final cost of the project. However, a serious preliminary project study is necessary in order to estimate the cost with such precision, since all projects differ and general rules for estimating costs are too broad. This is even truer as the complexity of projects continues to increase due to multiple external constraints, these becoming more and more restrictive. The urban environment in which construction is often carried out at the present time is also a complicating factor in many projects. Because of this, the size of the works no longer relates exactly to the study costs and often smaller urban constructions are clearly more complicated with regard to details and site organisation than large, repetitive constructions in natural settings.

These comments are equally valid for the frequently encountered activities of restructuring, repair and widening of existing structures where the problem of maintaining traffic flow on the structure undergoing reconstruction often dictates detailed studies whose costs are proportionately high compared with the global cost of the works.

Figure 4.1: Widening of bridges under traffic flow increased the global cost. Example of the Versoix bridge – Switzerland [3]

Consequently, the exact definition of the construction cost before any studies is idealistic and unrealistic. Also the complexity of the study is usually not always immediately apparent at the outset. Ideally, the contents of the study should also be perfectly defined and established through a close collaboration between engineer and client.

4.4 Selection criteria

The above arguments cast strong doubt on the use of cost as the only criterion for selecting consultants. Sometimes, in extreme cases, this method may even bring about fundamental errors in the selection of consultants. Well-equipped engineering firms with experience are able to estimate the extent of project difficulties rapidly in order to offer their services at a reasonable price. In the event of competition with a poorly-equipped, inexperienced firm, this advantage may rapidly turn into a disadvantage because their offer will most probably be higher than that of a firm which, unconsciously, underestimates the extent of the difficulties. Because of the European regulations in force, and of it's own wish of economy, the client will choose the lowest offer even though he may be poorly served by the selected consultant who has underestimated the task.

This example shows the inadequacy of this method because of poor knowledge of the exact extent of the project. The client may remedy this situation slightly, and make the chances for competing firms more equal, by restricting the invitation to tender to a selected group of firms.

The use of this type of selection method is only possible if the client is willing to take the time to define the project requirements exactly and to evaluate the received seriously offers. He should have sufficient competent technical advisors in order to write a complete and precise list of responsibilities, describing in detail the extent of the project and the services to be included in the invitation to tender. On this basis, he may choose the participants who are best able to provide these services and invite them to submit an offer. It is equally necessary to specify that no subletting of the main elements be allowed, in order to avoid large differences in price. Finally, the quality of the studies provided must be verified and all of the services included in the submitted offers must be fully compared. A detailed list of the services is required in order to accomplish this task,.

In making these various checks in a conscientious manner, the client carries out a procedure which is almost identical to the direct selection of a consulting firm based on its competence and references. Thus, he avoids the competition between firms of differing levels.

Firms differ as much by their human as by their material resources. The level of a firm can be directly related to its operating costs. A serious firm invests in the continuing training of its personnel and participates in research and the writing of technical publications. Firms must inevitably pay for this training so as not to stop the evolution of the creative process. In other words, the choice of a consulting firm should be made as a function of the following criteria, if possible, before considering purely economic aspects:

- human resources (CVs of key staff)
- calculation possibilities
- development of advantages (material or other) belonging to the firm
- participation in training
- participation in research and publications
- references from similar jobs
- declarations of clients concerning work previously carried out

Selecting a consultant based on these criteria reflects an energetic and creative personality of the designer. Such a selection process will provide the client with helpful hints during the search for economical and efficient solutions, as long as he does not limit his means too severely at the outset. A saving of 20% on the cost of initial studies is negligible in comparison to the savings on construction costs which may be achieved following initial studies of better quality.

For the client, it remains necessary to be prudent in the face of new tendencies to standardise the marks of quality of contractors and engineering firms such as quality assurance standards like ISO 9001. Being certified ISO 9001 does not necessarily guarantee quality services. As an example, the fact that a biscuit factory is certified ISO 9000 does not guarantee that it produces quality biscuits, but that it always produces its biscuits in the same way, with an identically consistent process. Similar conclusions may be drawn for engineering services.

Therefore, if a client chooses engineering consultants on the basis of an invitation to tender, practically speaking, he should consider the following selection criteria in the following order:

1. definition of services
2. degree of detail brought to the study
3. calculation possibilities and human resources
4. references of the firm
5. approximate cost of the study

4.5 Methods of remuneration

The current practices in force in most countries concerning remuneration methods for engineering firms are divided into 3 distinct categories:

1. remuneration based on a percentage of the works (cost tariff)
2. remuneration based on a fixed lump sum price
3. remuneration based on time required (time tariff)

4.5.1 Remuneration based on a percentage of the works (cost tariff)

Remuneration based on a percentage of the works permits, in principle, a relationship between an adaptation of the study costs and the actual cost of the project and its difficulties. This payment scale defines a coefficient of difficulty for the project or a part of the project, and proposes a coefficient of division of services which, as a function of a determined project cost, permits the calculation of the total fees.

The determined project cost includes all the expenditure related to the project, which is the object of the contract, after having deducted all previously agreed upon reductions and discounts.

4.5.1.2 Coefficient of difficulty

The degree of difficulty is a function of the nature and extent of the engineering task. The level of knowledge necessary for the carrying out of the contract also is taken into account in the determination of the coefficient of difficulty. This is based on the following criteria:

- amount of responsibility and the risk undertaken by the engineer,
- calculation difficulties and technical level of the construction,
- extent of the necessary work,
- complexity of the task,
- difficulties in carrying out the project,
- topographical, geological, geo-technical, hydrological and climatic conditions.

A further adjustment of the coefficient of difficulty may be agreed upon in cases with special situations linked to local conditions (climate, urban environment, ease of access, groundwater table ...) or in cases with specific organisational problems, for example, because of special requirements. A reasonable order of magnitude of coefficients of difficulty is given in Table 4.2. These values have been taken from various national codes and are essentially based on acquired experience.

A weighting coefficient of 1.1 to 1.5 may be applied to certain values in Table 4.2 as a function of certain particular circumstances, such as the use of sophisticated or occasionally used techniques, or in the case of major difficulties linked to a complex or restricted construction environment. This weighting should be decided in agreement with the client at the beginning of the planning mission. This consideration of the conditions of difficulty are, in principle, also necessary for the study of small projects which often present a large number of difficulties.

Figure 4.2: Construction of bridges in urban areas is often more complex than a large structure in natural settings. Example of the "Le corbusier" viaduc in Lille – france [3]

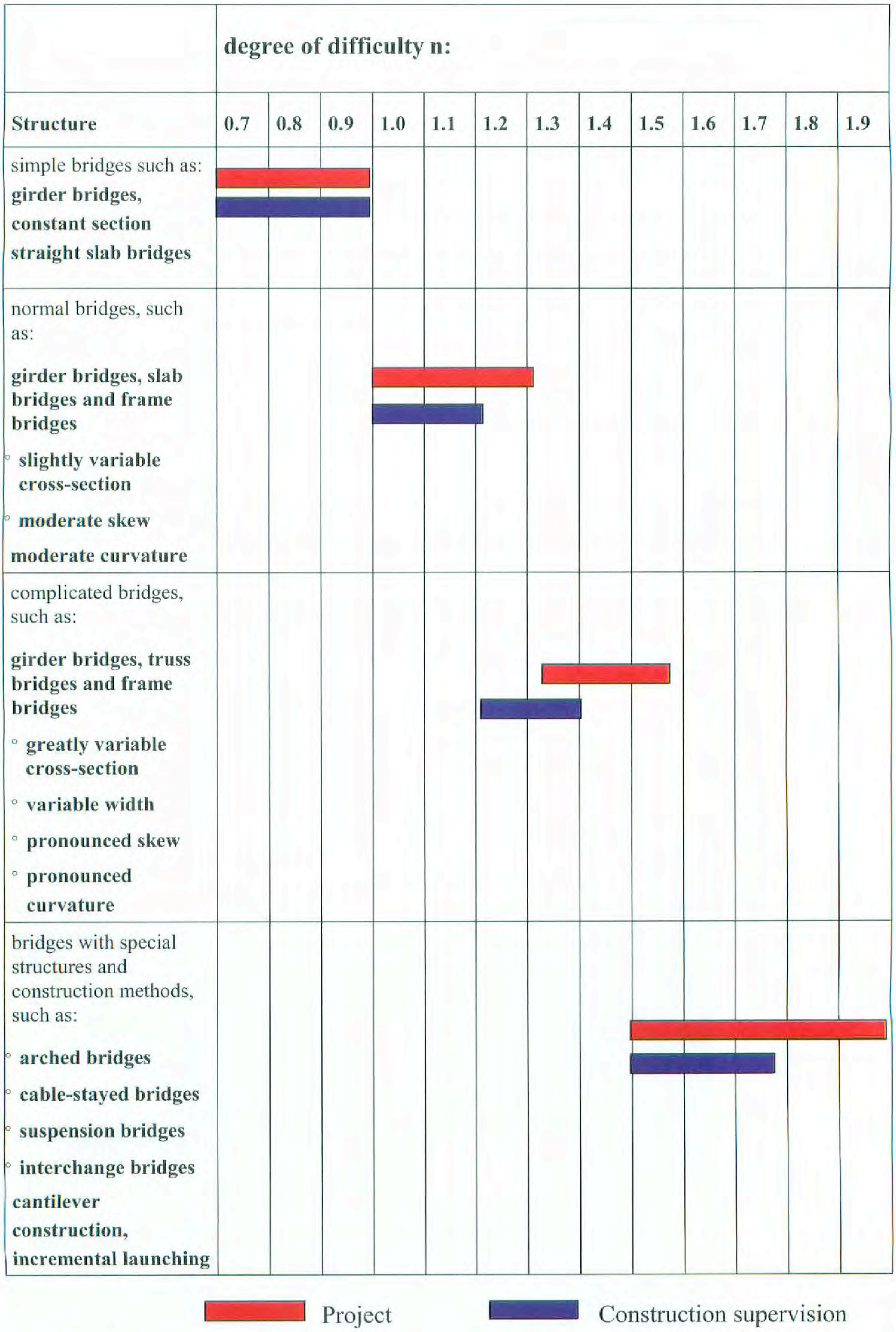


Table 4.2: Example of coefficients of difficulty n

4.5.1.3 Coefficients of division of services

Coefficients of division of services give a standardised distribution between the different project stages of the percentage fees. An example of coefficients of division of services is given in Table 3, defined for complete engineering services for the principal selected consultant for the load-bearing structure of the works.

		coefficient of division of services [%]	
		complete services: project + supervision of construction	partial services: project + technical assistance to client
Project	Preliminary project Final project Invitation to tender and comparison of offers Executive project	20 30 5 30	20 30 5 30
Total for project		85	85
Security verification			5
Detailed verification			10
Supervision of construction	General supervision Local supervision File of the completed structure Supervision of the guaranteed repair of any construction defects and final verification	10 30 5 2	15 (assistance) 5 (collaboration, receipt)
Total for supervision of construction		47	20
Total for project and supervision of construction		**132**	**120**

Table 4.3: Coefficients of division of services (130 % = all services described in table 4.1)

In the case of a partial mission, a weighting of the above coefficients is carried out in order to take into consideration the entirety of services to be accomplished which ordinarily overlap and belong to different project stages. The Belgian rule (FABI) proposes the following modification:

- Preliminary project studies alone +30%
- Preliminary project studies and project alone +60%
- Preliminary project studies, project studies and final project +85%
- Project studies and realisation based on a preliminary project

- study prepared by a third party +70%
- Examination of offers with report based on the study of a final
- project prepared by a third party +15%
- Examination with report of a tender variant exempt from the
- programme approved by the client +5%
- Examination of offers with report for a judging-competition whose
- list of responsibilities has been prepared by a third party +25%
- Collaboration for the supervision of the construction and receipt
- based on a final project prepared by a third party +40%

4.5.1.4 Calculation of fees

Based on these parameters, a general formula adapted for each national regulation permits the calculation of the percentage of the cost of the project which should be taken into consideration for the calculation of fees. For example, the percentage of fees for complete engineering services, including local supervision of the construction, the drawing of the executive and reinforcing plans, as well as site supervision, comes to approximately (coefficient of difficulty n=1):

Determining cost of the structure [USx10^6$]	Average percentage of fees for a normal case (n=1) [%]
0.1	12.3
0.5	10.0
1.0	9.1
1.0	8.6
1.5	8.6
2	8.2
5	7.3
10	6.6
20	5.9
50	5.6

Table 4.4: average percentage of fees for complete engineering services

These percentages are only a general indication of the practices in force and should not be applied without other verification of local conditions. However, they correspond to the real price for a quality service. The current tendency of certain administrations or contractors to continue to lower the above percentages in a systematic manner should be deplored. As has already been mentioned, this practice should be condemned, as it go against a global reduction in construction cost and does not favour the carrying out of quality services.

This remuneration method has the advantage of being adaptable to variations in the cost of the structure. It should be noted that it permits a good approximation of the upper limit of study costs for a large structure, but that it is often insufficient for small, complex structures. It is necessary to have enough flexibility to be able to adapt to the real difficulty of the structure. These payment scales are particularly suited to serious and qualified firms.

A disadvantage of this method is that it does not necessarily encourage a detailed study of material savings or research of economical solutions because of the evolution of the determining cost of the works following these methods. The more time that the selected consultant devotes to the research of economical solutions to reduce the global construction cost for the client, the less he will be paid. Alternatively, a consultant which voluntary increases the project costs due to insufficient study time or incompetence causes the client to lose money and finds himself rewarded by increased fees.

4.5.2 Remuneration based on a fixed lump sum price

Remuneration based on a fixed lump sum price should, in principle, be limited to exceptional cases. However, current unrestrained competition and clients' wishes to have complete control of the total costs before construction begins, have a tendency to generalise this practice which does not necessarily promote the quality of the completed project.

The total amount of the fixed lump sum price is calculated on the basis of a complete description of the services of the selected consultant and on an evaluation of the fees calculated either according to the payment scales presented in Section 4.5.1 or by an estimate of the necessary time to complete the contracted work. The price may be fixed for the entire project or may be established for certain, well-defined stages of the project. In the case of long term contracts, it is preferable to divide the services into several parts, with each part being the object of a separate estimate.

The fixed lump sum price presents a certain cost guarantee for the client, but does not permit an assurance of optimal quality for the project execution. As the selected consultant is limited by a certain budget, he may have the tendency to limit his input on secondary points in order to limit his costs. The chance that the quality of project details and finishing may suffer is increased.

A certain flexibility may be seen in remuneration based on a fixed lump sum price when certain sums of money are defined at the start of construction which recompense or penalise the selected consultant at the termination of the contract as a function of the obtained result or respect for deadlines. Precise criteria for the awarding of these bonuses are difficult to control and the assessment of the actual quality of the services carried out by the engineer remains the responsibility of the client. Different points of view as to the quality of service can merge at the termination of the contract when such a method is adopted.

As a general rule, a fixed lump sum price should not be used as a tool by the client to decrease study costs. In some cases it is advantageous to encourage its use and notably to promote the principle of bonuses awarded if the consultant's study brings about economy for the project as a whole. This is a good method for motivation of the team in charge of the work

4.5.3 Remuneration as a function of time (time tariff)

The third method for the calculation of engineering fees consists simply in keeping track systematically of all of the hours necessary for the completion of the project and then billing for these hours according to an hourly rate agreed upon with the client before the start of the project. These hourly costs are established for each category of personnel involved in the project.

This very simple calculation method is practical in cases of complicated structures where the nature of the contract impedes remuneration as a function of the construction cost. Bridge testing, repair and structural maintenance, so as some small projects, are more appropriately paid for by this method. Certain specific contracts which study special aspects or require expert appraisal are often remunerated according to a time tariff.

However, the client does not have such a good control of the costs of the study before the start of construction and verifying the amount of hours actually needed for the project is very difficult, possibly resulting in certain discrepancies. Also, a quick and experienced engineer may be put at an unfair disadvantage with respect to an average consultant who, for the same work, will spend more time to achieve a less convincing result.

The use of this payment method is favourable as long as the client establishes serious and regular procedures for the verification of the services carried out and the time needed by the consultants, in order that he may remain in control of the costs. In the case of a normal relationship between the consultant and his client in a climate of reciprocal confidence, this remuneration method is often used and prevents numerous discussions.

An average value of time tariff is indicated in the table 4.5. These hourly prices includes all operating costs of the consultant office. They have been determined as an average of the tariff observed in the developed countries. These prices are subjected to some variations, depending on the experience of the worker and on the relative length of the mission. A short worked period is hourly more paid than a long one.

Category	Average price [US$]	variation
International consultant	190	± 10 %
Specialist - Manager - Expert	170	± 20 %
Senior Engineer (+10 years experience)	130	± 15 %
Junior Engineer (5-10 years)	95	± 15 %
Beginner engineer Experimented Draftsmen	80	± 15 %
Draftsmen	65	± 15 %

Table 4.5: average hourly rates for engineering services (base year2000)

4.6 Expenses

In principle, in most European regulations expenses are not included in the calculations of fees and must be paid separately. Expenses which may be passed on to the client usually include:

- costs of documents and reproductions,
- costs for the work of third parties requested by the engineer with the accord of the contracting authorities (testing laboratories, geotechnical investigations, surveying ...),
- exceptional travel costs.

A lump sum to cover all the costs may be agreed upon at the start of the contract in order to avoid a complicated breakdown of costs. Any extraordinary costs are then negotiated and agreed with the client.

4.7 Conclusions, recommendations

A comparative analysis of the three remuneration methods described above does not enable the selection of an ideal method for all situations. However, it is possible to take advantage of the positive aspects of each method by using them for different project stages as a function of the development of acquired data and of project definition. Therefore, the following calculation method seems reasonable:

- preparatory work - preliminary study	**time tariff**
- preliminary project - final project - preparation of tender documents	**percentage of the works (cost tariff)**
- comparison of tenders - executive project - local supervision of construction - execution checks	**fixed lump sum price**

This method permits the selected consultant to act continually as a function of the evolution of the cost of the project until the allocated cost is agreed upon. The preliminary study stage, paid for as a function of time, permits the gathering of basic data and the carrying out of the first analyses of the problem without having to consider the cost of the project, still unknown at that time. The preliminary study also results in a series of variants with construction cost estimates for the different solutions in order to be able to continue the study with remuneration based on a percentage of the works (defined in Section 4.5.1). This change in the method of the calculation of fees permits the client to establish his budget more precisely, and encourages the selected consultant to use all his competence and technical means to establish the final project and the tender documents. Once the offers have been received, the actual cost of the project is known and a fixed lump sum price may be defined for the remainder of the study based on a precise list of responsibilities for the services to be carried out. This has the advantage of permitting the client to control cost, but also of favouring the research of economical and efficient solutions without having negative consequences on the total amount of fees received by the selected consultant.

The use of this method for the calculation of fees certainly requires more effort as it divides the services into several stages, each with a different method of calculating remuneration. However, this arrangement is equitable to both the client and the selected consultant. In spite of the fixed lump sum price fixed for the construction stage, the quality of the execution of the project should not suffer because of the solid bases on which the total lump sum has been decided upon.

From a point of view of relationships, a closer contact between the client and the selected consultant is necessary in order in assure the correct sequence of all of the project stages. A climate of permanent confidence between both parties is not only indispensable for the correct development of the project, but is also a gage of the success of the entire operation.

5 Design process and collection of data

5.1 General

The complete design process of a bridge consists of five major parts, each of which may be divided into several sections. The division of the work is defined by the steps required to achieve the major objectives, as follows:

◦ Global interaction with the road project	1. definition of the project
◦ Bridge conception	2. preliminary design
◦ Output of tender documents	3. detailed design
◦ Construction	4. final design, construction supervision
◦ checking	5. checking process

Each step of the work and particularly the preliminary design needs a substantial amount of technical and other data. These data are necessary to relate the bridge to the local conditions and to design it for the appropriate loads. In order to be complete and rational, a global list of all the required data is given below, before the description of each major step.

This collection of information can be itemised in several parts:

a. Geometrical data about the site and obstacles

- Plan of the site, showing all obstacles to be bridged such as rivers, streets, roads and railroads, pipes and electric lines.
- Plan showing the contour lines of the valley and the desired alignment of the new structure
- Longitudinal section of the ground along the axis of the bridge
- Accessibility and transport facilities for materials, equipment or structural elements
- Topographical data and environmental conditions (valley, mountain, old town, big city...)

b. Technical data about the site

- Soil conditions for foundations (ground characteristics, results of borings ..)
- Geological situation and soil mechanics data
- Weather and environmental conditions, floods, tide levels, period of drought, length of frost period, range of temperature
- Wind measures, aerodynamical conditions
- Earthquake hazards

c. Technical data about the obstacle to be crossed

- Type of traffic
- Conditions for clearance envelope or required flood widths of the obstacle
- Impact hazards (ship - railway - plane), definition of the impact forces and duration

d. Technical data about the new bridge

- Local codes
- Type of traffic loads
- Safety and Serviceability requirements
- Width of the bridge (traffic lanes, walkways, median lane, safety rails...)
- Vertical alignment of the new route
- Drainage requirements
- Technical equipment for water, electrical or communication supply crossing the bridge
- Technical requirements for safety and for maintenance strategy (electrical equipment, cleaning systems, accessibility...)
- Military requirements (destruction of the bridge)

e. Environmental data (integration and ecology)

- Scale of the environment
- Environmental requirements regarding aesthetic quality (important for bridges in towns or for pedestrian bridges which need more special shaping and surface treatment)
- Noise protection (sensitivity of the site, limit values..)
- Classified site

f. Political and Economical aspects

- Materials available or favourite in the country
- Local ways of construction or of philosophy of construction
- Which level of technology can be used in the country
- Analysis of the local market of construction firms

5.2 Description of the major parts of the design:

5.2.1 Definition of the project

Before studying and designing the bridge itself, a first view must be given to the global road project in order to localise the bridge and to define it's interaction with the road alignment. The bridge engineer should take part in the discussions of the road planners and make a contribution to the global definition of the alignment of the future road or way. In this phase, all the geometrical data on the site and the new road must be set up. Depending on the size of the project, general drawings with a scale of 1/1000 to 1/500 should be produced.

5.2.2 Preliminary design

The most important and most creative part of the bridge design process is undoubtedly the preliminary design. For the engineer, this is also the most exciting part of the job in which his imagination gives the bridge its initial shape. In order to succeed in this task, the designer should have conscientiously observed and studied many structures. He should know all the criteria and the typical configurations which lead, for example, to a specific choice of spans and structural systems. An optimal design is the result of knowledge, experience and mature judgement combined with thorough engineering studies and a good knowledge of esthetical forms and proportions.

But bridge design is not an exhaustive science; each construction brings its amount of specific problems due to local conditions. Even though the task or the function bridge may be well defined, the shape and proportions must be adapted, in each case, to suit the particular requirements of the location. The knowledge must be continuously adjusted to the site conditions. Therefore, the designer has first to fully assimilate all the local data. He is also responsible for collecting them or asking the owner to provide it.

For the most efficient work, the designer should get all described data before the beginning of the project and should have studied it conscientiously. Unfortunately, this is rarely the case and the design must be started with only a few geometrical data on the site and the new road which should have been established during the previous phase. Plans of the site, longitudinal sections of the ground along the axis of the bridge and topographical data should at least be placed at the designer's disposal. The geological situation and soil mechanics data are essential information, including the results of a the borings on site. If needed, the engineer asks for supplementary investigations on soil conditions and for 1 or 2 borings near the possible piles foundations. He should see and take some pictures of the bridge site and its environment.

The aim of the preliminary design consists first of determining the critical criteria which may lead to the development of the different solutions. The problem is different if for instance the accessibility of the site is difficult and requires a special method of construction, or if the bridge is located in a open area with determinant aerodynamical effects. The different alternatives must be developed and carefully compared for the most constrained criteria. The final choice of the project which must present the optimal design, is made after a complete comparative study of the following considerations:

- Aesthetic
- Functional
- Safety
- Construction and erection
- Economic
- Local politic

The concept has to fulfil most of these aspects or must be the best compromise of all determining data. The choice of the alternative is made on the basis of drawings, perspectives short descriptions and succinct costs and stability calculations. The general arrangement of the bridge depends more on the geometrical configuration of the site and on the local soil conditions, when the geometry of the superstructure is dictated by the specified route alignment and the required clearances above and below the roadway. The required documents for a good comparison of the different solutions are usually:

- plan view and elevation of the bridge at scale 1/1000 to 1/200 (depend on size)
- typical cross sections (at midspan - on piles) - scale 1/100
- piles and foundations drawings - scale 1/200
- perspectives from several critical view-points
- erection method (sketches or text)
- global cost estimate
- short description
- simplified models in plastic, cardboard or wood of the whole bridge or of specific details (connection deck into pile, section transition...)

Some of these documents are essential for a good evaluation and a faithful representation of the scheme. The best esthetical comparison of the solutions is undoubtedly achieved by the construction of very simplified models where shapes and proportions can be readily evaluated. The same can be done with the perspectives and with computer design software, but is very sensitive to the choice of the view-point, the way of treating the depth of the drawing and the shadows. All these documents are also very helpful to the client to assist him with his choice.

At the end of the preliminary design the client and the designer choose together the final project and usually prepare the competition data with the documents listed above.

Figure 5.1: Designing a bridge in sensible environment needs a complete set of data on local conditions. Pedestrian bridge over the Swisslake – Czech Republic, [22]

5.2.3 Detailed design

The detailed design constitutes the most significant amount of work required for the bridge project. The aim is to satisfy the structural behaviour of the structure regarding the ultimate and the serviceability limit states. The dimensions of the elements are finalised and the necessary prestressing and reinforcement are determined. These analyse are carried out on the basis of the dimensions determined in the preliminary design, using an elastic or an elastic-plastic analysis. The global static and dynamic stability of the bridge are also checked. Such analysis requires all data regarding the forces acting on the new bridge, that is to say statical and dynamical loads, wind pressures, floods, tide levels, earthquake hazards, impact hazards. The complete geo-technical information and soil mechanics data must be available with a sufficient amount of borehole results. All these data must be precisely defined. If needed, complementary studies for instance wind measures on site, new borings for geo-technical investigations or flood measure are required by the designer.

The structure must not only satisfy the structural requirements established by the owner or the applicable specifications, but also the geometry dictated by the specified route alignment and by the required clearances above and below the deck. Such data are dependent on whether the bridge is to carry railway, highway, or airplane traffic or whether it has to cross over a navigable body of water, a highway or a railway. The designer must work with precise plans of the site showing all obstacles to be bridged. Depending on the size of the project, the establishment scale of this documents remains between 1/1000 and 1/100. Some specific areas for instance abutments or piles layout should be drawn to a greater scale.

The conceptual part of this step is to consider all connection details, drainage and equipment of the bridge including lighting, sidewalks, guard-rails, expansion joints, bearings, surfacing and waterproofing. The results of these considerations is drawn in a complete set of plans to scales varying from 1/10 to 1/200 and is added to the general drawings of the structural elements. The precision of this phase must be sufficient to determine all material quantities and to write tender documents. For the price estimation, a range of ± 5% on quantities is usually reached at this stage.

The tender documents to be established after the detailed design are indicated as an example in the following non exhaustive list:

Documents:

- General:
  - Letter of invitation
  - Instruction to tenderers
  - Form of contract
- Scope of work
  - Technical report
  - Architectural report
- Design requirements, specifications
  - Loads, design specifications
  - Materials specifications
  - Geo-technical specifications
  - Environmental requirements
  - Construction requirements
  - Special requirements
- Quality system
- Bill of quantities

Drawings:

- Alignment
  - General: Keymap and longitudinal profile
  - Bridge: Plan and longitudinal profile
- Bridge superstructure
  - General arrangement
  - Typical superstructure cross sections
  - Superstructure details
  - Bridge deck furniture, drainage
  - Bearings schedule
- Bridge substructure
  - Substructure, general arrangement
  - Pier foundations
  - Pier shaft, details
  - Abutments, general arrangement
  - Abutments, details
- Construction methods
  - Substructure, construction method
    Superstructure, construction method
  - Time schedule
- Mechanical and electrical equipment
  - General, power supply, single line diagram
  - Electrical and mechanical installations
  - Traffic control system, layout

5.2.4 Final design, construction supervision

The final design consists of the establishment of all documents needed for a correct construction of the bridge. All details must be studied and all results should be drawn up in a complete set of drawings. The total number of drawings necessary for this stage depends on the size and on the complexity of the bridge. The bridge construction is an evolutive process where drawings should be permanently adapted to the decisions and the changes, according to its evolution.

To finalise the project, the data listed above must be available and include the results of complementary measures requested by the designer. The final statical and dynamic design of the structure is based on these data. The reinforcement and prestressing are dimensioned using the internal forces issued from the global and local structural analyse. All construction stages should be checked regarding the final erection process. The determinant forces or displacement used for construction supervision and geometry control must be listed at each stage. A systematic process of construction survey should be set up in order to follow carefully the erection and to guarantee a good quality of the structure. The complete design and supervision process including the rules regarding the checking and distribution of each document can be described in a document called " project procedures manual ".

This document, established as an internal status, consists of the quality assurance system of the design group.

Figure 5.2: Detailed Design of a bridge situated in severe conditions requires special investigations like wind measures, water current measures, specific natural and environmental observations (fauna, flora...). Example of the Oeresund bridge – Sweden - Denmark [3]

5.2.5 Checking process

The technical checking process is an additional service which consists of ensuring the checking and the evaluation of a project established by a third party for the project owner. These checks may be divided into two different levels:

- safety checks concerning only the project and its conception
- a complete check of the entire project, its conception and its construction

The aim of the checking process is to reduce the risks of a malfunction or eventual damage to the structure which could be due to an estimation error on the part of the designer or to a poor basic calculation hypothesis or to the conception assumed by the author of the project. The subjects of stability and durability of the structure are the main topics addressed.

No fees for project development or construction supervision are ever included in this task. The checks are carried out in a spirit of collaboration and through dialogue with the builders of the project who retain all of their initiatives and prerogatives. No intervention by the person carrying out the checks should appear as a substitution of the designer. The tasks are clearly distinct and complementary. The observations and points of view should be exchanged in a constructive spirit leading to improved quality of the structure. Each party should contribute his experiences to the benefit of the structure.

The process consists first of checking the study, the plans and the contract agreements established by the authors of the project. Qualified specialised engineers in the various sectors included in the project examine the technical options which form the basis of the project, the results of field tests and the foundation systems, the type of structure, the basic hypotheses and the calculation methods, the executive plans, the choice of materials and the prescriptions carried out with a critical but constructive spirit. No additional calculations are undertaken, only a few rapid checks in order to get a feel for and to establish the validity of the designers' results.

In order to be able to be carried out efficiently, the checking process must begin at the preparatory study phase. The person carrying out the checks may thus discuss all points with the designers at the moment in which the main lines and the basic hypotheses of the project are being decided upon. Thus the need for subsequent laborious and detailed information meetings is often avoided. The choices and successive decisions by the designers may thus be analysed directly and, in case of differences of opinion between the designer and the person carrying out the checks, immediate discussion is possible, which permits the orientation of the rest of the development of the project along the solid concepts accepted by the entire group of specialists. Through such rapid intervention a good number of false starts are avoided and precious time is saved.

In the case of a complete check, the task continues during the supervision of the construction by regular visits to the construction site, factories or workshops, in order to ensure that the production conforms to the plans and edited documents. These regular checks concern the correct production and the observation of accepted construction practice as well as the quality of the materials used and the respect of the contract agreements defined during the project. The frequency of the visits depends on the rapidity of the work progress and the complexity of the completed structure.

6 Conceptual design

6.1 Introduction

As already shown in the previous chapter, the preliminary design aims at the definition of the conceptual design of the bridge ; the basic idea consists of comparing different solutions which differ by :

- the longitudinal configuration, i.e. the distribution of supports and the corresponding distribution of spans ; the structural type (beam, arch, cable-supported bridge...) ; the main dimensions (structural depth of the different bridge elements) ;
- the definition of the cross-section of the different structural members ;
- the construction material or materials ;
- and the erection technique, including the definition of the major steps of the construction sequence.

All these basic elements of the conceptual design are correlated : the dimensions of the deck and its shapes depend on the construction material ; the structural type is directly and strongly dependent on the material and the construction sequence. This is why the preliminary design and the selection of the structural solution are based on the experience and intuition of the designer, who should know all the criteria and the typical configurations which lead, for example, to a specific choice of spans and structural systems. A good conceptual design is the result of knowledge, experience, intuition and mature judgement combined with a clear view of the flow of forces, a preliminary evaluation of the main loads, a global perception of all engineering aspects of the structure and a great feeling for aesthetical forms and proportions as well as of integration into the site.

Of course, there is no computation made to support the different solutions which are compared, except if a very specific problem is foreseen ; the selection is based on an engineering judgement supported by experience and competence.

The second part of this document - Part 2 - is totally devoted to the conceptual design of classical prestressed concrete, girder bridges which constitute the very large majority of medium span bridges in prestressed concrete ; in this chapter we shall give a general view on the conceptual design of all types of bridges.

6.2 Motorway over-bridges and under-bridges

The motorway overbridges constitute a very important class of bridge, by their number and the impact on the landscape produced by their repetition.

Certainly, the most important decision concerns the number of spans and the location of supports. Three basic solutions can be compared :

- a classical four-span structure, with a support in the middle of the motorway ;
- a two span bridge, with a support in the middle of the motorway and with strong abutments in the embankment on each side ;
- and a three span bridge which completely opens the view on the motorway and is for this reason, by far, the most appealing solution.

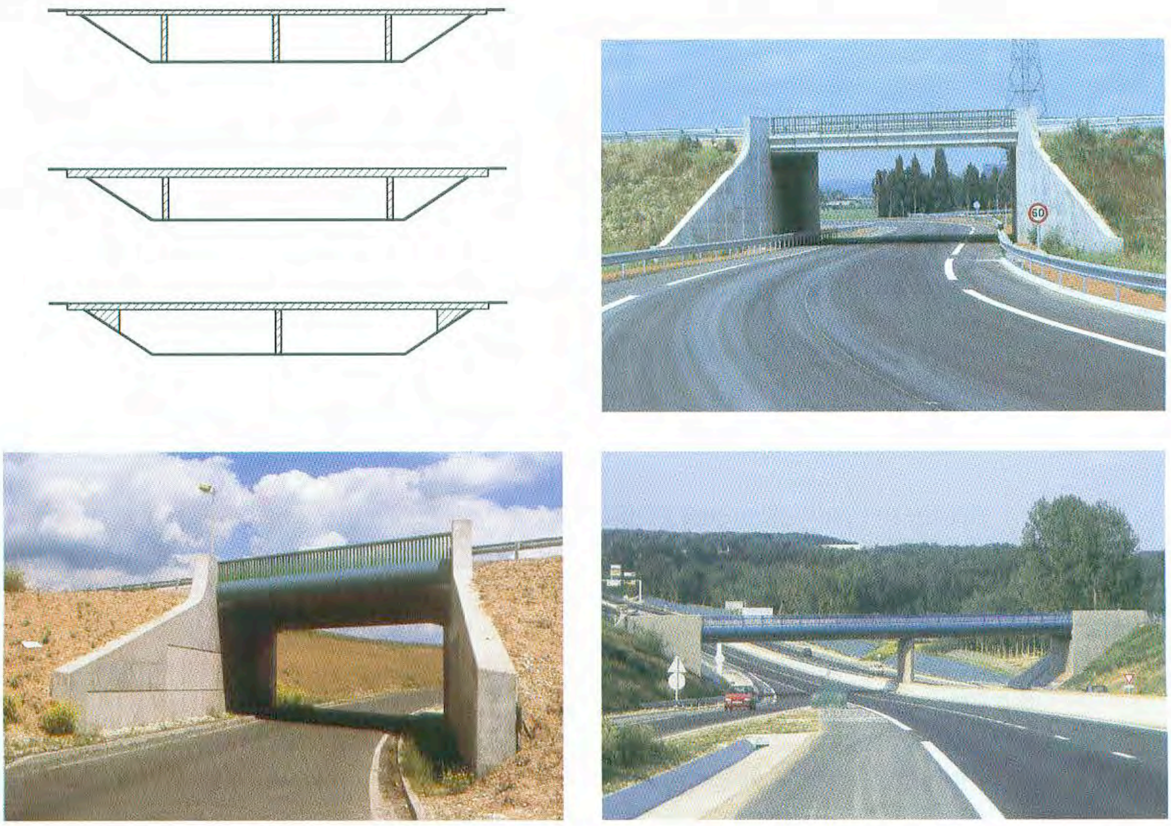


Figure 6.1. Different bridge types for a normal path or a narrow motorway [9]

Of course more specific designs can be developed, such as bridges with inclined supports, depending on the precise site conditions: site configuration, soil characteristics, etc.

The difference can be enormous between a standard solution such as a four-span bridge, even with a decoration ; a clear and simple bridge which opens the site and looks light and clean ; and an original structural solution which gives to a bridge a very special look and a great personality.

Much care in details to produce a perfect coherence between all structural elements, a high quality for all fittings and equipment (hand-rails, barriers, bearings,...), possibly some very light decoration in agreement with the structural system or an agreeable arrangement of local materials, for example, can transform a simple but very clean solution into an excellent one.

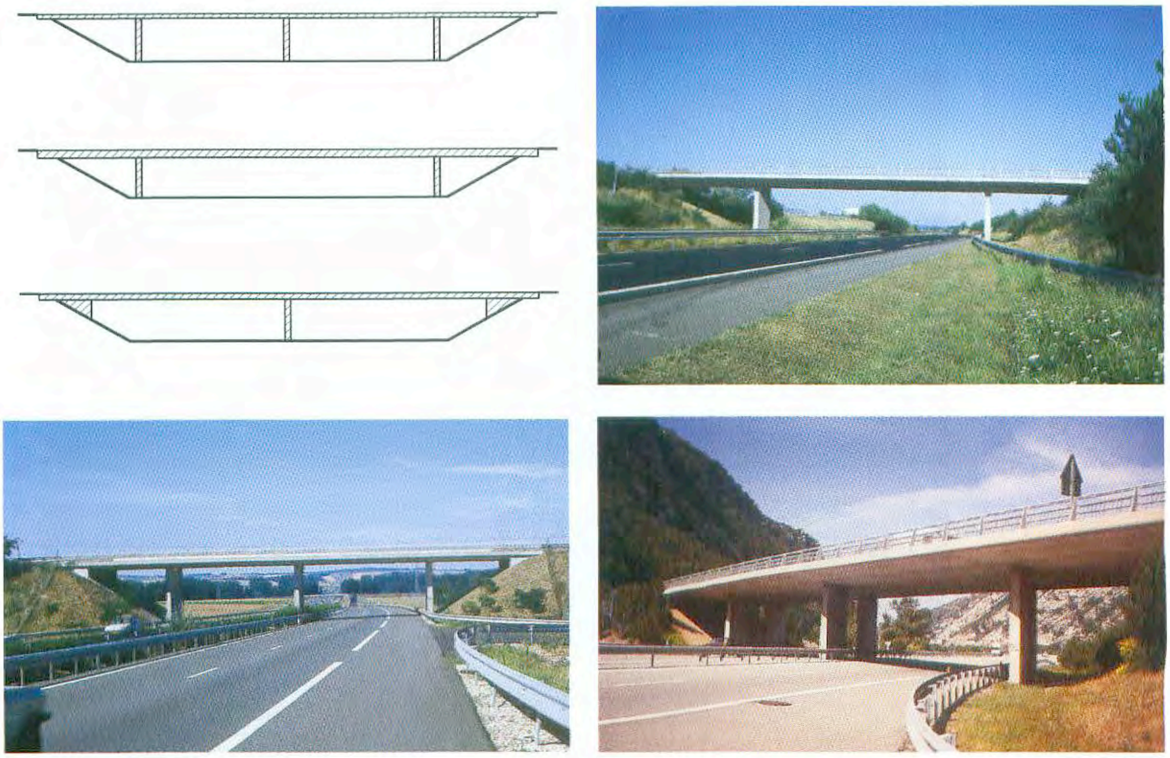


Figure 6.2. Different bridge typesfor a wide motorway [9]

The same can be said with motorway (or highway) under-bridges. For these bridges, the shapes and surfacing of retaining walls can produce excellent designs, or very poor ones.

Clearly, the collaboration with an architect can be extremely helpful for all these almost standard bridges; but he must not aim at producing an artificial - and thus often ridiculous - decoration, but at helping a good designer in selecting pleasing shapes, producing light and shadow, colour and at the end the desired elegance (figure 6.3).

6.3 Structural types

Of course, the definition of the conceptual design is much more creative for large bridges, with the multiplication of the possible structural types which, evidently, are related to the span range and to the selected material.

6.3.1 Some sites are specially adapted to specific solutions. Arch bridges, for example, are well adapted to rather deep valleys when foundations can be easily installed on the rock, at a short distance from ground surface (figure 6.4). But the design of an arch must also consider construction, and it is often impossible to install the arch springings in the slopes due to the difficult access conditions for construction equipment.

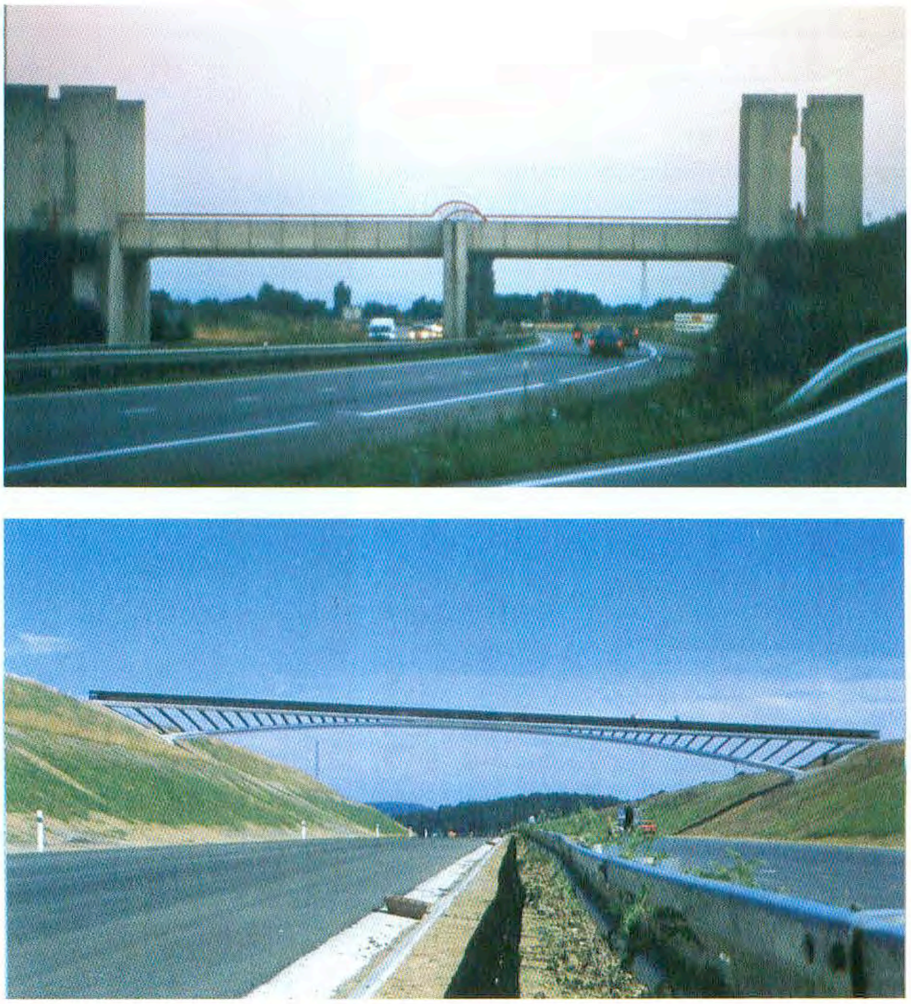


Figure 6.3. Contrast between artificial poor decoration and structural beauty[3], [19]

Figure 6..4 Bloukrans Bridge in South Africa [1]

We can evoke at the same time, though they are structurally very different, bridges which have a similar look, like bow-strings. A span can be isolated with a bow-string structure, with access spans of a different type on one or both sides ; but a continuity can also be provided. In modern designs, the girder deck can be continuous, but in some traditional constructions the arch in the bow-string can be a truss continued below the road or railway in the access spans to produce the desired continuity (figure 6.5).

6.3.2 Bridges with inclined supports on each side of the main span constitute an elegant alternative to arches, specially when the main crossing is extended on one or both sides by some access-spans, a situation which is not always favourable to arch types.

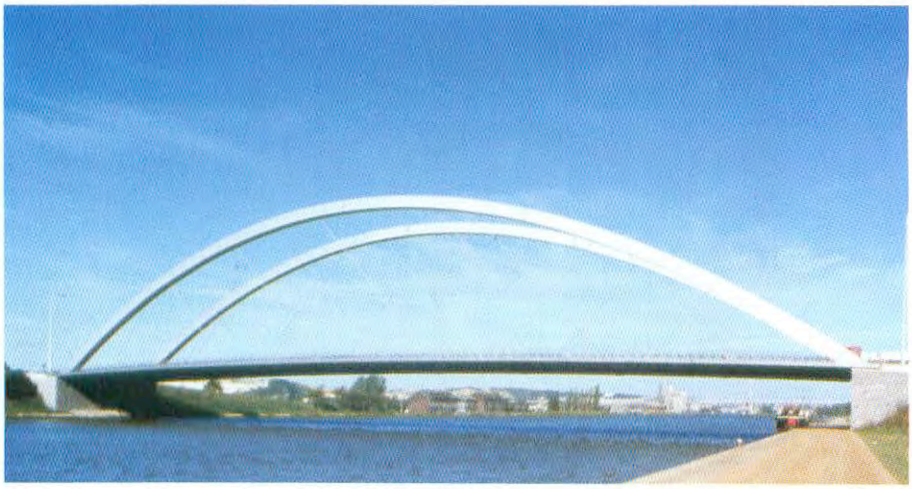


Figure 6..5. Bow-string bridge: Haccourt bridge in Belgium [2]

Of course, this solution with inclined supports can be varied and opens up many opportunities for the imagination and creativity of the designer. This results, for example, in V-shaped piers which produce, when desired, a strong resistance to longitudinal forces. An extreme situation is with Y-shaped piers, which raise more erection problems but which can help to reduce bending forces in the superstructure with rather long spans

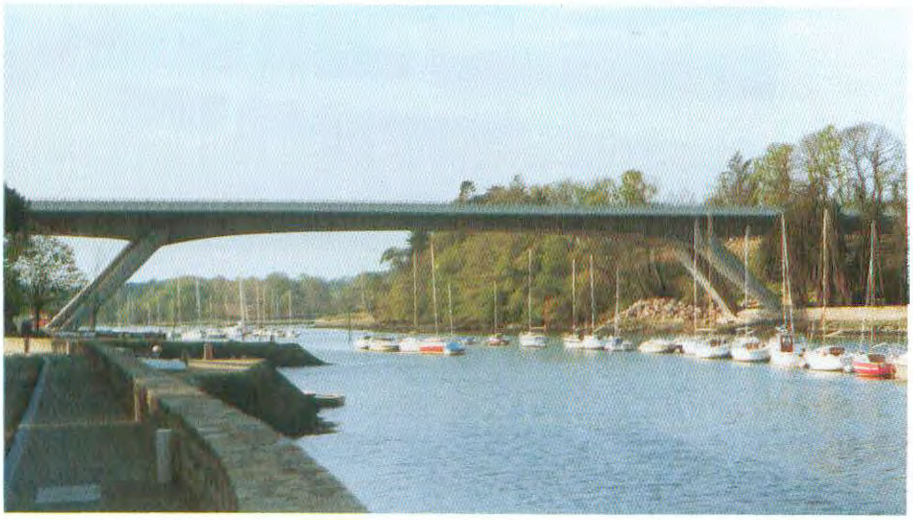


Figure 6.6. St Goustan bridge – Auray, France [1]

6.3.3 For long spans, cable-supported bridges are by far the most adaptable. Cable-stayed bridges are nowadays preferred for spans between 150 metres and almost 1000. Of course, they are in competition with box-girder bridges in prestressed concrete until about 200-250 metres, and with steel box-girder and truss bridges until 300 metres; cable-stayed bridges allow very slender decks compared with the structural depth of box-girder bridges which increases with the span, resulting in very heavy proportions for the longer spans, especially when the bridge is rather low above the ground or the water.

Figure 6.7. Skarnsundet ridge – Norway; 530 m span with a concrete deck

Figure 6.8. Diepoldsau Bridge – Switzerland; first cable stayed bridge with slender deck (simple 46 cm thick concrete slab)

The very large variety of shapes in cable-stayed bridges - deck, pylons, distribution of cables... - and the very large number of parameters open many possibilities and give the designer the opportunity of elegant designs. Cable-stayed bridges are among the most modern structures (figure 6.9).

6.3.4 For the longer spans, greater than 800 or 1000 metres, suspension bridges remain the preference. Their shapes are classical and well accepted ; but suspension bridges are handicapped by the high cost of cables and of anchor blocks, except when the cables can be directly anchored in the rock. In addition, anchor blocks may be intrusive in the landscape.

Suspension bridges can also find some elegant applications for smaller spans, specially for pedestrian bridges (figure 6.10).

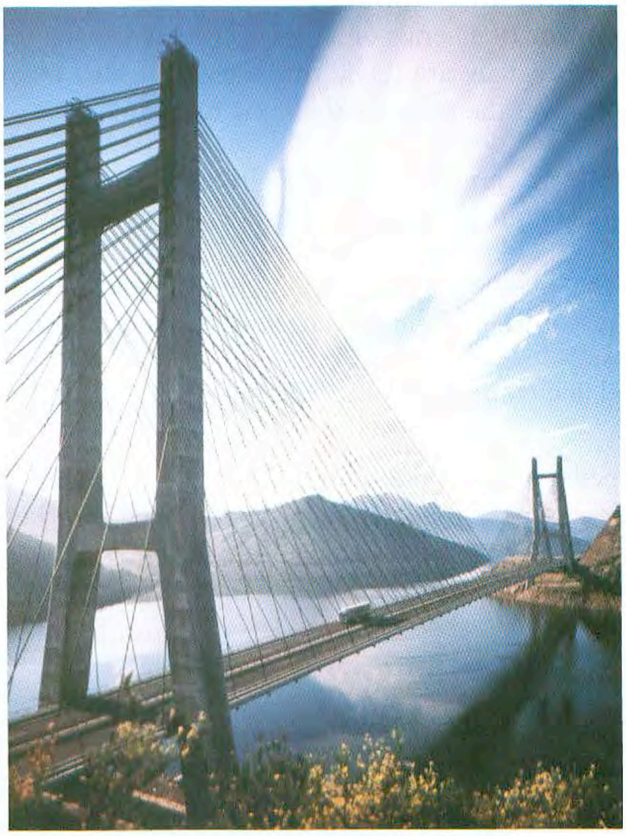


Figure 6.9. Barios De Luna bridge – Spain; another example of an elegant design [10]

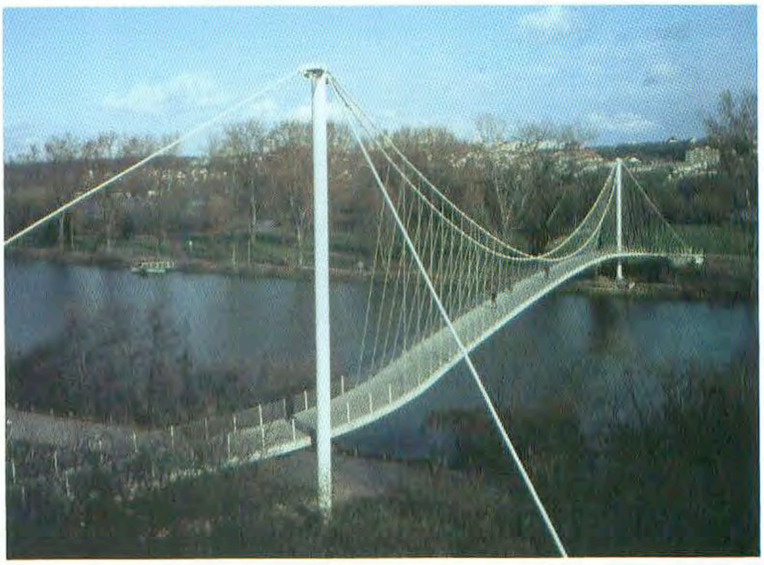


Figure 6.10. Max Eyth See Bridge – Stuttgart, [11]

6.3.5 But the very large majority of bridges comprises girder bridges. Many different types of girder bridges exist, which differ by their cross-section and by the construction material, steel or concrete, including of course composite structures.

The girder can be continuous or divided into a series of simply supported spans. Clearly, unless very specific reasons require it - such as probable settlements due, for example, to mining activities - the superstructure should be fully continuous to reduce the number of expansion joints as much as possible because joints are a source of traffic discomfort and of maintenance problems. When the superstructure has to be made of a series of simply supported spans for constructional reasons - for example when they are made of concrete precast beams -, the slab must be made continuous over the piers for the same reason.

In railway bridges joint positions may be influenced by the need to accommodate rail joints which must be located away from curves.

But the position of expansion joints is also influenced by the construction technique, for example in bridges built by the cantilever method.

6.3.6 Steel and composite girder bridges mainly differ by their cross-section :

- I-shaped girder bridges, of constant or variable depth. The most successful solution is now made of two parallel I-shaped beams connected by a limited number of light cross-beams. But in some of them the slab can be directly supported by floor-beams or with the help of struts.

Figure 6.11. Example of a concrete deck with steel girders [1]

- Box-girder bridges, of constant or variable depth. Generally, the bridge is made of a unique two-web box-girder, possibly with transverse floor-beams or with a system of struts to support the slab on each side. But it can also be made of two parallel box-girders.

Figure 6.12. Examples of composite deck with Steel Box girder. Viaduct of Pompadour – France [9]

- Truss bridges, of constant or variable depth.

Figure 6.13. Example of a composite deck with longitudinal steel Truss beams. Viaduct of Lully – Switzerland [12]

6.3.7 Prestressed concrete bridges mainly differ by erection techniques which strongly influence shapes and proportions. Part II of this guidance will give a precise description of each erection technique of prestressed concrete bridges.

The most important families are as follows :

- Bridges cast in situ on centering or scaffoldings, in one single operation or in a series of successive steps, depending on the bridge size. Of course, this construction technique allows for greater freedom as regards structural shapes which are only limited by shuttering facilities. As an alternative, some parts of the bridge can be built in one position and later placed in the final one, by rotation for example.

- Bridges made of precast beams, prestressed by pre-tension or by post-tension for longer spans. Such a technique produces strong limitations in shapes for deck and piers.
- Bridges built in situ span by span on a launching casting beam ; or made of precast segments installed and prestressed span by span with the help of a launching beam of the same type.
- Bridges built by the incremental launching method, which must be of constant depth and fulfil some very strong geometric requirements to allow for launching.
- Bridges built by the cantilever method, either with cast in situ or precast segments, with as an alternative bridges built by the progressive construction method, mainly with precast segments, either with the help of temporary stays or any other system.

This is why the erection technique is a major element in the design of a prestressed concrete bridge, and why a serious design cannot be developed with no consideration of the construction conditions.

But, on the other hand, construction conditions must not undermine the major design goals which are - following Vitruvius - firmitas, utilitas and venustas. When contractors propose alternatives to change the erection technique, or to adapt the design to their existing construction equipment, it must not alter the elegance of the structure nor affect operation conditions nor durability.

The recent development of design and build (or even design-build and operate) contracts gives major responsibilities in the design to contracting companies which logically orient the design for an efficient and economical construction. But the importance given to construction technique and equipment must not be such as to produce poor or inelegant structures.

6.4 Distribution of spans

The distribution of spans cannot be separated from the structural type, but we shall here mainly concentrate on the most frequent situation with girder bridges.

In some cases, for example when the bridge has to cross a wide navigation channel and to be protected from ship collisions, the location of supports and the distribution of main spans cannot be varied. But in most cases, there is a choice between shorter or longer spans within the overall bridge length.

As regards cost, it is the sum of substructure (piers and foundations) and superstructure. Substructure is cheaper for longer spans (fewer piers) while superstructure is more expensive. Hence there may be an optimum range for the spans to produce the lowest cost. When ground conditions are poor and foundations expensive, either due to their depth or to their construction conditions - for example in a deep sea channel or in a difficult river -, or when their size has to be increased due to special situations such as resistance to ship collisions, longer spans will be more appropriate. This may be the case also when piers are very high or when their construction is difficult due to specific situations ; but this situation is rather rare since the relative cost of piers is normally very low (5 to 10 % maximum).

On the contrary, when ground conditions are good, the shortest spans will be the cheapest. But proportions have to be well balanced for a suitable inscription into the site and for elegance : the length of spans and the depth of piers must be selected considering the proportions of the polygons drawn by ground, deck and piers. The distribution of spans may be influenced by the site, either to adapt to some specific constraints (such as avoiding roads, waterways or other obstacles) or to produce pleasing proportions ; for example to cross a valley the depth of which increases regularly from each side to its centre, the span length may increase from each end to the central span, proportioning the span length to the vertical distance from deck to ground.

Of course the structural depth of the deck is part of the final decision concerning the distribution of spans, as well as the possible variation of the girder depth in the spans : as already shown, all design parameters interact in the selection of the solution.

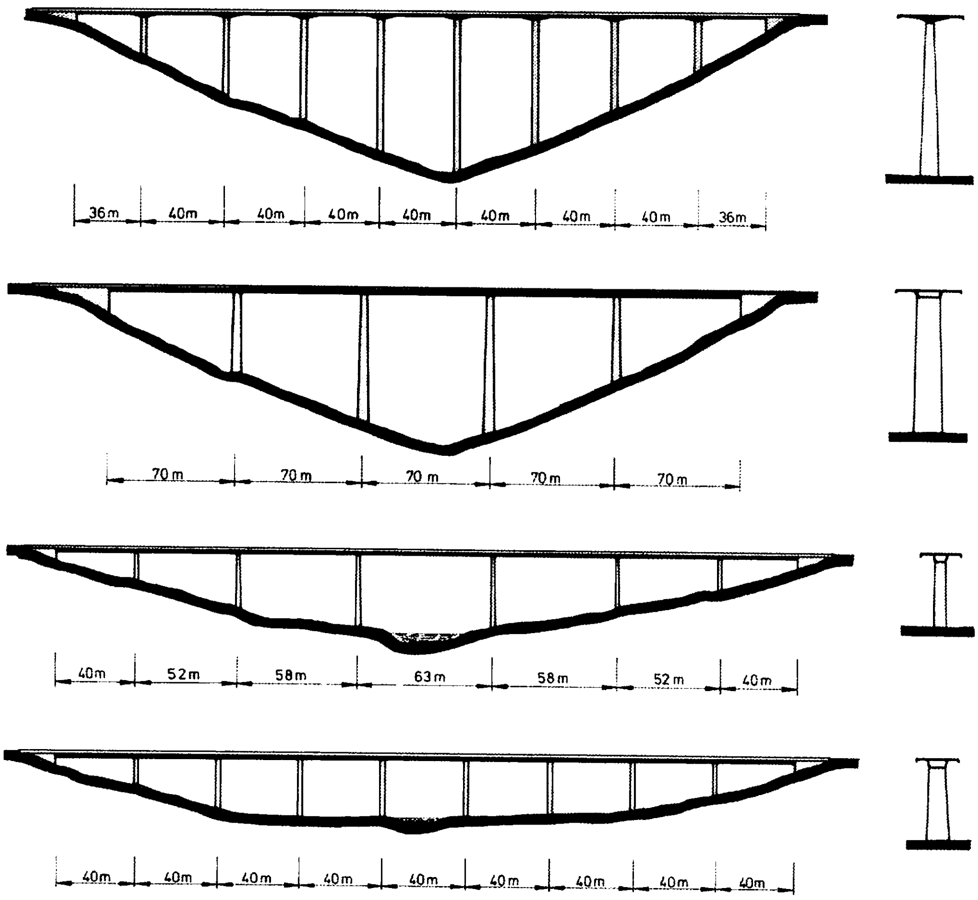


Figure 6.14 Influence of the distribution of spans on the bridge aesthetics [6.2]

6.5 Structural depth of girder bridges. Depth variations

The structural depth of girders in girder bridges (but also of all other structural elements such as piers or deck in complex structures including arches, bow-strings and cable-supported bridges) depend on very many factors :

- the selected material,
- the structural type, as previously discussed,
- the selected cross-section.

The classical slenderness ratio, ratio of the structural depth to the span, is given for each bridge type in the second part of this guidance book. We must simply say here that the slenderness has to be greater when the bridge is at a small vertical distance from the ground, to produce an elegant proportion between the superstructure and the void below.

Depth variation must be considered also : the girder weight is seriously reduced in concrete bridges at midspan when the girder depth varies (generally with a parabolic shape) ; this reduces costs in the deck, by reducing bending moments and thus the number of prestressing tendons, and also in foundations ; but, as a counterpart, this makes construction more complex, specially as regards shutters. Clearly, the longer the span, the greater the advantage.

Depth can vary parabolically, as previously said, but also according to a cubic formula ; the depth variation is then very rapid in the support vicinity which can become critical. An alternative exists with linear gussets (a linear variation of the structural depth close to supports, on a distance which is about 15 or 20 % of the span length), or with long parabolic gussets (a parabolic variation of the structural depth close to supports, on a distance which is about 20 or 25 % of the span length).

A parabolic variation is often preferred when all spans have the same length, when the longitudinal profile and the alignment give to the bridge a quiet and simple appearance. On the contrary, parabolic variations must be avoided when the spans have different lengths due to the crossing conditions : the parameters of the parabola would be different in all spans, with some limited construction problems, but what is more important with a very poor aspect resulting from the disorder in lines and proportions. Such a situation is very well adapted to the use of linear or parabolic gussets which can receive the same length and shape in all spans, the difference in span lengths being covered in the central part of the spans which is of constant depth.

A constant depth is finally the better solution when the site is extremely complex, with a series of different crossings, with very different span lengths : the uniformity of the superstructure gives a new unity to the site and improves the situation. A special case is when the bridge is made of two parallel decks with skew crossings of rivers, roads or railways : a constant depth is absolutely necessary to avoid the disorder produced by the longitudinal mis-alignment of supports between the two box-girders. The same is to be made when the curvature of the bridge alignment is very large : with a depth variation in the same time, shapes would become confused and unsatisfactory.

In some situations, it is interesting to associate a variable depth in the main spans, to mark the passage above water for example, with constant depth in side spans on both sides, to limit their visual importance, or to adapt to a very different situation (smaller distance above ground ; variable span length due to the existence of obstacles...).

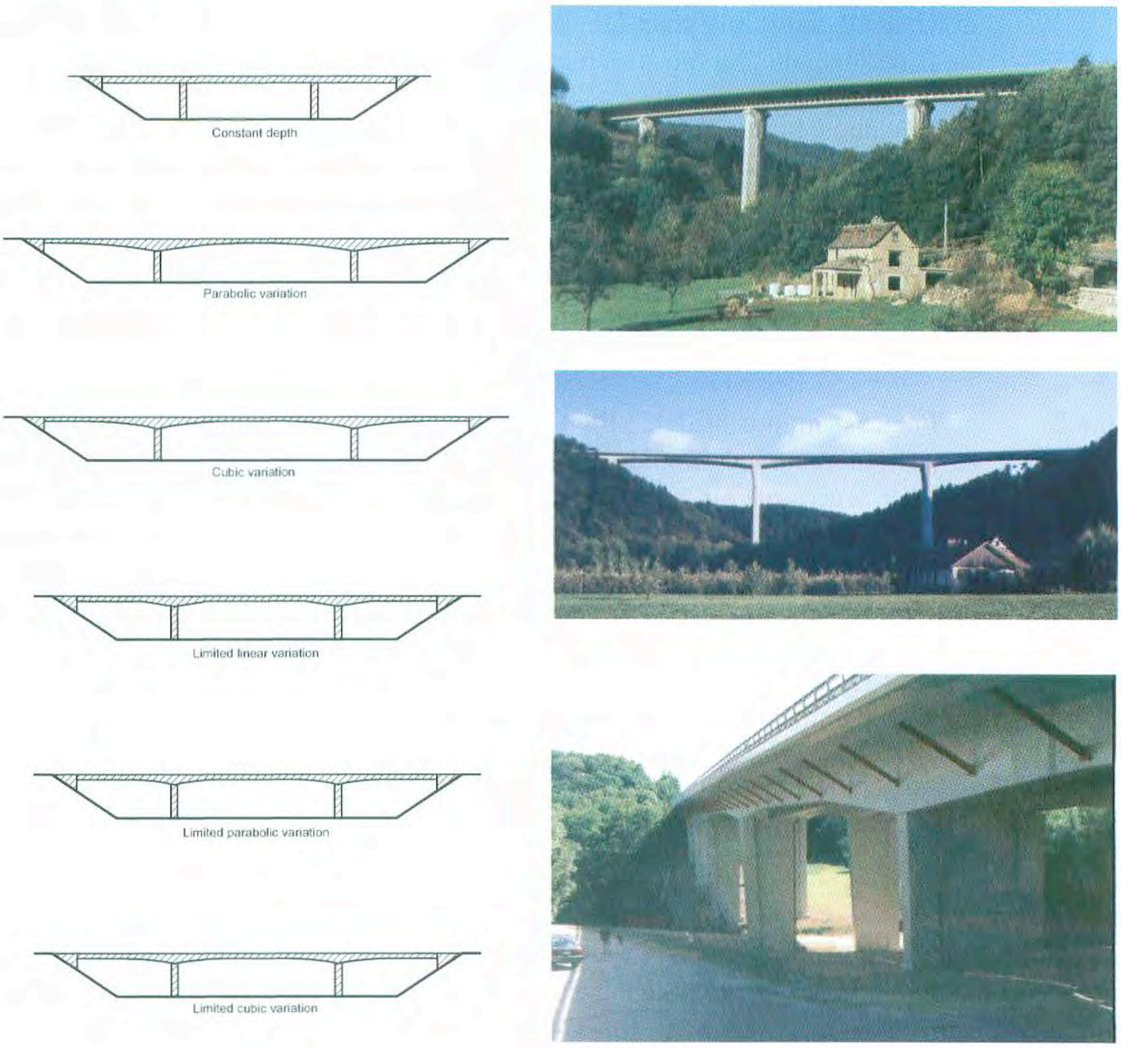


Figure 6.15. Examples of parabolic and linear variation of depth – Linear: Rioulong bridge – France [1]; Parabolic: Mentue bridge – Switzerland [3]; Limited linear: Versoix bridge – Switzerland [3]

6.6 Cross-section of prestressed concrete bridges

As for steel bridges, the design is influenced by the relative costs of materials and manpower ; the historical evolution of structures is totally governed by these economic parameters.

6.6.1 In the pioneer applications of reinforced concrete before the second world war, when cement and reinforcement steel were expensive and labour relatively cheap, it was more efficient to design complex shapes to limit the volume of concrete ; at that time, decks were often made of a very thin concrete slab with small rectangular ribs in both directions. The superb bridges by Robert Maillart, or those by Albert

Caquot for another example, give an excellent image of these constructions. The situation is completely different nowadays in developed countries were labour is extremely expensive and materials very cheap.

Shapes are thus made as simple as possible to ease construction and limit working time, even if this increases the necessary quantities of materials. Of course, as we shall show, the span length will also have an influence on the design due to the need for reducing the structure weight.

But another decisive factor appeared : the labour cost is much cheaper in a factory than on site for many reasons. At first because the equipment in a factory is much more efficient and allows for important reductions in working time ; also because workers are protected from weather hazards ; and mainly because standardisation produces large economies. This is why a clear difference must be made between cast in situ bridges and bridges made from precast elements, fabricated in a fixed factory or in a temporary one installed close to the site.

6.6.2 For cast in situ bridges, the design is totally governed by the span length : for very short spans the simpler - and heavier shapes - are preferred, because they can be built with an extremely limited manpower ; when the span length increases, shapes must become more efficient to reduce the weight in order to limit bending forces ; when the span length becomes very large, priority must be given to the structural efficiency, to save weight. This is why the shape passes, progressively, from rectangular slab - for very small spans - to classical box-girders.

Two parameters can illustrate this evolution :

- the geometric efficiency coefficient :

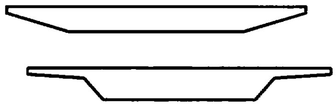
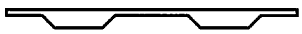
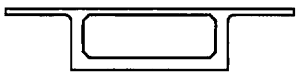
$$\rho = \frac{I}{Bvv'}$$

where I is the flexural inertia, B the cross-section area, v and v' the distances from the centre of gravity to extreme fibers. It varies from 0.333 for a rectangular slab to about 0.55 - 0.65 for a classical box-girder ;

- and the rate of placing concreting, which is a good indication of the construction ease and, indirectly, of the labour time.

To give an idea, we can consider that the cross-section type changes for a more efficient shape when permanent loads produce more than 75 or 80 % of bending moments at the critical cross-sections.

Of course, the transition between the rectangular slab to the rectangular ribs is extremely progressive ; the real jump corresponds to the passage to a box-girder.

Name	Cross-section	Classical span-range (m)	ρ	Concreting rate (m3/hour)
Rectangular slab		< 12 to 15	0.333	25-30
Slab		10 to 25	0.33 to 0.36	25-30
Ribbed slab		20 to 35	0.35 to 0.38	25-30
Rectangular rib		30 to 45	0.36 to 0.42	20-25
Box-girder		> 40 to 45	0.50 to 0.65	6-10

6.6.3 Passing to bridges made from precast elements, we have to consider three different families : bridges made from precast girders, bridges made from segments and bridges made from complete precast spans.

In all cases, and mainly for the first two families, prefabrication allows for more sophisticated cross-sections, designed to increase structural efficiency and to decrease weight. This is specially clear for precast girders : with thick lower flanges, the geometry efficiency coefficient can reach rather high values, almost those of classical box-girders.

The construction of bridges from complete precast spans is a relatively new trend, specially adapted to large projects which allow for the use of very heavy lifting equipment.

6.7 Incorporation of steel elements in concrete bridges. Composite designs.

6.7.1 Engineers must always remain open-minded, and on some occasions concrete bridges can be made more efficient, more economical, simpler or lighter by the incorporation of steel elements.

As a first example, the struts which support the overhanging deck elements can be steel tubes or even stainless steel tubes like in the Val-Benoit bridge in Belgium.

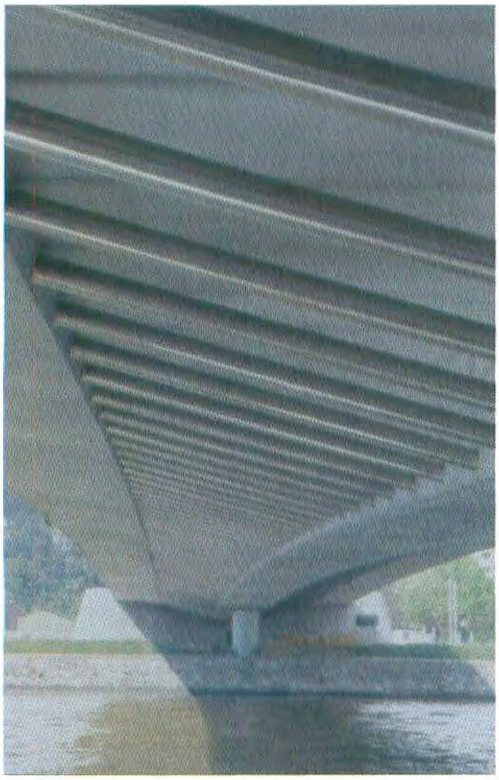


Figure 6.16. Beautiful details of the stainless steel struts of the Val-Benoit bridge - Belgium [2]

Complex structures, such as cable-stayed bridges, give many opportunities for introducing steel elements :

- the struts inside the box-girder, which transfer the cable-staying force from the cable anchorage to the lower node of the concrete webs, can be in steel as in the Wandre and Ben Ahin bridges ;
- cables can be anchored in the concrete towers through steel anchorage elements, such as in the Alex Frazer Bridge to the Anacis Island, or in steel anchorage boxes as in Normandy bridge or Wandre and Ben Ahin bridges ;

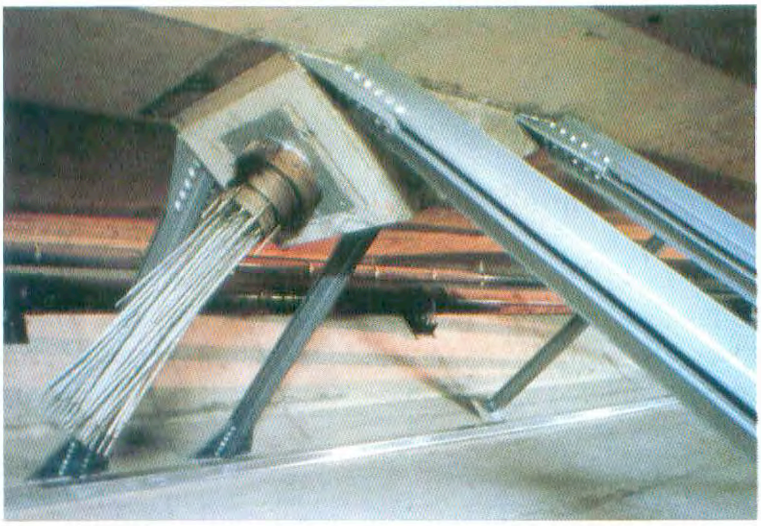


Figure 6.17. Inside strut of Wandre Bridge [5]

Figure 6.19. Steel anchorage structure in Normandy bridge pylon [9]

- transverse floor-beams which connect the two main concrete ribs, in cable-stayed bridges of this type, and which support the concrete upper slab, can be in steel as in the East Huntington bridge, more recently in the Vasco de Gama Bridge over the river Tagus in Lisbon or in the Schaffhausen Bridge over the river Rhein in Switzerland

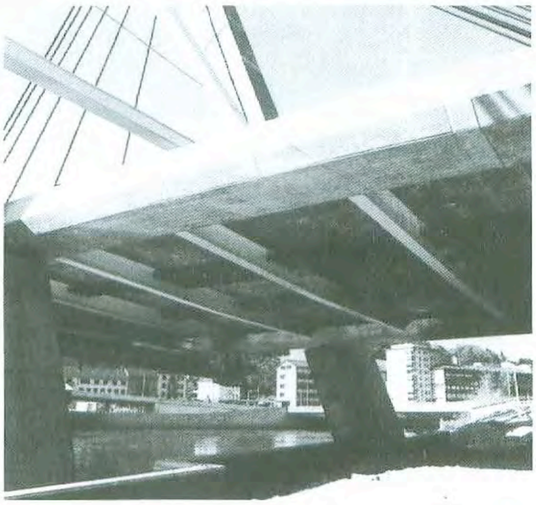


Figure 6.20. Example of a concrete deck with steel crossbeams –Rheinbrücke N4 Cable stayed bridge – Schaffhausen, Switzerland [6.1]

6.7.2 Different types of association between steel and prestressed concrete can prove efficient and economical :

- a composite deck can be installed on a concrete arch as for the Chateaubriand and Morbihan bridges (figure 6.21).
- Some sections of a bridge can be built in steel and others in prestressed concrete. This can be efficient in classical girder bridges as evidenced by different examples : in the Mathilde and Cheviré bridges, a steel orthotropic span is supported by prestressed concrete cantilevers ; in the Tortosa Bridge, the box-girder is made continuous ; it is made of prestressed concrete upon supports, and in steel in the spans (figure 6.22).
- The idea is specially useful in cable-stayed bridges to lighten the main span, by using steel, such as in the Tampico, Ikuchi and Normandie bridges (figure 6.23).

Figure 6.21. The composite deck of the Morbihan Bridge – France, to be noted: the pedestrian walkway following the arch line. [4]

6.7.3 Classical composite bridges received a very large development recently, in several countries, and were found to be extremely competitive. The concrete slab of these bridges must not be underestimated and must be carefully designed to avoid or limit cracks and produce the desired durability.

New ideas have emerged to associate steel and concrete in non classical cross-sections, replacing the concrete walls of typical concrete cross-sections by steel elements such as steel folded webs or steel plane trusses. But these new solutions did not prove to be particularly economic.

6.7.4 As a conclusion, designers must consider at the same time steel and prestressed concrete, and must be able to take advantage of the qualities of both materials.

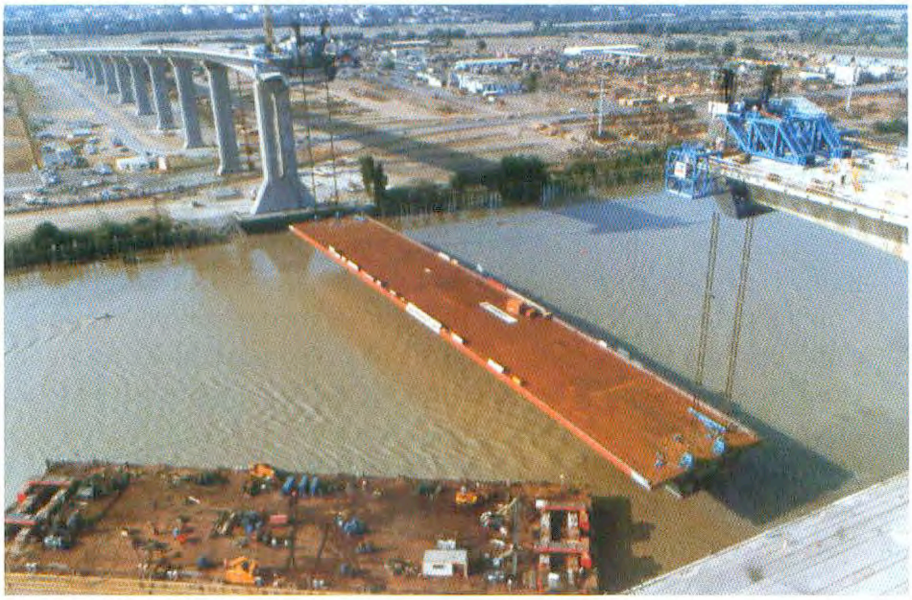


Figure 6.22. Steel girder between two prestressed concrete cantilevers for the Cheviré Bridge in Nantes (during lifting) [9]

Figure 6.23. Light steel deck for the central part of the Normandy bridge [9]

6.8 Progressive collapse

Bridges as all other structures have to be designed to resist permanent and variable loads, cyclic and random actions such as those resulting from temperature and wind, as well as accidental situations resulting from earthquakes, ship collisions, etc.

The design must be developed in such a way to eliminate the risk of a progressive collapse coming from an accident above the specified or predictable scale, a ship collision for example.

If many supports might be impacted by a ship and cannot resist to impact loading larger than the design value – selected after an appropriate risk analysis-, the bridge has to be divided into elements in such a way that the damage would be limited to a part of the structure. The Ré Island bridge in France and the Confederation bridge, in Canada, provide two examples: drop-in spans divide the Confederation bridge into independent structures which can stand alone, and joints close to a mid-span section divide the Ré Island Bridge into several independent viaducts. But in many other conditions, including construction stages, the designer has to consider the risk of progressive collapse which has to be eliminated or limited by an appropriate design.

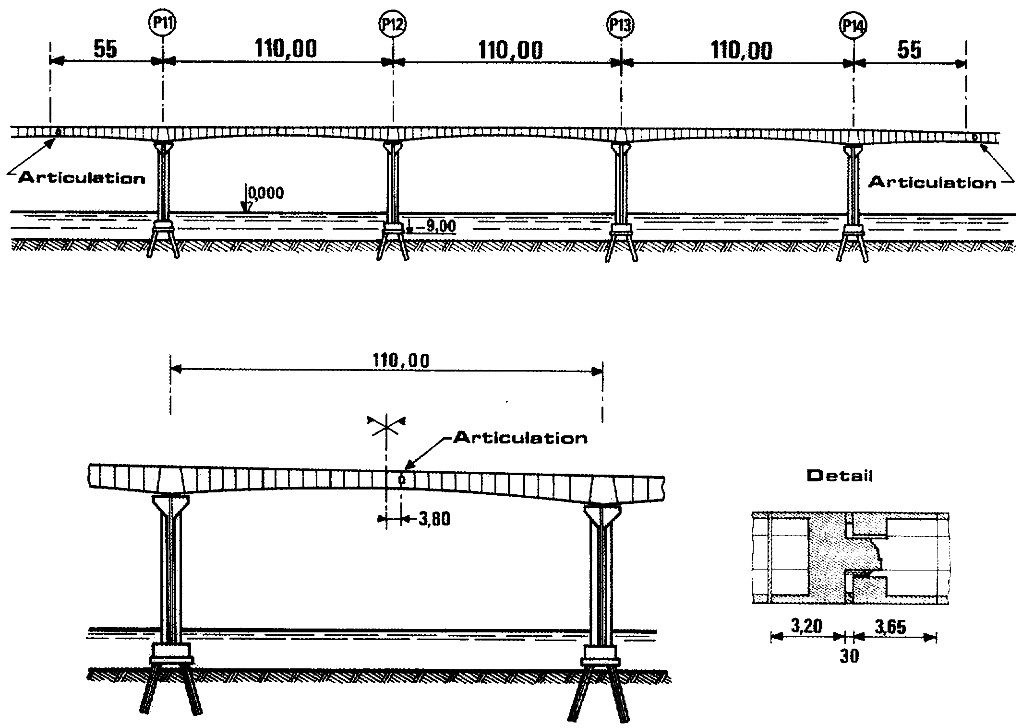


Figure 6.24. Span arrangement of the Ré Island Bridge - France [1]

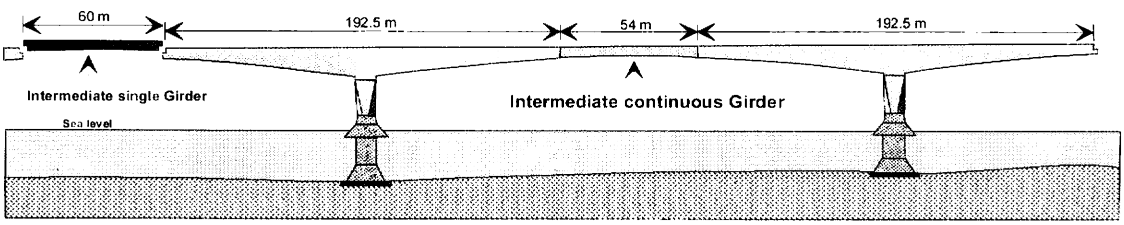


Figure 6.25. Span arrangement of the Confederation Bridge - Canada [20]

6.9 Development of the detailed design

The development of the detailed design is a second step which follows the conceptual design, generally after the decision has been made to accept it. All details must be then analysed, all equipments must be precisely located, all technological requirements must be fulfilled, specially with regard to prestressing and cable systems (in cable-supported bridges).

6.9.1 Technology has an important role in the detail design. A project cannot be seriously developed without selecting all necessary equipment (bearings, joints, barriers,...) and systems (waterproofing, prestressing, cables for cable-supported bridges). It must be clear that the designer has to select, for his design, specific suppliers for each or almost each of these items.

This creates some contradiction with the necessary competition, at a later stage, between the different possible suppliers (and even the contractors). This is not a reason - though some owners believe it is possible to design without selecting a 'reference' supplier for each equipment or system - to develop a design without entering those details which fix many dimensions.

The solution consists in considering these specific equipment and systems - the suppliers of which do not need to be named - as 'reference' equipment and systems which can be changed by the selected contractor on condition that he develops, under his own responsibility, the necessary amendments to adapt the design to the production of other suppliers.

6.9.2 Analyses and computation must be developed by steps ; simplified ones first, to really understand and master the flow of forces, and detailed ones later to control precisely the design.

It happens sometimes that these analyses and computations identify unexpected problems which make it difficult to proceed with the selected shapes or dimensions. The designer must then avoid 'amending' the design by adding elements or distorting shapes: the concept itself must be reconsidered and adapted to the results of the analysis. Either the problems can be eliminated by an evolution of the conceptual design, or the conceptual design can be adapted to these unexpected forces ; but in no case should the designer remain prisoner of his initial drawings and accept unsatisfactory solutions to overcome the difficulties.

This is why simplified analyses and computations must be made very rapidly after the selection of the conceptual design to confirm its validity, and even more its performance. And this is also why the designer must have a clear idea of the flow of forces, generally precise enough to avoid the greater problems.

6.9.3 The detailed design is sometimes developed with the collaboration of an architect. This must be done respecting the philosophy of the conceptual design ; shapes, decoration if any must be in agreement and expressing the flow of forces. This is the condition of a clear expression of the structural concept.

It is better to have no architect associated with the project if he is not devoted to the expression of the flow of forces and of the structural solution.

6.9.4 Finally, the detailed design must be developed with ambition and rigour. The designer must eliminate all unnecessary secondary forces : by an adequate location of bearings, of prestressing anchorages and of cable anchorages if any ; by a perfect geometry, with centre-lines intersecting in such a way than normal forces have no eccentricity ; with a design of prestressing tendons such that curvature radii are large, that tendons are almost straight with limited friction losses ; with such shapes that concreting is easy and can provide a high concrete quality, that reinforcement can be designed properly, with a limited ratio and an easy installation.

6.9.5 Inspection, maintenance and capacity to strengthen the structure if necessary must be seriously considered in the design ; they are keys for the structure's durability and for an easy operation by the owner.

An efficient, economic and easy operation, is - a designer must not forget it - the major goal of the design.

7 Philosophy of prestressing

With the wider development of prestressed concrete structures, and with the evolution of national and international codes produced by the notion of limit states, the real philosophy of prestressing has sometimes been forgotten or weakened. It is worth going back to this basic philosophy to have a better understanding of prestressing, the more as code requirements cannot lead alone to well designed and durable structures.

7.1 Historical background

The structural behaviour of reinforced concrete has only been understood in steps, decades after its pioneer applications by Lambot and Monier. At the turn of the century, Considère, Rabut, Morsch and others proposed the convenient models, either from a truss analogy or for section analyses, balancing in the reinforcement tensile forces produced by bending.

Clearly, the structural behaviour of reinforced concrete produces cracks in zones were tensile forces develop.

As engineers did not feel comfortable with these cracks, evidenced in erected structures before they have been really understood, they tried to avoid them by artificially introducing compressive forces. The idea came very soon of tensioning reinforcement to produce the desired compressive stresses; in fact, some decades passed before the real development of prestressing by Freyssinet, who used high-strength steel wires to produce durable compressive forces in concrete members.

Basically, engineers invented prestressing to produce such compressive forces in concrete members that tensile stresses and cracks could be eliminated or controlled.

7.1.1 Prestressing forces to balance loads

Engineers very soon developed a great skill for designing prestressed structures; Freyssinet - and later Guyon - stated that prestressing forces were to be organized to balance loads, in order to produce the desired distribution of forces.

"Balancing loads", these are the key words to understanding prestressing and to basing this understanding on a fundamental principle, much more important than any code.

This idea is very clear, also, in the drawings of Franz Dischinger's patent: he designed external tendons, in the thirties, to balance loads in a very modern design, fifty years before the modern applications of external prestressing (figure 7.1).

A more theoretical approach can be given by the two methods of introducing prestressing effects in a structure.

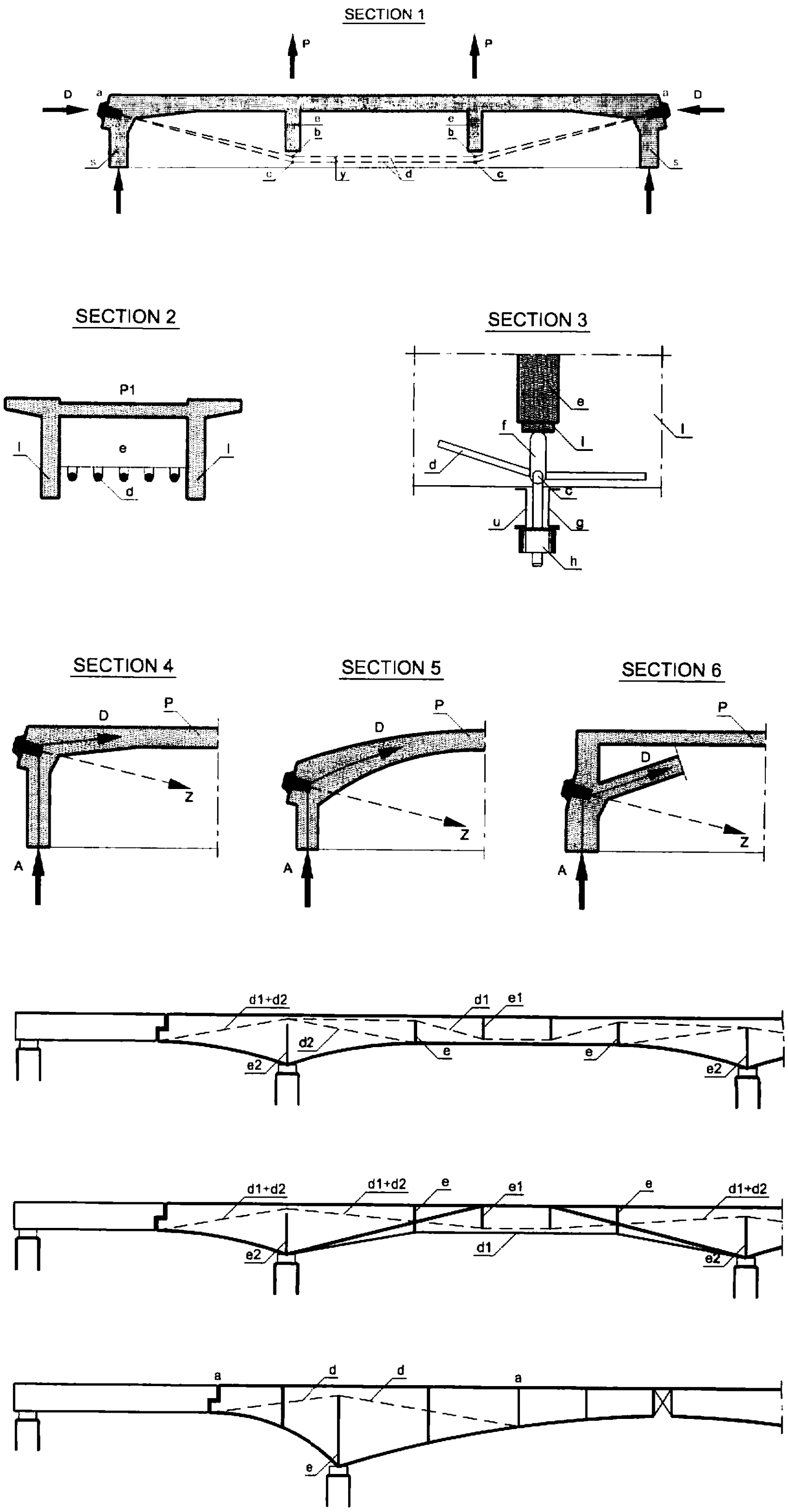


Figure 7.1. Drawings in Dischinger's patent

7.1.2 The internal approach of prestressing effects

The "internal" approach consists in introducing, in each concrete cross-section of a structure,

the prestressing isostatic effects (figure 7.2):

- normal force : $N = F \cos \alpha$

- shear-force : $V = F \sin \alpha$

- and bending moment for a flexion in the plane G x y : $M = e F \cos \alpha$

Of course, these isostatic effects can produce in addition hyperstatic forces in hyperstatic structures.

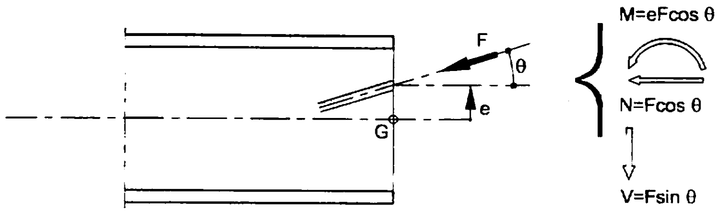


Figure 7.2. "Internal" approach of prestressing forces – Internal prestressing effects

7.1.3 The external approach of prestressing effects

The "external" approach consists of directly introducing the forces produced on concrete by prestressing tendons (figure 3):

- the anchorage forces, F_A and F_B ;

- the distributed normal force produced by the tendon curvature, $\dfrac{F(x)}{R(x)}$

-and the distributed friction force, parallel to the tendon, $\dfrac{dF(x)}{dx}$

The external approach clearly shows the balance of loads by prestressing forces: the distributed normal force $F(x) / R(x)$, directly balances a part of external loads.

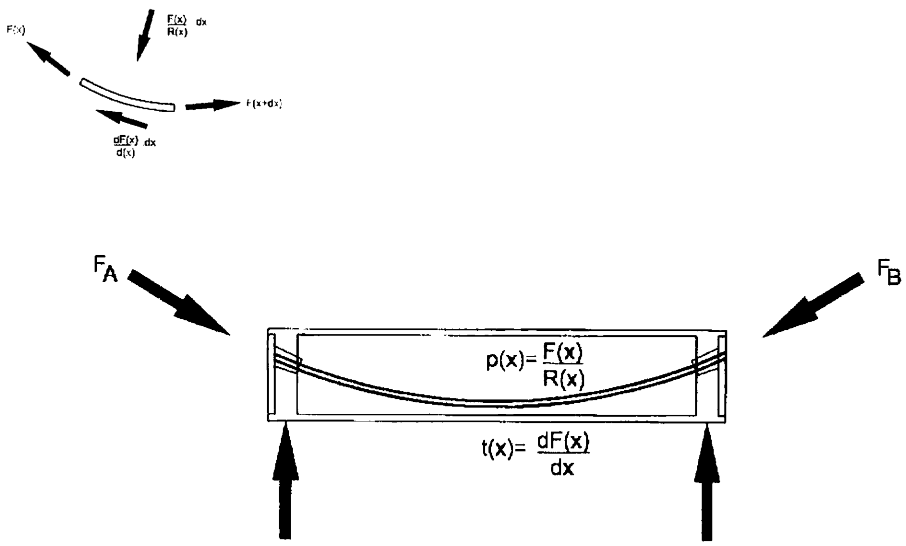


Figure 7.3. "External" approach of prestressing forces: External forces on concrete

7.2 Code requirements

Modern codes are based on limit states to produce structural safety and durability.

7.2.1 Ultimate Limit States

Structural safety is guaranteed if Ultimate Limit States are not reached under factored loads.

In the corresponding analyses, no philosophical difference is made between reinforcement and bonded prestressing tendons. Of course, prestressing effects are logically considered, with partial safety factors in some codes; but under factored loads, bonded prestressing tendons receive tension variations as reinforcement, and safety can be achieved either by greater prestressing forces or by a higher reinforcement ratio.

The excessive consideration which is given by some engineers to Ultimate Limit States, disregarding the other aspects of design, can lead to an underestimation of the interest of prestressing, specially in bridges with medium or long spans where reinforcement cannot be predominant.

7.2.2 Service Limit States

Service Limit States govern durability - mainly by limiting crack width - and fatigue resistance, even if rupture of tendons by fatigue could be considered as an ultimate limit state: frequent loads - which govern fatigue - correspond to the probable situation of the structure, and not to the highly improbable situations which are modelled by factored loads.

Service Limit States are also supposed to govern deflections. But existing codes cannot really guarantee a limitation of creep-induced deflections of bridges. Only a good balance of loads can control long-term deformations and deflections; this need for a good balance of permanent loads will be analysed in more details at paragraph 7.3

7.2.3 Control of construction geometry

Finally, even if some codes do not cover this major aspect, design and construction must allow for a perfect control of the structure geometry. This is specially important for bridges built by steps as for example the free cantilevering method.

An evaluation of the structure deflections, at each of the construction steps, is to be performed, based on the probable value of loads. Pre-cambers are then estimated from these predicted deflections when necessary. The corresponding structural analysis follows the construction history step by step; at the last design stage it must consider precisely time dependent effects such as development of friction losses, concrete creep and shrinkage, as well as steel relaxation in prestressing tendons.

Later, precise geometric surveys at the different construction steps allow for a control of predicted deflections. If they evidence some unexpected variations, loads and prestressing tendon tensions (so as cable tensions in cable supported bridges) might be re-evaluated and countermeasures taken to adjust the construction geometry. Some measurements of the tension in prestressing tendons, organized to control friction losses, would help the analysis and the evaluation of the actual bridge conditions; of course measurements of cable tensions in cable supported bridges is of even greater importance for control of geometry.

7.2.4 Construction situations

Due to the practical importance of the step by step analysis of complex structures, the specifications as regards ultimate limit states are often limited to the final situation of the bridge under operation. It would be extremely demanding to formally analyse structural safety at each construction step and this cannot practically be required, at least in the present conditions; but the designer must be conscious of the need and can in many cases confirm safety with simple practical checks that there is no risk of collapse.

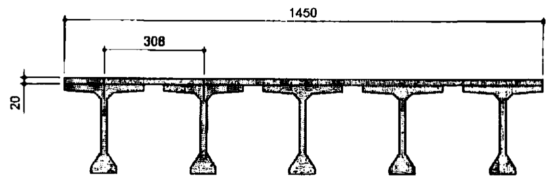
7.3 Balance of permanent loads. Control of deformation

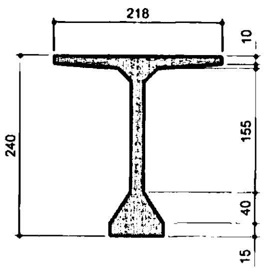
To reduce creep-induced deformations and deflections, which cannot be predicted with a great accuracy, it is necessary to balance by prestressing bending moments a major part of permanent loads.

We shall show with the help of some examples that unbalanced permanent loads, or on the contrary excessive prestressing effects, produce unfavourable deformations and deflections.

7.3.1 Precast-beams

Precast beams are used to build very light structures; due to their limited flexural inertia, live loads produce important stress variations. To satisfy with code requirements - especially when aiming at a "full prestressing" - it is necessary to introduce, by prestressing forces, very large compressive stresses in the lower flange. The distribution of stresses under permanent loads is thus extremely unbalanced, with limited stresses in the upper flange and large stresses in the lower one (figure 7.4).





σj'= 30 MPa when tensionnig
σ28= 35 MPa

	TENDONS	
	Full prestressing	Partial prestressing
First group of tendons	4x9T15	4x8T15
Second group of tendons	5x6T15	5x5xT15
Total	66 Strands	57 Strands

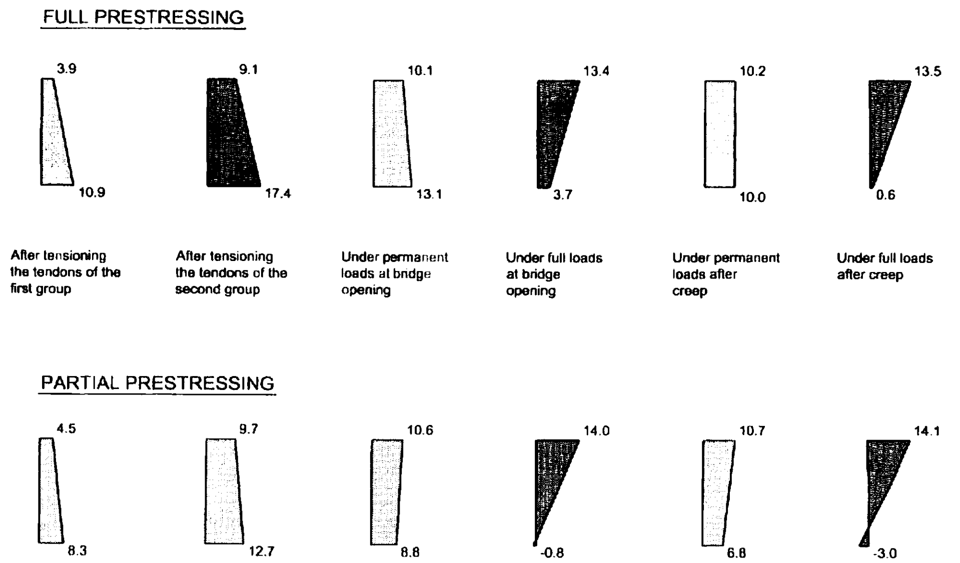


Figure 7.4.1

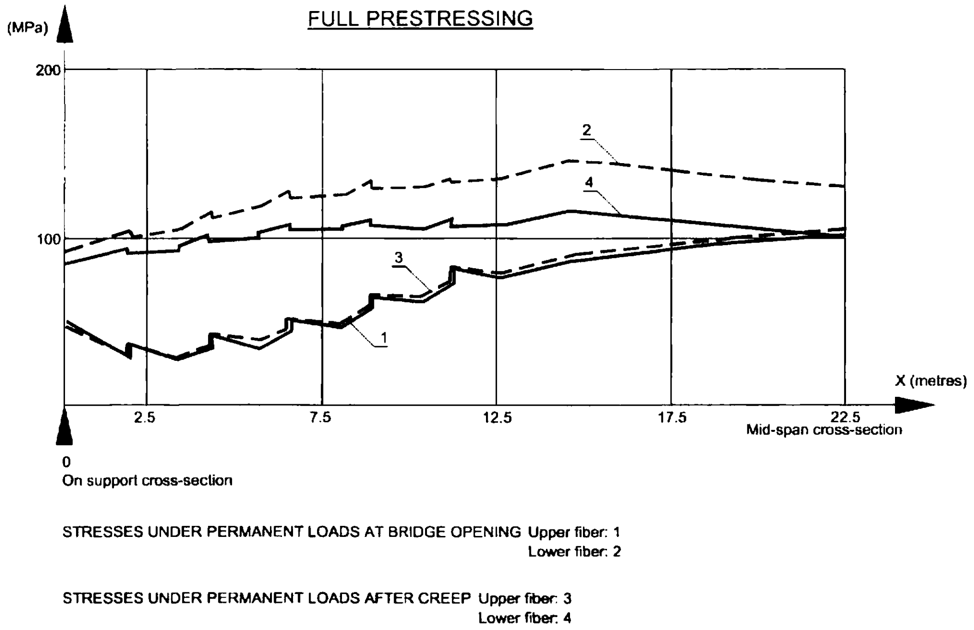


Figure 7.4.2

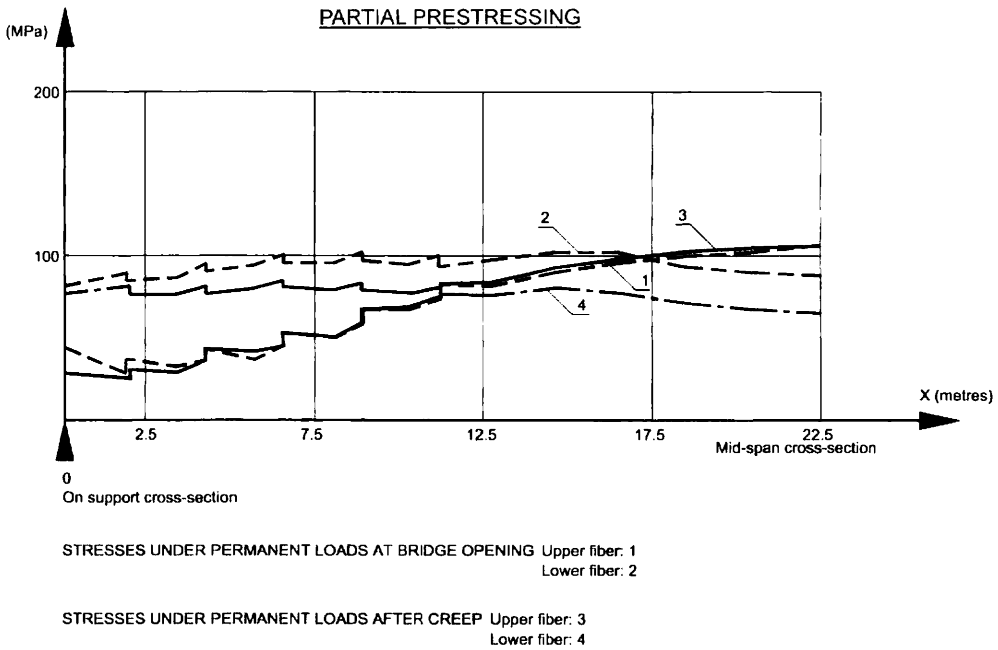


Figure 7.4.3

Figure 7.4: Example of a bridge made of precast beams prestressed by post-tension. Comparison of stress diagrams at midspan with full and partial prestressing, at the main construction steps. Distribution of stresses, longitudinally, in both cases

In such a situation, creep will camber the precast beams and the whole bridge upwards. As the intensity of creep depends on many uncontrolled factors, the upwards deflection can be much smaller, or much greater, than the expected one.

A good design must evidently limit these uncertainties and, for this reason, produce a better distribution of stresses in the cross-sections through a better balance of permanent loads. Increased flexural inertia and partial prestressing are evidently favourable.

7.3.2 Incrementally launched bridges

When external prestressing has been developed for incrementally launched bridges, in the eighties, some contractors proposed the use of straight and centred external tendons installed before launching and left in this situation for the bridge in operation.

Evidently, straight and centred tendons only produce normal force and balance no load at all. They would have resulted in important shear-stresses in the webs, and in unbalanced distributions of normal stresses in cross-sections, with important compressive stresses in the lower slab on supports and in the upper slab at mid-span. Creep would have produced important downwards deflections, with all the corresponding uncertainties.

Deviated tendons have been preferred (figure 7.5): they reduce shear-stresses in the webs ; and they balance a large part of permanent loads.

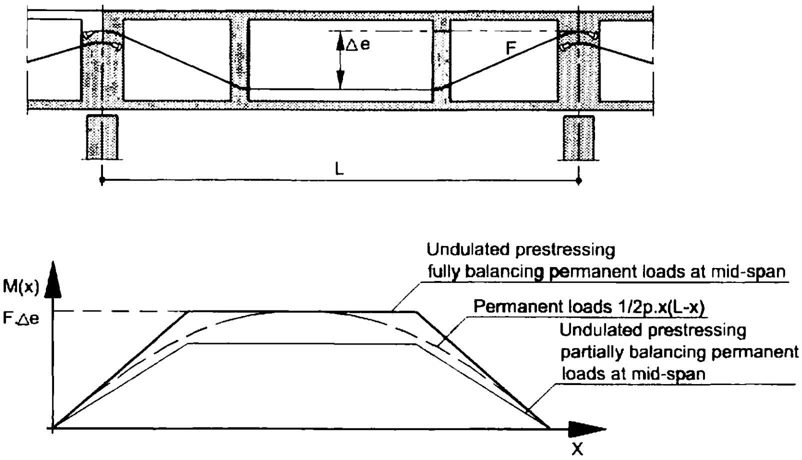


Figure 7.5.: Classical orgnisation of external tendons in a bridge incrementally built

7.3.3 Bridges built by the cantilever method

But the most classical example is for bridges built by the cantilever method.

In the first applications by Finsterwalder, or in France by Courbon, the cantilevers were joined at mid-span by simple articulations. Of course, only a part of the negative bending moments produced by permanent loads was balanced by prestressing bending moments, taking advantage of the normal force produced by prestressing. This resulted in large deflections produced by concrete creep. These deflections, much larger than expected, were explained by creep effects - not precisely evaluated at the time - but also by relaxation in prestressing steel and by friction losses at a time when the technology was not as well advanced as it is now.

Articulations at midspan were abandoned long ego, and bridges made continuous. But creep induced by partly unbalanced permanents loads remained, and produced unexpected or under-evaluated hyperstatic effects; some bridges built in the sixties and early seventies had to be strengthened with additional prestressing. And in some cases, for long spans with a box-girder of limited depth at midspan, much larger deflections appeared than were acceptable (Parrot's Ferry Bridge, Grand Mere Bridge).

With the development of scientific knowledge on concrete creep and of computations in the late seventies, the evaluation of creep-induced deflections could have been considered reliable, as the evaluation of hyperstatic effects of concrete creep. Unfortunately, the recent erection of the Cheviré Bridge evidenced large deflections in the articulated spans (one in the access spans on each bank, and two in the main span, one on each side of the suspended orthotropic box-girder) which can be explained by creep effects only. But in these spans only a small part of permanent loads is balanced by prestressing moments due to the bridge configuration: compressive stresses are very high in the lower slab and very small in the upper one

It must be concluded that, despite our better knowledge of concrete creep and our large computational capability, the prediction of creep effects - especially deflections - is not as reliable as necessary when compressive stresses are distributed with a very inclined diagram, due to a very partial balance of permanent loads.

Renaud Favre recently developed the same idea in several publications based on accurate analyses of some bridges [R. Favre a. al, 1995 - 7.1].

7.3.4 Cable-stayed bridges

Cable-stayed bridges give the best example of a perfect balance of permanent loads: in good designs, cable tensions are selected to balance permanent loads exactly, and prestressing tendons are centred in the zones were they are needed to maintain this perfect balance (figure 7.6). Of course, creep produces deformations, but almost uniform in all cross-sections; creep effects then remain limited and can be easily compensated after some years by a limited adjustment of cables.

Some problems can appear when these simple principles are forgotten. The deck of a small cable-stayed bridge in France received too deep and thin longitudinal ribs ; to avoid high tensile stresses under extreme live loads, the designer decided to produce compressive stresses in the bottom flange by off-centre tendons and increased cable tensions. Creep was well controlled during construction, but when the erection programme had to be altered, the distribution of stresses immediately produced unexpected creep deflections which could not be later compensated.

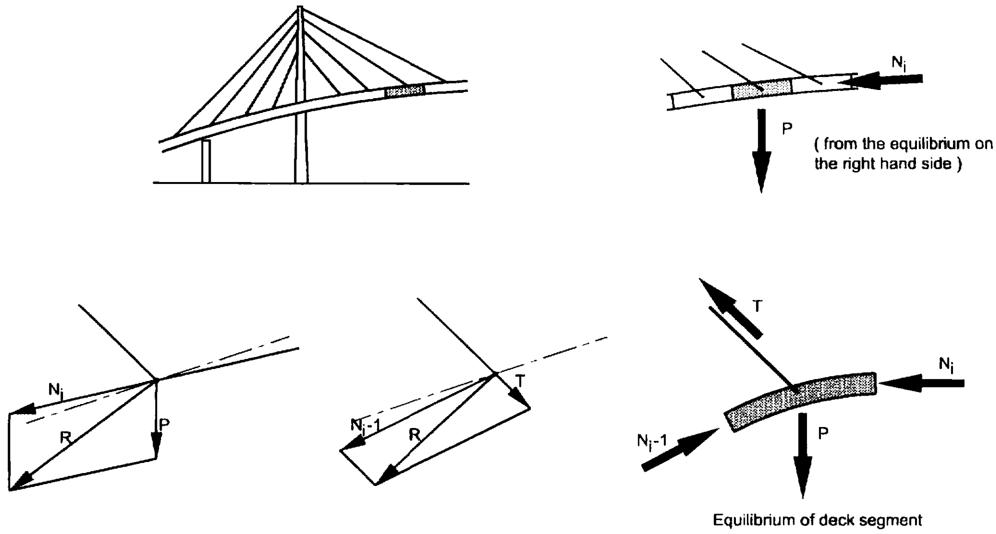


Figure 7.6. Perfect balance of permanent loads in flexible cable stayed bridges

7.3.5 Conclusion

As a conclusion, it is very clear that permanent loads have to be well balanced by prestressing moments (or by cable tensions in cable-stayed bridges) to produce good designs: to limit shear-stresses (and torsional stresses in curved and skew bridges); to produce diagrams of longitudinal stresses as uniform as possible under permanent loads; and to control creep deformations and effects, especially deflections.

Some engineers consider that all or almost all permanent loads must be balanced by prestressing moments, 80 to 90 % for Renaud Favre as an example; but, as this is generally uneconomical, it can be considered convenient - as demonstrated from experience - to balance by prestressing moments 60 or 70% of permanent loads in classical bridges with medium spans. Of course the proportion of permanent loads to be balanced increases for very long spans - such as long span bridges built by the cantilever method - and for more flexible structures ; it must be close to 100% for very flexible concrete elements such as cable-stayed slabs.

Codes cannot specify such recommendations which must be considered as major elements of bridge designing art to produce safe and durable structures. But owners or engineers can specify in contracts that at least a given part of permanent loads has to be balanced in that way.

7.4 Bonded and unbonded tendons

7.4.1 With the recent return to external prestressing, some words must be given to clarify the different structural behaviours of bonded and unbonded tendons.

In bonded tendons, prestressing steel is connected by grout to the concrete at the same level and supports the same strain variations under live loads. If stress variations are noted positive in tendons when corresponding to tension, we have (figure 7.7) :

$$\Delta\varepsilon_s = -(\Delta\delta u + \Delta\delta\omega\, y_s)$$

where $\Delta\delta u$ and $\Delta\delta\omega$ govern the strain distribution in the concrete cross-section and y_s is the ordinate of tendon. The relation is valid up to failure if cracks are distributed, and has to be abandoned only in open joints between precast is below reasonable limits.

If the tendon is unbonded, without friction between duct and tendon, only its segments or in wide cracks of cast in situ segments when the reinforcement ratio length variation between the two anchorages is given by the corresponding

$$\Delta \cdot l_s = -\int_{s_1}^{s_2} (\Delta\delta u + \Delta\delta w y_s) ds$$

elongation of concrete:

For an external tendon, the length variation is given between two successive deviators by their displacements if the tendon cannot slip on them (figure 7.8) :

$$\Delta l_s = \sqrt{(l + u_{i+1} - u_i - e_{i+1}\omega_{i+1} + e_i\omega_i)^2 - (v_{i+1} - v_i + e_{i+1} - e_i)^2} - \sqrt{l^2 + (e_{i+1} - e_i)^2}$$

$$\approx u_{i+1} - u_i - e_{i+1}\,\omega_{i+1} + e_i\omega_i$$

Where e is the tendon eccentricity, u and v the longitudinal and transverse displacements, and ω the rotation at the deviation corresponding to the index..

Of course, friction between tendon and duct can alter the relation for unbonded tendons, and slip on deviators can change the distribution of tension in external tendons.

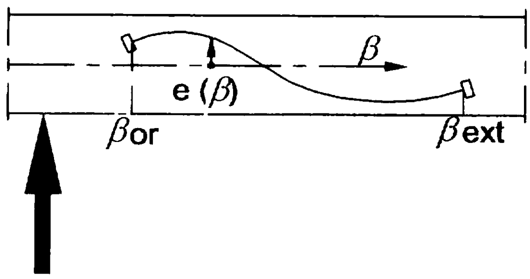
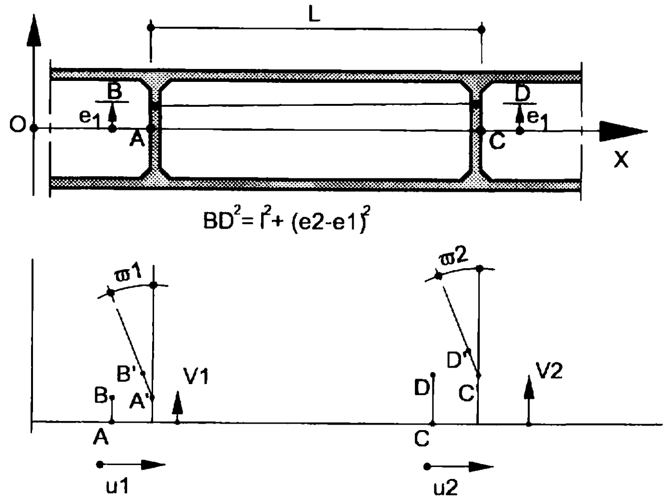


Figure 7.7. An unbounded tendon in a beam



$$\Delta x = l + u_2 - u_1 - e_2 \sin\varpi_2 + e_1 \sin\varpi_1$$
$$\Delta y = v_2 - v_1 + e_2 \cos\varpi_2 - e_1 \cos\varpi_1$$

Figure 7.8 An external tendon between two deviators

But, to conclude, bridges built with internal tendons can be analysed with classical methods: the analysis can be conducted in each cross-section, because tension in internal tendons is directly connected locally to concrete deformations. On the contrary, tension in unbonded or external tendons is given by the structure global deformation, and not section by section. The analysis in ultimate conditions of bridges prestressed with unbonded or external tendons must be done with adapted methods which cannot be detailed here, except if tension variations are neglected in unbonded or external tendons as a conservative simplification. For Service Limit States, it is logical to consider that there is no tension variation in unbonded and external tendons, since the small deflections reduce the tension variations to very little.

fib intends to produce a document devoted specifically to unbonded and external tendons. In view of this, the present paragraph is intended only to indicate that unbonded and external tendons cannot be analysed as classical grouted tendons in ultimate conditions.

To be absolutely clear, both internal prestressing and external prestressing can constitute an excellent solution provided that the design is developed in accordance with the specific behaviour of tendons and that the quality of construction produces the desired durability.

7.4.2 Though it might be considered a detail, it must be underlined that it is dangerous to couple all external tendons in bridges built span by span: the destruction of a "rear" span, for any reason, would produce a progressive collapse of all "front" spans. Though such a tendon arrangement did not produce the progressive collapse of a viaduct in the Bubiyan bridge when one of its spans was destroyed by bombs during the Gulf war (the cement grout blocked the "rear" anchorages on one support), this phenomenon developed in another viaduct of the Bubiyan bridge and in the Injaka bridge, in South Africa, after the connexion between the launching nose and the box girder failed during launching.

7.4.3 It must be stressed that prestressing tendons are often not grouted during construction stages, so that the structural safety of many bridges must be specially analysed for these situations, considering the tendons unbounded. Practically, no stress variation can develop in these tendons, as in external tendons, when accidental loads or actions occur. If this situation is not clearly understood by the designer, it might lead to failure as in a cantilever bridge broken during erection. This example made of precast segments evidently without reinforcement in joints showed its cantilever broken at mid-span when loads were not properly balanced by prestressing forces in cantilever tendons and a joint opened up to the failure.

7.5 Ductility

Finally, it must be clearly stated that prestressed concrete bridges must be ductile, that prestressing cannot produce ductility alone and that adapted reinforcement must be provided to avoid global and local brittleness

It must be clear, also, that ductility can be given to bridges built with precast segments. When joints open in ultimate conditions, compressive forces are concentrated in one of the two slabs, and specially in the corresponding nodes (figure 7.9). It can be shown that it is extremely dangerous to have wide joint openings close to supports where shear-forces are large, but joints can generally open at midspan under factored loads. In all cases, reinforcement must be introduced in the compressive zone on both sides of open joints, mainly in the nodes, to avoid a local split of the compressive member and the corresponding failure (figures 7.10 and 7.11).

A special chapter in a future document will be devoted to the design of precast segments and to the organisation of shear keys to produce the necessary safety and ductility.

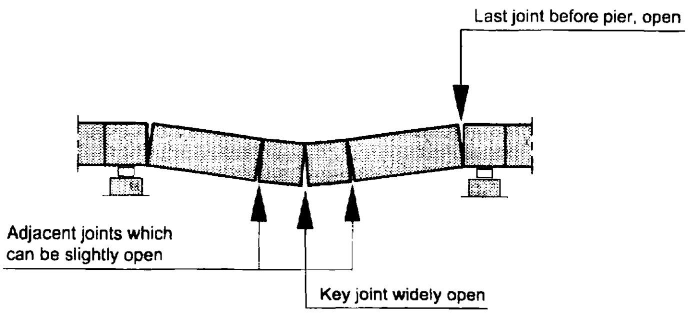


Figure 7.9. Distribution of open joints in a loaded span of a continuous bridge at ultimate limit state

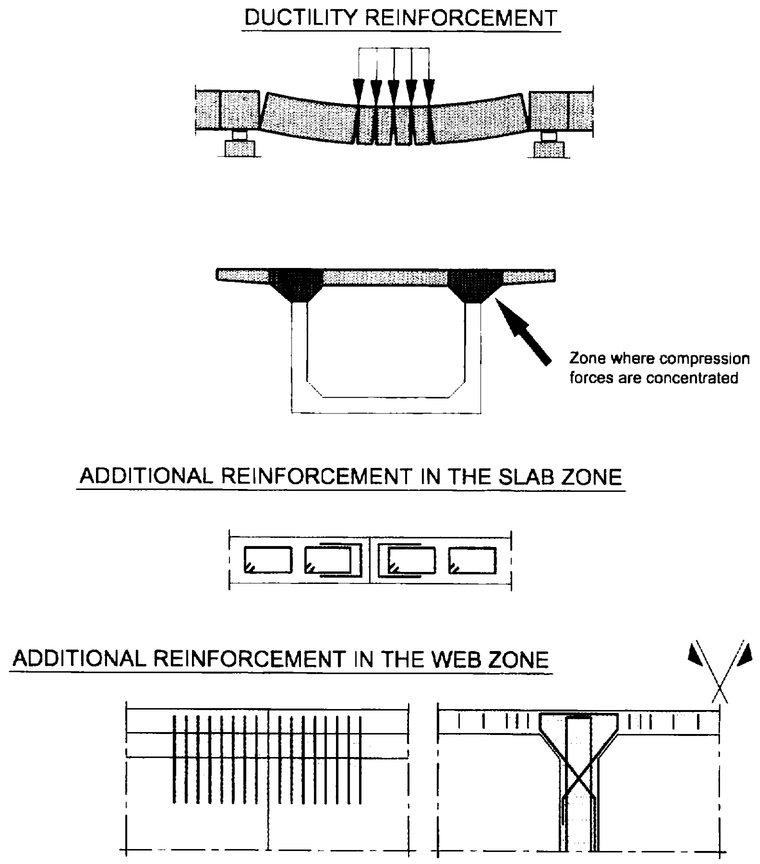


Figure 7.10.Reinforcement in joints at midspan to produce local ductility

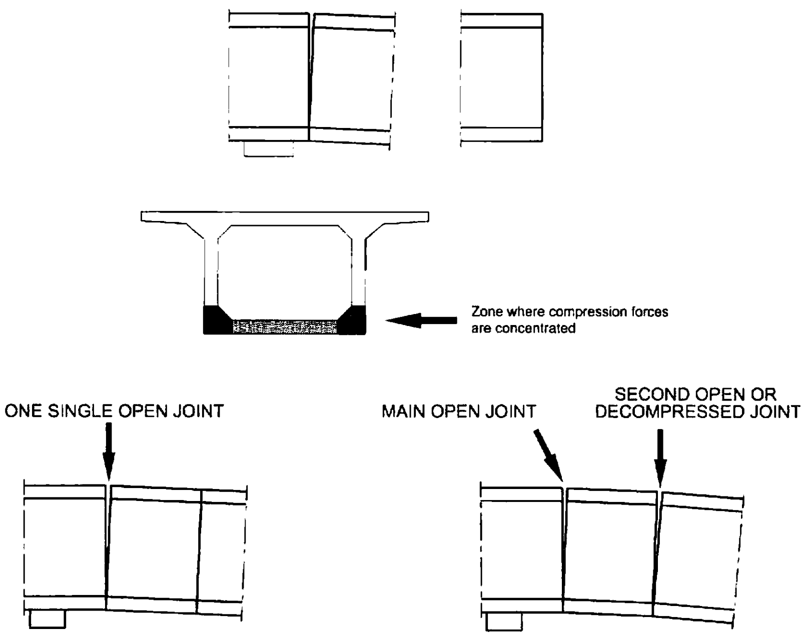


Figure 7.11: Reinforcement in joints adjacent to supports to produce local ductility

7.6 Durability

Durability of concrete structures is mainly governed by the quality of materials and, above all, by the quality of concrete. The more compact concrete is, the more durable ; the recent development of high performance concrete is a major step forward as regards durability, especially when the concrete strength reaches, or is above, 80 MPa. Looking back to the durable constructions of some good engineers, such as Freyssinet, it can be noted that they were made with a concrete of exceptionally high strength for their time.

The other important factor of durability is the cover of reinforcement which must be large enough to prevent the corrosion of bars and subsequent destruction of the concrete cover. Of course, this must be in conjunction with a large compacity of concrete which makes the concrete cover more efficient for the protection of reinforcement.

But for the durability of prestressed concrete, the major problem is the corrosion protection of prestressing tendons. The first major point is the selection of prestressing steel ; engineers must reject steel wires, strands and bars which are highly susceptible to corrosion. A lower susceptibility to corrosion must be preferred to higher strength or lower cost. The second point is the efficiency of the system developed for the corrosion protection, ducts and cement grout for the larger number of tendons : the cement grout (or the product which replaces it) must be conveniently designed, in its chemical content, to eliminate all corrosive agents and agents which could favour corrosion, and for a good injectability to avoid voids, segregation and bleeding ; finally the corrosion protection system must be installed according to the state of the art by qualified agents, and seriously controlled.

The quality of prestressed concrete structures relies directly on the quality of prestressing steel and cement grout and on the professional competence of those who install them with an adequate control. No design specification can obviate a lack of quality in these aspects, or compensate for it.

The influence of design on durability is not so great, but it cannot be ignored. The interest of prestressing, in this field, is often underestimated. Durability cannot but be increased by introducing prestressing forces - longitudinally and transversely - to eliminate tensile stresses and cracks under permanent and frequent loads, especially in areas where de-icing salt is intensely used in Winter or in any other aggressive environment.

7.7 Reinforcement ratio

With the evolution of codes, which try more and more to cover all details and become so complex that their bases are not fully understood, engineers often lose control of their designs. Uncontrolled use of computations by the finite element method and unlogical superimposition of structural effects and of constructional code requirements often result in an inflation of reinforcement. It can produce some construction problems when the reinforcement ratio reaches too high a level in some areas, but this is also a dramatic drawback in the competition with steel and composite constructions.

A clear understanding of the mechanical behaviour of the structure and of the flow of forces is a necessary condition for a good design, which can be helped by simple or more sophisticated models built from the struts and ties theory, for example.

Engineers must master and control their designs and clearly understand the purpose for which the prestressing tendons and reinforcement bars are placed in the concrete structure, and which forces they control.

7.8 Conclusion

To conclude, the designer must clearly organize the flow of forces in the structure and use prestressing to balance loads, especially permanent loads, to achieve an efficient and durable construction.

Pioneers of prestressing clearly mastered their designs and were able to control forces as they wanted. The development of construction sciences with their resulting complexity, the greater demand from the public and authorities, the pressure for lower cost, the uncontrolled facilities given by computers and the complication of codes make it much more difficult now. But this is the challenge for modern designers to develop simple and efficient designs, to organise the flow of forces, and to control deformations, cracks and durability in a much more complex situation.

8 Conclusion

8.1 Structural variety

There is such a large variety of structural shapes that engineers can easily design efficient and elegant bridges, each adapted to its site and to the local conditions, and give them personality through shapes and colours without artificial element or decoration. The search for diversity in bridge design, to avoid boring repetition, is a major element for the constitution, in each country, of a series of beautiful modern bridges. Some examples of this wide structural variety can be found hereafter. No comment is needed, images express themselves the perfect harmony reached between the structure and its environment. Showing the results and the art provide by bridge designer's work is also the best conclusion which can be brought to this Part I of the Guidance for good bridge design.

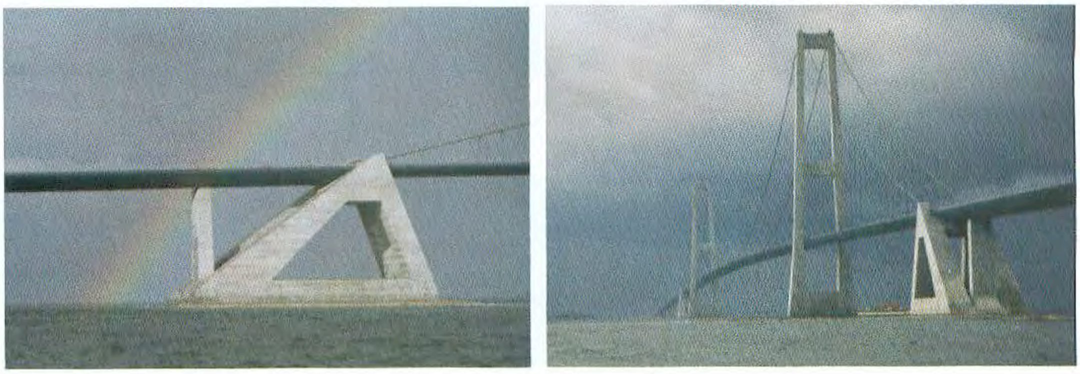


Figure 8.1. Great-Belt suspension bridge – Denmark [3]

Figure 8.2. Kelheim Pedestrian bridge – Germany[11]

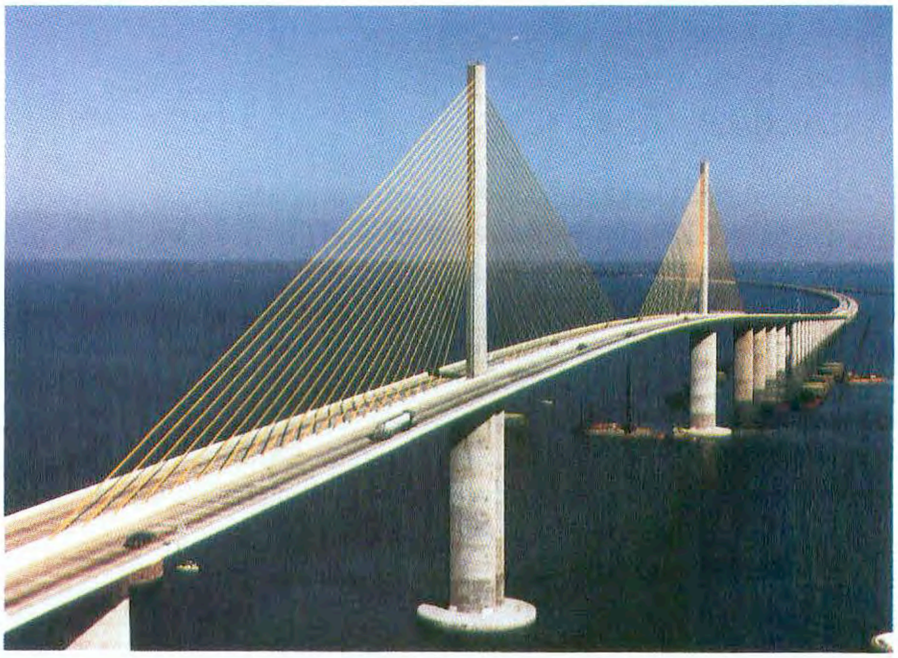


Figure 8.3. Sunshine skyway bridge – Florida – USA [8]

Figure 8.4. Bridge over the Lerez river at Pondevedra – Spain [10]

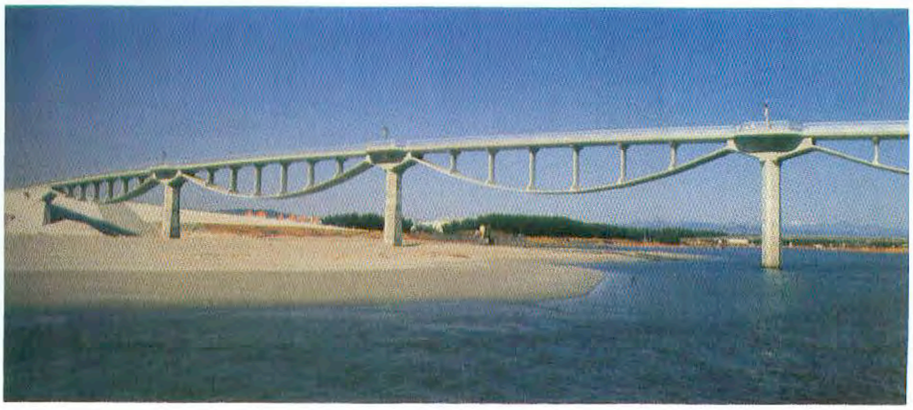


Figure 8.5. Shiosai Bridge – Japan[13]

Figure 8.6. Sunniberg Bridge – Switzerland [14]

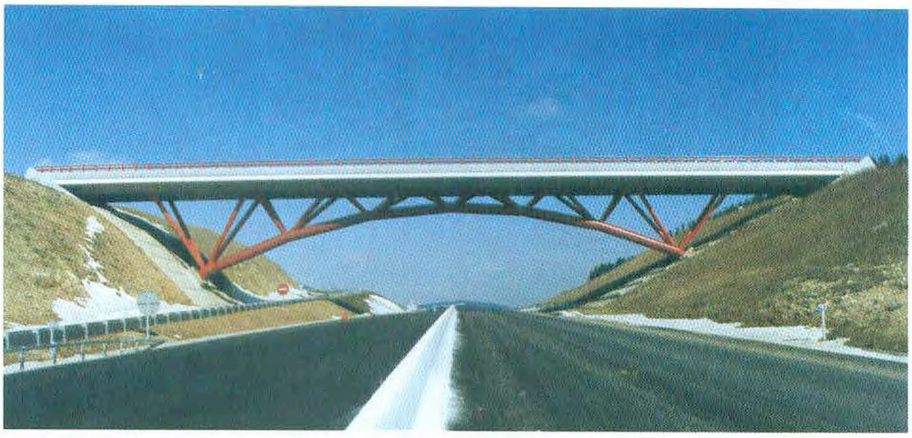


Figure 8.7. Antrenas Bridge –France [4]

Figure 8.8. Pforzheim Bridge – Germany [11]

Figure 8.9. Ganter Bridge – Switzerland [14]

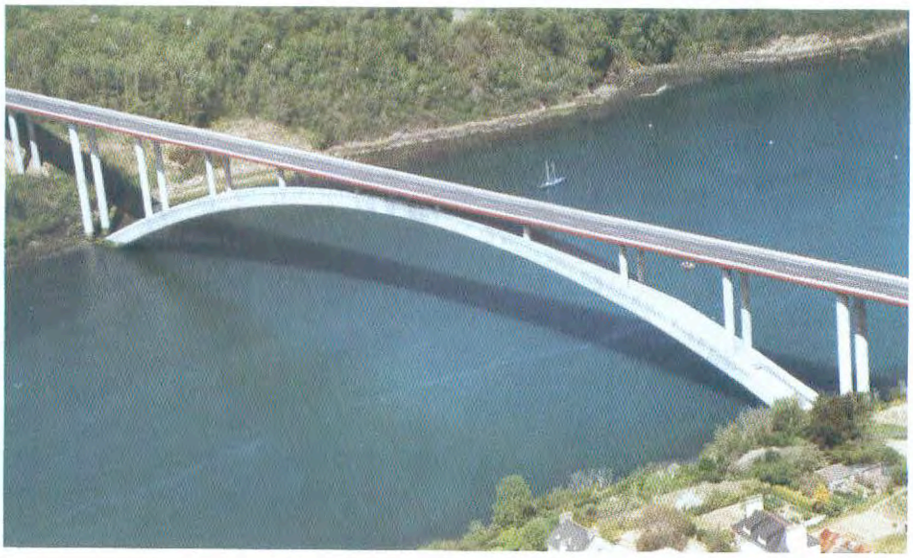


Figure 8.10. Chateaubrillant Bridge – France [1]

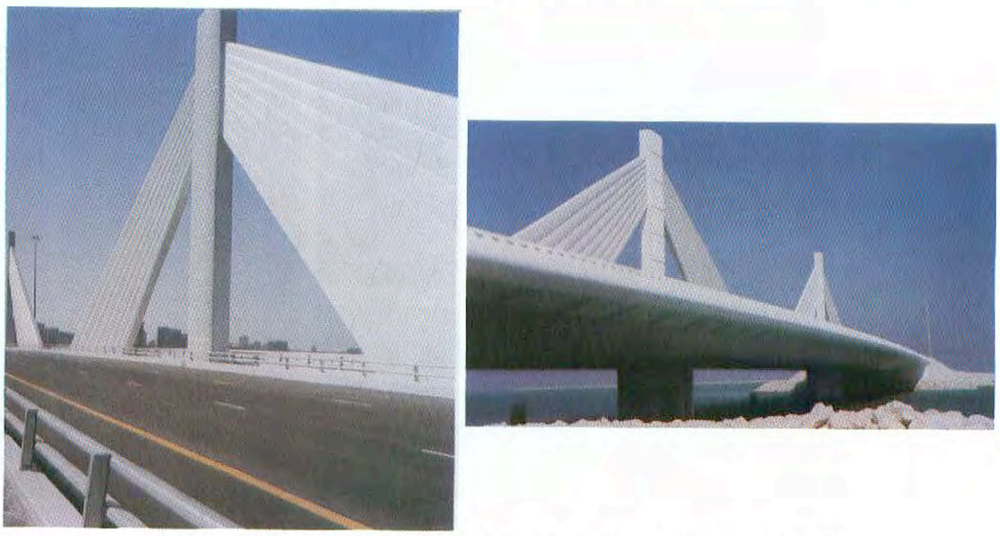


Figure 8.11. Shaikh Isa Salman Bridge – State of Bahrain [14]

Figure 8.12. Avignon Bridge – France [4]

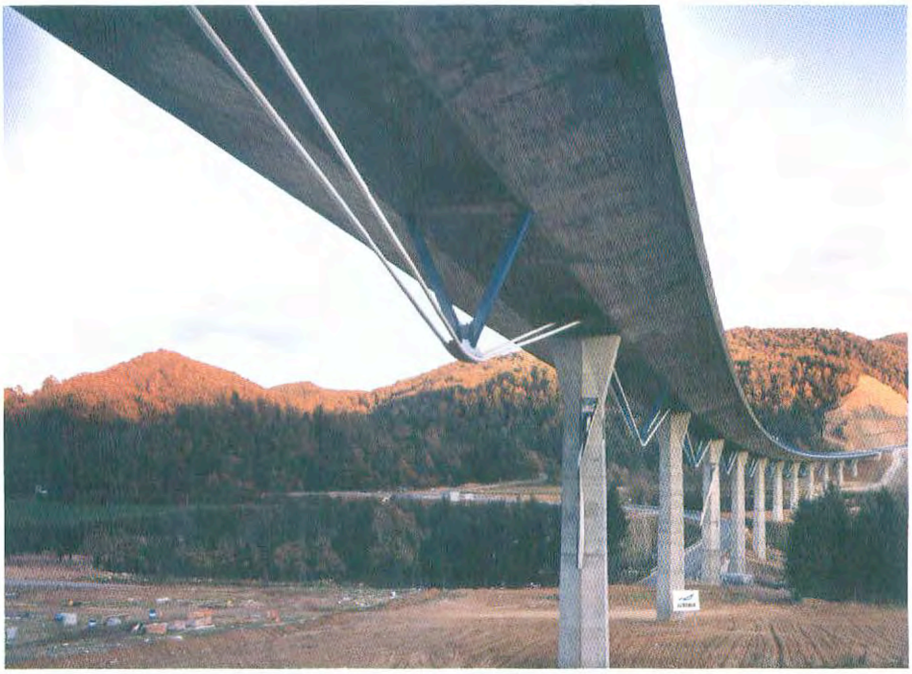


Figure 8.13. Osomort Bridge – Spain [16]

@Seismicisolation

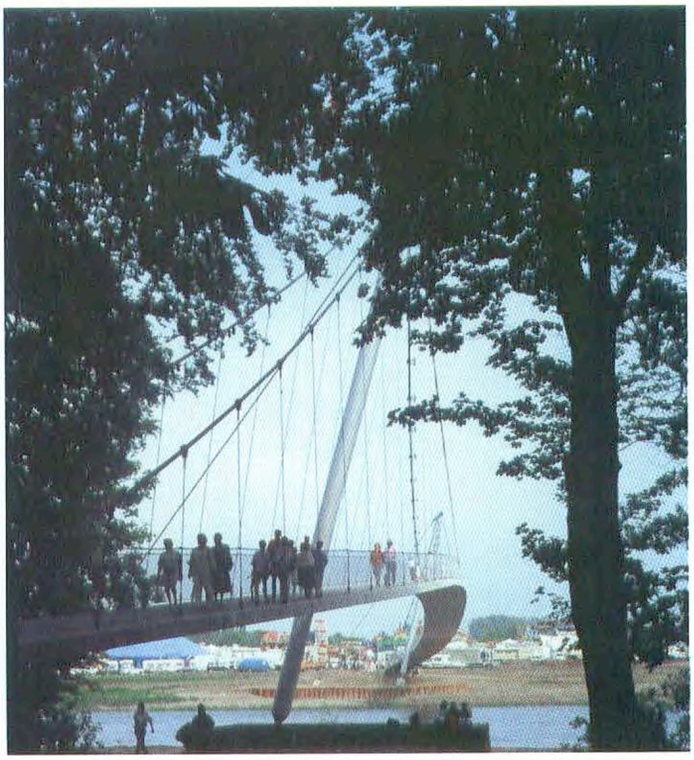


Figure 8.14. Minden pedestrian Bridge – Germany [11]

Figure 8.15.La Arena Viaduc– Spain [23]

Figure 8.16. St Maurice bridge over Rhône river– Switzerland [3]

Figure 8.17. Perfect shapes and proportions of the steel bridge "La Barquetta" over the Guadalquivir river in Sevilla –Spain

PART II

1 Introduction

This second part of the Guidance for good bridge design is devoted to the development of the conceptual design of prestressed concrete bridges including construction aspects which are of major importance, but which must not overcome the main goals of design, functionality and elegance.

The development of a bridge design is mainly governed by the selected span range and by the erection method adapted to this span range, or preferred among those adapted.

1.1 Span range

For some bridges, the span range directly results from the site conditions and from the constraints on the implementation of supports. More frequently, the span range is selected after a comparison between different solutions as shown in Part I chapter 5 and 6 ; this second part of the guidance for a good bridge design aims at guiding the development of the design of any given solution with its corresponding span range.

This span range will influence the selection of the cross-section as already shown (Part 1 chapter 6) ; for cast in situ bridges, for example, the designer will generally select the simpler of shapes adapted to the span range which will normally result in the most efficient solution economically.

The span range will also influence the erection method since each erection technique is adapted, or mainly efficient in a given span range.

To give only some examples :

- precast and pre-tensioned I-girders are adapted to spans between 10 and 30 metres ; in some countries their use is extended for longer spans, even up to 40 metres in the United States when using High Performance Concrete ;
- precast and post-tensioned I-girders are adapted to spans between 25 and 45 metres, and sometimes up to 50 metres ;
- bridges built by the incremental launching method can usually reach 40 metres with a ribbed cross-section and 50 to 60 metres with a box-girder cross-section, depending on the detail of the launching procedure and equipment. These limits are of course widely overpassed when temporary intermediate supports can be installed ;
- bridges built by the balanced cantilever method can be built for spans from about 50 metres (and even less in specific conditions) to 200 metres and even

up to 300 ; but the cost increases rather rapidly with the span (see table in chapter 6.6.2 part I).

1.2 Erection method

Many erection methods, or techniques, directly influence the design :

- the distribution of bending moments produced by self-weights during construction, and also in the final bridge configuration, results from the construction steps and thus from the erection method. The most evident example is given by the balanced cantilever method : self-weights are introduced in the cantilevers with no moment at mid-span ; the final distribution of permanent forces results from the sequence of the construction steps and of the installation by steps of prestressing tendons ; in the same time it is altered by creep deformations directly related to the construction history.

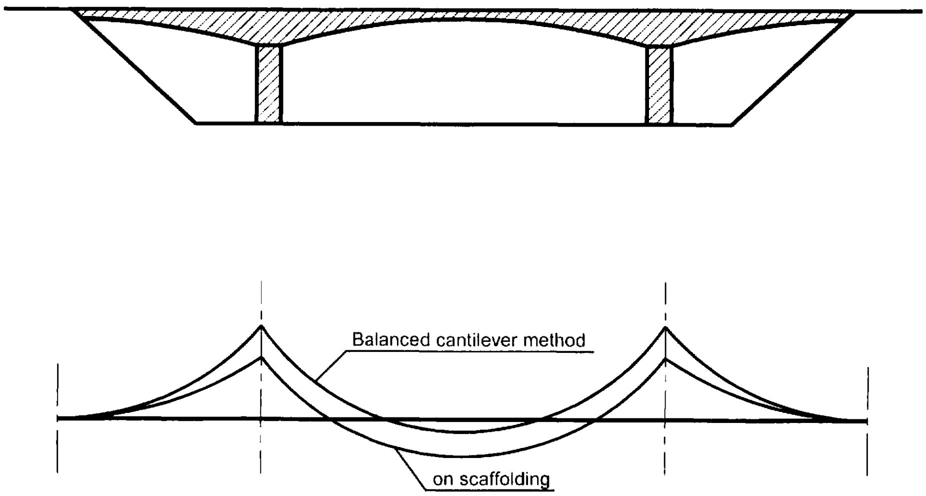


Figure 1.1: Bending moments under self weight – Influence of the construction method

Some methods strongly influence the bridge geometry itself. For a first example, in bridges made from precast and post-tensioned girders, all bays must have the same span ; the curvature of the bridge alignment must be limited ; and the shapes of the cross-section are dictated. For another example, bridges built by the incremental launching method must have a specific geometry. In the simplest situations, the road alignment is straight and the longitudinal profile has a constant radius ; or the profile is straight and the road alignment is with a constant radius, provided that the lower slab of the box-girder is in a plane.

This is why the design of a bridge cannot be developed ignoring the construction method, which is really part of the design.

1.3 Control of the design

But it must be insisted on the major goals of the design : safety, functionality and elegance. The erection method and through it the construction equipment are major parts of the design, with a decisive impact on the project economy ; but they must not dominate the design and take such an importance that safety, functionality and elegance could be affected.

As already shown (Part 1 chapter 6), alternatives proposed by contractors to adapt the design to a different erection technique, or to its existing construction equipment, must not change the bridge operation condition - its functionality -, nor its elegance. To give one example, when the bridge alignment has to be curved and different from a pure circle with a constant radius, it is unacceptable to design a box-girder with its webs following a perfect circle to allow for a construction by the incremental launching method, with slab overhangs of very variable width on both sides which would give a very poor aspect to the structure.

The development of design and build contracts gives a major responsibility in design to contractors, with evident advantages as regards the selection of the construction method and the resulting economies. But some other construction actors must resist to an extreme influence of the construction considerations. . For an example, when a very large bridge is made of a series of typical viaducts in a very uniform site, it is unacceptable to reduce the length of side spans even if this would give a more efficient distribution of bending forces and allow for a more practical construction (for specific construction techniques) ; the longitudinal profile and the bridge alignment must be selected considering the elegance of the bridge line and not only to reduce the cost of piers by lowering the deck as much as possible ; when the shape of piers has to be adapted to allow for an easy passage of a launching beam, this must be done with elegance, even if this leads to a slightly increased cost.

1.4 Goals of this second part of the guidance for a good bridge design

Considering the importance of erection techniques in the development of the bridge design, this second part of the guidance for a good bridge design will begin by a description of the different construction methods, with their advantages and drawbacks, with their specificities and of course with the definition of the domain of their most efficient applications ; the following chapter will be devoted to the design of the cross-section, detailing what was already initiated in Part 1 chapter 6 ; finally, a last chapter will insist on the influence of the construction technique on the design.

2 General comments on methods of construction

2.1 Insitu Construction on Scaffolding or Stationary Falsework

The use of scaffolding or stationary falsework to support formwork for insitu concrete construction provides the designer with the greatest degree of flexibility with regard to the shape and form of a structure. This type of support is suitable for the construction of both simple and complex structures, particularly bridges built over land provided that the ground can provide a suitable foundation and that the structures are neither too high nor too long.

The use of classical scaffolding, however, is not suitable for bridges over water or other significant obstacle, due to the relatively high cost of using this material to create a temporary structure which is capable of spanning any significant distance.

2.1.1 Classical Scaffolding

Scaffolding consists of cross-braced posts or towers which are designed to transfer loads from the formwork to the ground. The most widely used form of scaffolding is the metal tower, comprising standardised, re-useable cross-braced modules.

The apparent simplicity of this type of support system can be the cause of insufficient care and attention being given to its design and erection, as a result of which accidents are not unknown.

The most important considerations relating to the stability of scaffolding are:

- A suitable number of posts or towers should be provided, such that each transfers a moderate load only to the ground. It is also essential that small settlements only should occur.
- Posts or towers should not be located in areas of doubtful stability, such as on slopes or near the tops of steep slopes.
- The scaffolding must be adequately braced in order to maintain its rigidity and to resist the effects of wind and other horizontal forces. Particular care should be given to its erection in order to avoid eccentricity of loading.

2.1.2 Stationary Falsework

Stationary falsework differs from classical scaffolding in that loads are transmitted to the ground by means of a limited number of structural frames, rather than a multitude of relatively small posts or towers. A typical system may support the formwork by means of a grillage of steel girders which, in turn, would span between steel trestles.

The normal practice is to use standard rolled steel beams for the supporting members, with their span usually limited to between 10m and 20m. It may therefore be necessary to provide temporary intermediate supports when using such a system to construct a bridge over a river, provided that this does not interfere with navigation.

2.1.3 Stages of Construction

In the case of short bridges, the entire structure is supported on stationary falsework and the whole bridge is concreted in a single stage. The prestressing cables may be tensioned when the concrete has achieved sufficient strength, after which the falsework may be removed.

The same construction method is not usually suitable for longer bridges, due to the cost of providing an overall falsework and the problems associated with placing large volumes of concrete in a single pour. It is therefore more usual to construct such bridges in a series of stages, thereby reducing the amount of concrete to be placed at any one time and permitting the re-use of a relatively small amount of falsework.

The number of stages will be dependent upon the length and configuration of the structure, and it will be necessary to at least partially prestress each stage prior to the removal of the falsework. The most common sequence is to construct these bridges on a span-by-span basis, although it is also possible to construct two or more spans in a single stage.

When many stages of construction are required, it may be economical to provide some form of (partial) mechanisation to assist with the moving of the falsework. The ultimate form of this mechanisation is, of course, a launching girder or gantry (figure 2.1).

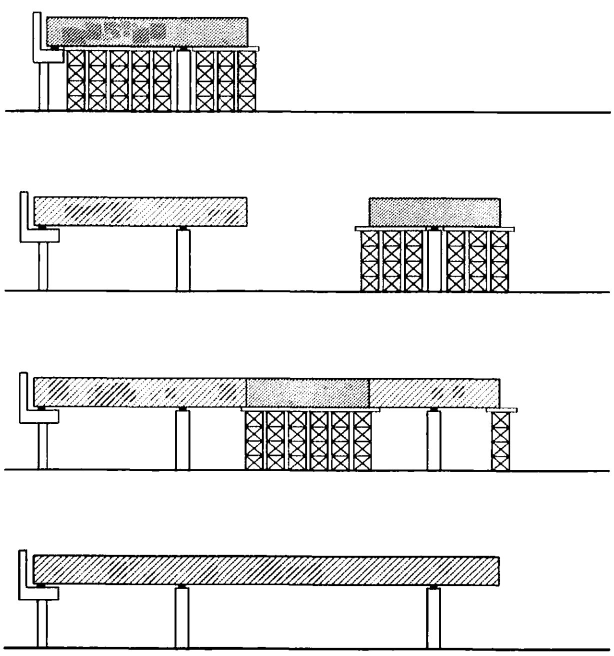
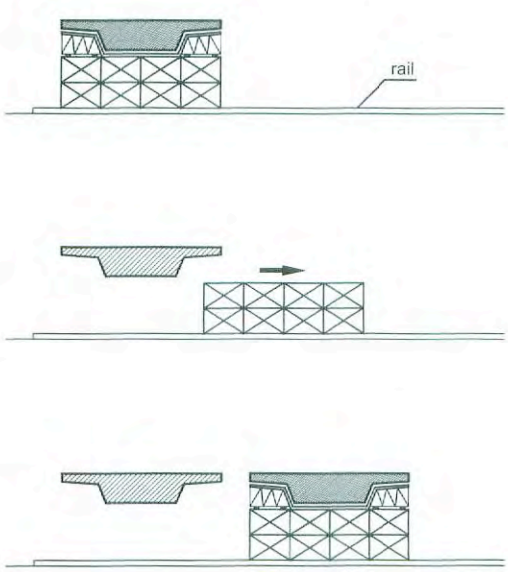


Figure 2.1: Construction in a series of stages

Although most long bridges are constructed in a series of longitudinal stages, it may be worth bearing in mind that wide bridges which comprise several identical structural elements may, alternatively, be built in a series of transverse stages (figure 2.2)



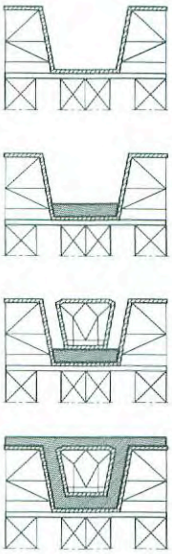


Figure 2.2 Transverse stages

Figure 2.3: Concreting stages

In the case of box-girder bridges, it is rarely practical to place concrete to the entire cross-section in a single operation. For these structures, it is usual for the bottom slab to be cast first and allowed to harden prior to placing the formwork for the remainder of the section. The webs and top slab may then be cast in either one or two operations (figure 2.3).

Figure 2.4: Vasco de Gama Bridge – Portugal [X]

Box-girders are invariably used for the construction of long spans. The spans may, however, be such that it is uneconomical to concrete their entire length in one operation, in which case it will be necessary to construct the bridge in a series of elements on the falsework for each span.

This sequence of construction leads naturally to the method of building by successive cantilevers, supporting the formwork for each element on falsework from the ground or using movable devices.

2.2 Launching Girder or Gantry

The use of a launching girder or gantry is only a particular application of the incremental method of construction described previously. The method requires, however, the use of a special movable girder or gantry which is supported from the previously completed parts of the permanent structure.

This erection method can be considered for spans in the range of 30 to 60 metres, although it is more often used for spans of 40 to 50 metres.

The use of this method enables long bridges to be constructed in a short time and, if there is a sufficient degree of repetition of the construction stages, can increase considerably the efficiency of the working teams.

These highly mechanised systems are used to support the formwork and then transport it from one span to another. They are mainly composed of the following components:

- A supporting beam (gantry or girder) for the stationary falsework,
- A system of bearings, specific to the various concreting and construction stages,
- The mechanism for moving the supporting beam,
- The formwork, with mechanical means of adjustment and removal.

The supporting beam for the stationary falsework is made of one or several girders, placed either above or below the deck. These supporting frames are known as gantries if they support the deck from above, or girders if they support the deck from beneath.

These support systems are suitable for decks of various cross-sections such as box-girders or ribbed slabs. They are used mainly for the construction of continuous decks where an entire span is concreted in one operation, usually with construction joints located at sections of minimum bending moment.

2.2.1 Launching Gantries

Gantries consist of a supporting beam located above the deck. This main beam supports several transverse members which, in turn, support the formwork and the working platform. It is common for the length of the beam to be approximately twice the length of the span, and for it to be fitted with a launching nose to facilitate its movement to the next span.

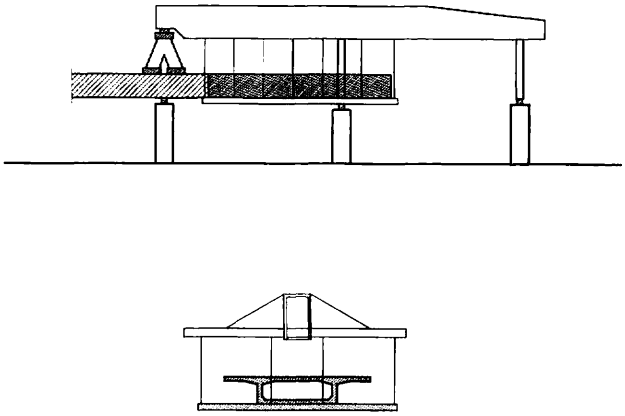


Figure 2.5: Launching gantry

When in position, ready for concreting, the rear of the gantry will be supported by a roller bearing, fixed to the gantry, which bears on the leading end of the previously cast span. The gantry will also be provided with a roller bearing on a temporary prop located near the pier of the span being cast, as well as a similar support to the launching nose at the next pier.

Once the span has been concreted and prestressed, the gantry is launched forward on its roller bearings, by means of the advancing jack, to the next position.

2.2.2 Launching Girders

The launching girders are made of several steel beams, usually box-girders or trusses, located beneath the deck.

This kind of girder is particularly suitable for ribbed slab bridges, because it is possible to locate one beam under each side cantilever slab and a few others below the central deck. In the case of a box-girder bridge, one or more beams have to be located under the box-girder, which is more difficult, and one under each side cantilever.

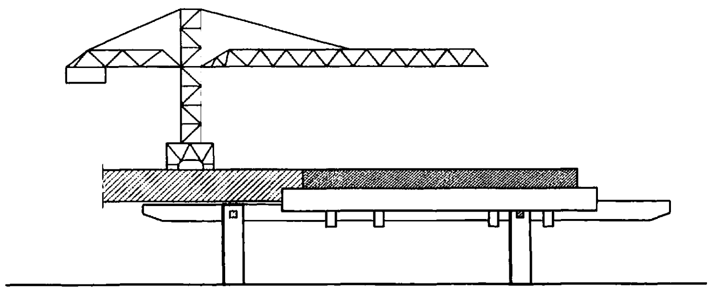


Figure 2.6: Launching girder

The principle of an incremental launching girder is the same as that of a launching gantry.

2.2.3 Advantages and Disadvantages of the Method

The main disadvantage of the method is certainly the cost of providing the launching gantries or girders, the latter being particularly heavy. The investment in such equipment can be recovered only for very long bridges.

The launching gantries are usually cheaper, due to the saving in material which results from being able to utilise a greater structural depth. This, in turn, leads to the general use of truss construction, which also has the advantage of increased stiffness. Gantries are, however, more difficult to use, due mainly to the requirement to provide hangers which have to pass through the structure to support the soffit, thereby making it impossible to prefabricate the reinforcing cages. They are also difficult to launch, due to problems in negotiating the piers, and the working platform is inherently difficult to reach.

With favourable conditions (i.e. long bridges with multiple identical spans), the use of the incremental falsework method allows an impressive rate of construction, up to one span per week, to be achieved. This can explain the profitability of the method, despite the high capital cost. This is particularly true for ribbed slab construction, where the entire section may be cast in one operation. The method is less economic for box-girder bridges, however, because the need to cast the section in at least two operations leads to a reduced rate of construction.

2.2.4 The Use of Precast Elements

The amount of concrete required for a single span on construction stage can be very significant, and may often be more than can normally be placed in a single day (about 200 to 300 m^3 in such conditions). The use of precast elements provides a solution to this problem by enabling large quantities of concrete to be cast in advance.

The principles described above for using launching girders or gantries to support formwork for insitu construction are equally applicable for supporting assemblies of precast elements. This method of construction has proved to be very efficient, due mainly to the speeds of construction which can be achieved. The method can often enable two spans to be completed in a week, or even one span in a day in extreme cases (figure 2.7).

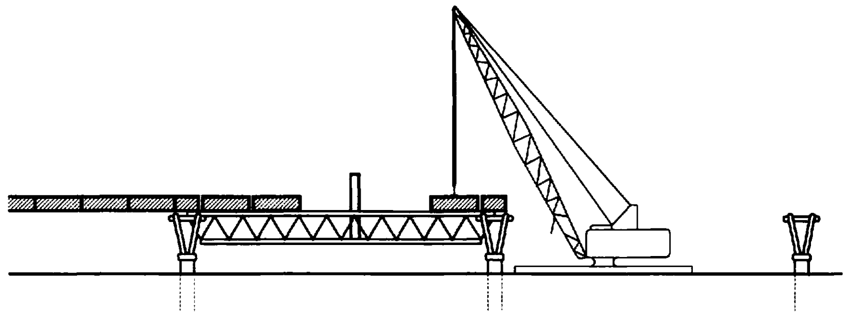


Figure 2.7: Gantry to support precast elements

A recent example of this method of construction is the Seven Mile Bridge in Florida (figure 2.8). For this bridge the precast elements (with the exception of the element over the pier) were assembled on a beam on a barge and partially prestressed to form a complete span. The barge was then towed into position beneath the span to be built, and the element over the pier was lifted into position by means of a launching beam supported on the previous span. The complete assembly of precast elements was then lifted into position on its beam and insitu concrete placed to close the joints at each end of the span. The use of rapid hardening concrete enabled the final prestress to be applied at an early age, allowing the assembling beam to be lowered onto the barge in preparation for a new cycle.

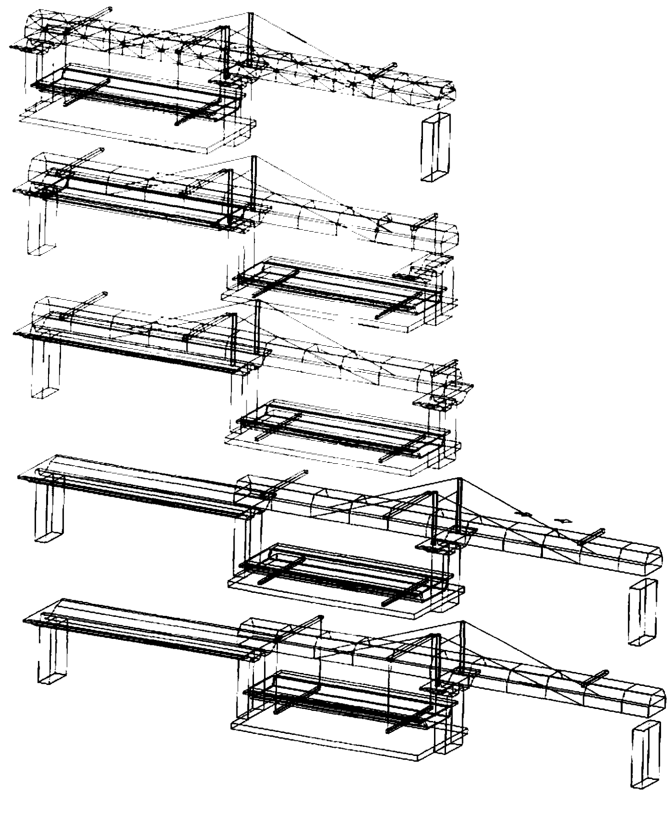


Figure 2.8: Lifting of a complete span formed of precast elements –Seven Mile bridge, Florida

Another example is the Bubiyan Bridge in Kuwait (figure 2.9). For this structure the precast elements were delivered along the previously completed parts of the bridge and loaded in the correct sequence onto a launching gantry. The gantry supported the elements until they were stressed together to form the completed span, after which the elements were released and the gantry advanced to the next span. The advantage of this method is that there is no need for insitu joints to be cast.

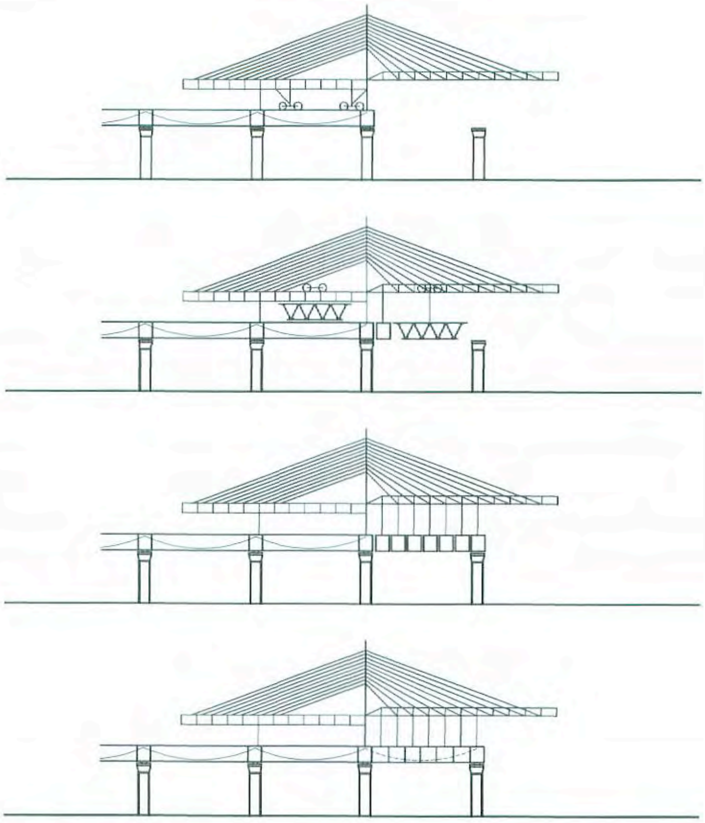


Figure 2.9: Launching gantry to support precast elements – Bubiyan bridge, Kuwait

Figure 2.10: Bubiyan bridge during construction

There are some similarities between this method and that of cantilever construction, although the bending moments induced during construction, and hence the prestress required, are significantly different. When a launching girder or gantry is used, the precast elements are capable of supporting themselves only after the entire span has been completed and prestressed. By contrast, cantilever construction enables each element to be self-supporting as soon as its increment of prestress has been applied. Cantilever construction, however, can produce bending moments which are up to five times greater than those resulting from the use of a launching gantry or girder, which is why it is often more efficient to provide some system means of temporary props or stays.

2.3 Cantilever Construction

This method of construction offers many advantages, and has therefore become very popular. Bridges built by the balanced cantilever method can be built for spans from about 50 metres (and even less in specific conditions) to 200 metres and even up to 300 metres.

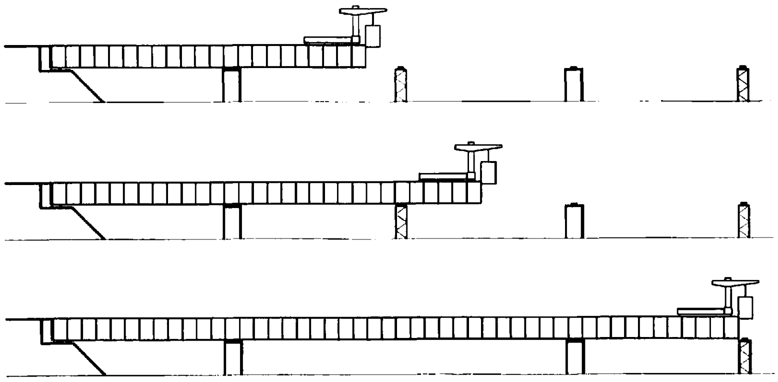


Figure 2.11: Incremental cantilever construction

The method involves assembling the elements, or segments, of the bridge deck by building outwards from the piers.

Each segment is prestressed as soon as it has been placed, thereby enabling it to support itself by cantilevering from its predecessor. This therefore reduces the loads which need to be carried by either a lifting mechanism or temporary support girder.

The most common use of this technique is the 'balanced cantilever' form of construction, where the segments are placed on either side of the pier alternately, although it is possible to use the incremental method where segments are placed sequentially from one end of the span to the other (figure2.12).

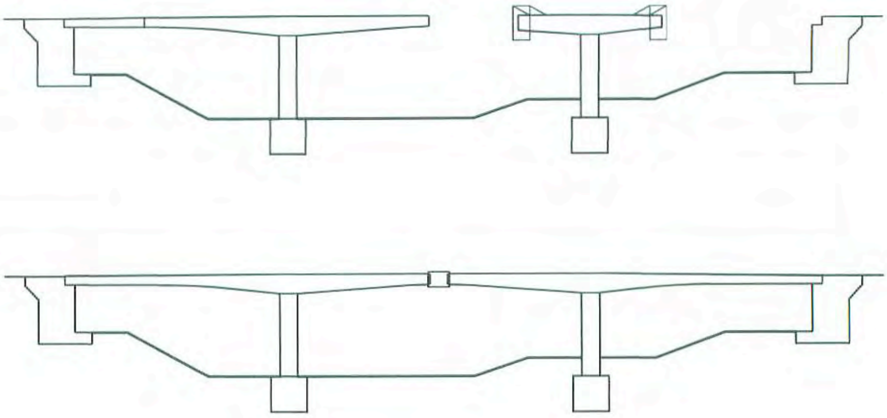


Figure 2.12: Balanced cantilever construction

As mentioned previously, the cantilever method can lead to very large and often unacceptable bending moments in the structure which may require the use of temporary intermediate supports or cable-stays.

The cantilever method of construction is not limited to a particular type of bridge, although it is usually associated with continuous beams of either constant or variable depth. It may, however, be employed for the construction of arch, portal and cable-stayed bridges.

Figure 2.13: Balanced cantilevering of the Jesse H. Jones Bridge – USA [18]

2.3.1 Construction Sequence and Span Distribution

The most usual sequence is to construct the cantilevers on each pier from the ends of the bridge and to progress towards the centre. For very long bridges it may be possible to construct several cantilevers at the same time, dependent upon the method used, the availability of materials and the speed of erection. The length of each pair of cantilevers will be the sum of he half-spans on either side of the pier.

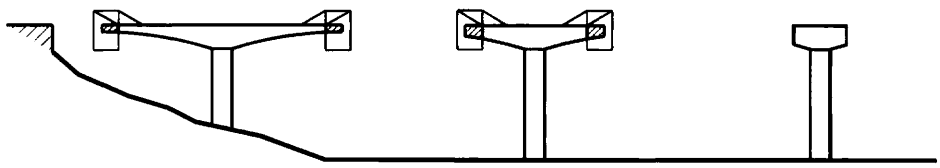


Figure 2.14: Balanced cantilevering of long bridges

2.3.2 Insitu Construction

With this method each segment is cast insitu, using formwork which is suspended from a steel frame supported from the previously cast segment of the deck. The steel frame is a movable falsework which supports the segment until the concrete has attained the required strength and the prestress has been applied. At this time the segment will be self-supporting, allowing the formwork to be released and the falsework moved to the next position.

A balanced cantilever bridge is always built symmetrically, and it is therefore necessary to provide a movable falsework at the end of each cantilever. The length of each segment is usually limited to between 3m and 4m, in order to limit its weight and avoid overstressing the previously cast part of the structure. Lengths of 5m or 6m can be achieved, but these can lead to problems with deformations.

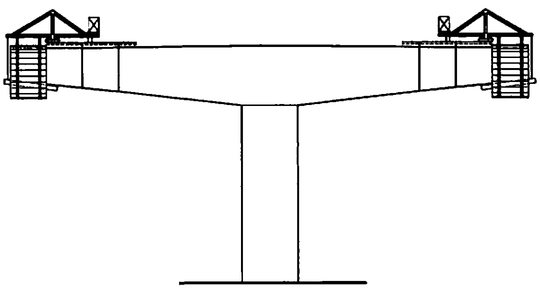


Figure 2.15: Movable falsework at the end of each cantilever

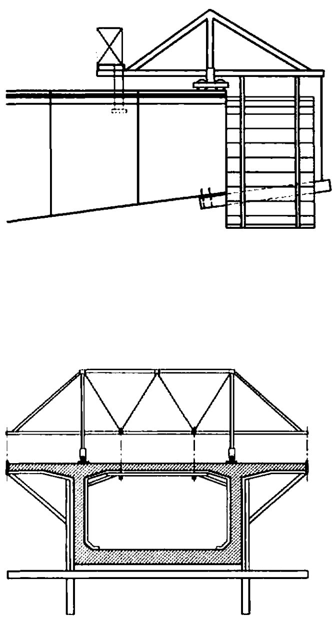


Figure 2.16: Typical movable falsework

The most common movable falsework consists of a steel frame mounted on the surface of the deck.

When in use, it is supported by the most recently cast segment at the tip of the cantilever and anchored at some distance back, usually in the next segment. It will therefore usually be necessary to provide a counterweight whilst the falsework is being moved.

When starting to construct a cantilever, it is first necessary to cast the segment at the pier. This segment will typically be between 7m and 10m long, which is approximately twice the length of the subsequent segments. The construction of this segment may take several weeks, owing to its complicated geometry and the large amount of prestress and reinforcement required.

Figure 2.18: Free cantilevering of the Felsenau bridge – Switzerland [14]

Each segment is usually cast continuously, starting with the bottom slab then progressing the webs and top slab. It is usual to provide openings in the inner formwork of the webs to facilitate adequate vibration of the concrete.

A typical rate of construction is two segments per week, with the prestress being applied when the concrete has attained a strength of 25 MPa. The use of chemical additives or high strength concrete can reduce the time required to construct each segment to 4 or even 3 days.

The movable falsework has to be rigid enough to avoid cracking at the joint between two segments, and for this reason the assembly is usually heavy (typically 50 to 80 tonnes). This weight can represent a significant temporary load on the structure which must be included in the calculation of stresses (particularly when the load is finally removed after the structure has been made continuous).

In addition to the most typical form of movable falsework described above, it is also possible to provide a system of falsework which is supported from beneath the previously cast portion of the cantilever (figure 2.19). These systems offer the advantage of being able to place the entire reinforcing cage in one piece, because the working area is completely free of obstruction, but their use can lead to problems of excessive deflection.

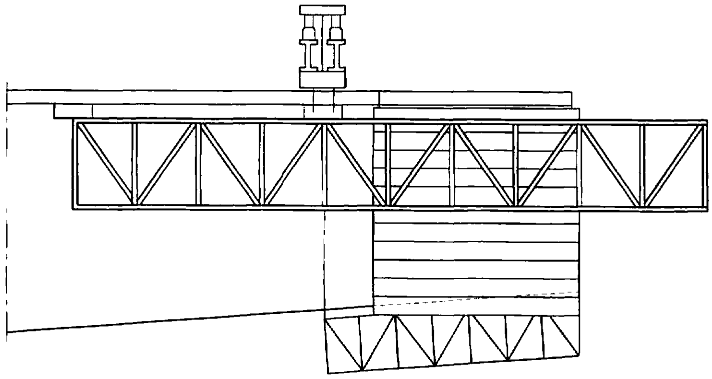


Figure 2.19: Falsework supported from beneath

2.3.3 The Use of Precast Elements

2.3.3.1 Length of the Elements

The length of the precast segments is related directly to their weight and to the means of transportation and lifting to be used. Typical lengths are in the range 1.8m to 3.5m, depending upon their width and depth. It is important that the segments are not too long, in order to limit the bending moments and to avoid having to anchor a large amount of prestress at one location.

Because of its greater size and weight, the segment over the pier may have to be delivered to the site in sections and subsequently prestressed.

2.3.3.2 Assembly of the Elements

An important consideration of precast construction is the treatment of the joints between the segments. There are three main types of joint: coupled joints, mortared joints and insitu joints. Of these, coupled joints are the most common.

With coupled joints, adjacent segments are 'match cast' (the segments are cast against one another) to ensure perfect alignment when they are in position and stressed together.

Experience has shown, however, that these joints are not watertight even when prestress is applied across them. The prestressing cables which pass across the joint are therefore vulnerable to corrosion, particularly because of the need to provide a joint in the sheathing at this location. For this reason it is normal practice to fill the joint with an epoxy resin glue.

The purpose of this glue is:

- to lubricate the matching surfaces, thereby making it easier to position the segment correctly,
- to fill and seal the joint completely upon the application of prestress,

- to avoid the risk of stress concentrations which could occur due to any imperfections of the precast surfaces,
- to improve the shear capacity of the joint.

The disadvantages of this method of construction are the cost of the glue and the additional time required for it to be applied and achieve the necessary strength. There is also the possibility of inadequate shear capacity of the joint in the event of the glue not attaining its specified strength.

A mortared joint is an alternative to a glued joint. In this case, the precast segments are placed approximately 3cm to 7cm apart and the joint then filled with fresh mortar.

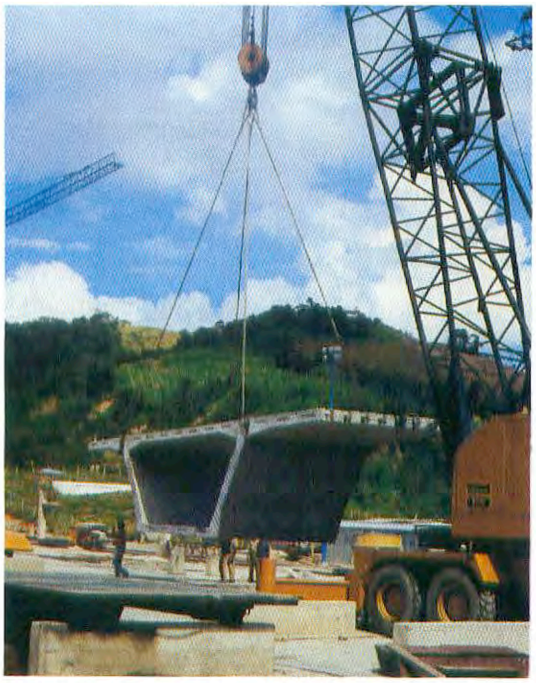


Figure 2.20: Lifting of a precast element – Romulo Betancourt bridge – Venezuela [18]

Joints can also be made with insitu concrete, provided that the gap between the precast segments is at least 15cm to 20cm. This type of joint, however, requires the use of additional formwork fixed to each segment, which experience has shown can be difficult to align.

2.3.3.3 Precasting the Segments

The precasting of the segments may be carried out either on site on at a specialist facility. Careful consideration has to be given to the geometry of each segment, in order to achieve the intended final profile of the structure. For this reason it is essential to make allowances for all the short-term and long4erm deformations of the segment which will occur during erection and in service. The complexity of this problem depends upon the complexity of the bridge.

2.3.3.4 Placing the Segments

The method used to place the precast segments will be dependent upon the nature and constraints of the project, as well as the availability of plant and materials.

The most common method is to provide a steel truss, approximately twice as long as the span, from which to support the segments. This method is suitable only if the segments are to be delivered along the previously completed deck. An alternative is to deliver the segments, either by land or water, to beneath the structure and to lift them into place by means of a crane mounted on the tip of the cantilever. In either case, it is usual for each segment to receive either temporary or partial prestress at the time of placing, and for the final prestress to be applied when the bridge is complete.

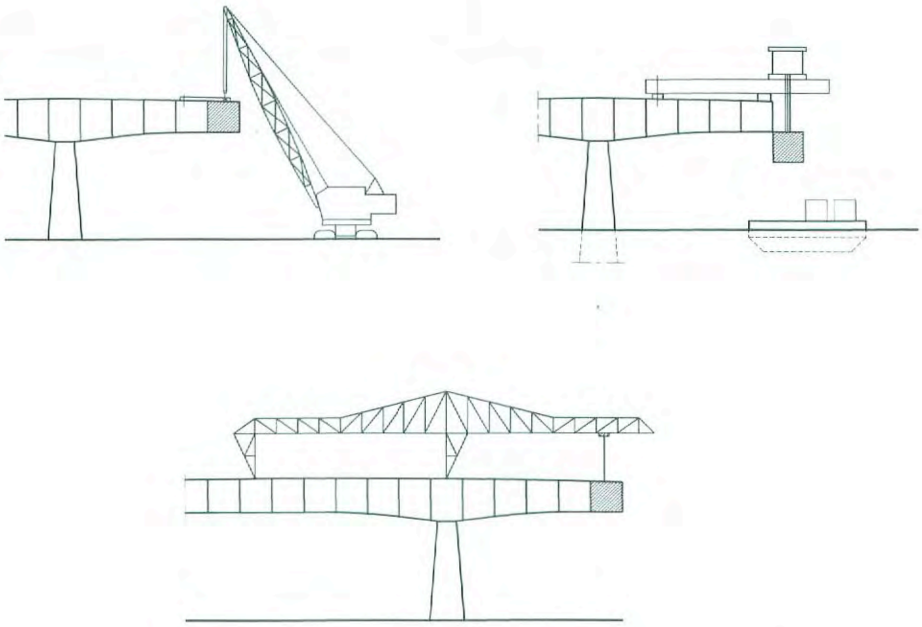


Figure 2.21: Different method of lifting and placing precast elements

Figure 2.22: Launching girder of the Ré bridge – France

2.4 Incremental Launching

The principle of incremental launching is extremely simple. The bridge is concreted on the ground, in a succession of segments, in a static concreting facility located at one end of the bridge. When each segment is cast, it is partially prestressed to the previously-completed part of the structure. The whole assembly is then advanced forward towards its final position, thereby enabling the casting area to be cleared and prepared for the construction of the next segment.

The weight of the whole deck placed by incremental launching can be very high and reach several tens of thousands of tons (65000 tons for the ship canal-bridge at Houdeng in Belgium – figures 3.2 and 4.10).

The method is suitable for the construction of long bridges with various spans. The range of suitability is varying from 15 to 20 metres for slab decks, from 60 to 70 metres for box girders and even more when temporary supports can be used.

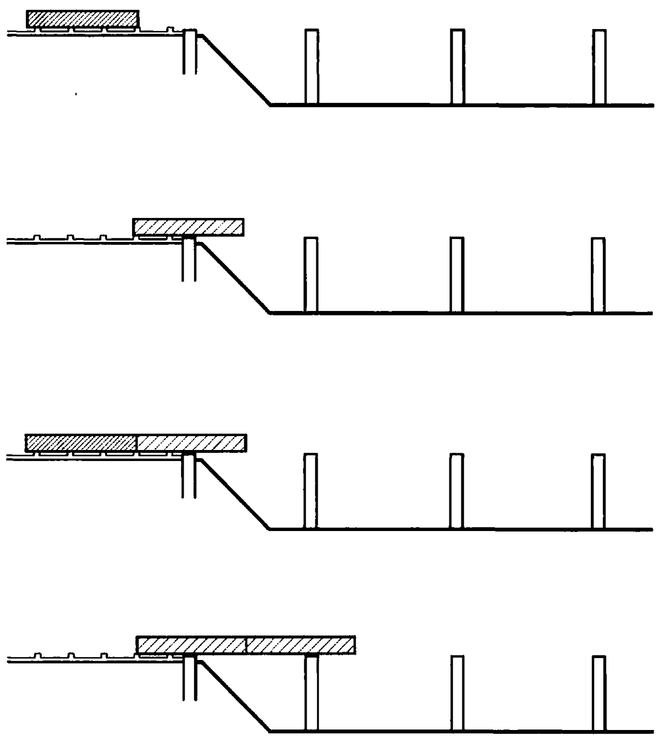


Figure 2.23: Illustration of the incremental launching construction method

It is most usual to provide one casting facility only and to launch the entire bridge from one end until it reaches its final position. The length of each segment can range between 12m and the full length of the span.

Each section of the bridge will be subjected to a considerable range of bending moments during the construction process, due mainly to the reversal of sign from sagging to hogging as the section passes over an intermediate pier during the launch. For this reason it is usual to provide a uniform prestress to the section during launching, although eccentric prestress may be required in the first span where the bending moments are predominantly hogging.

Once the launching process has been completed, additional prestressing cables are added in the upper parts of the section over the piers and in the lower regions within each span in order to resist the bending moments which will experienced due to subsequent live loading on the bridge. The design of the prestress, therefore, is dependent upon the full range of bending moments experienced by each section during the launching process and whilst in service. If it is not possible to apply the prestress m stages, then it will be necessary to greatly reduce the spans of the bridge, or take other measures, in order to limit the hogging bending moment experienced during launching.

2.4.1 Launching Nose

The most common solution to the problem of the hogging moments experienced during launching is the use of a launching nose. These may often have a length equal to 60% or 80% of the span and are intended to reduce the hogging moments by spanning onto the next pier as soon as possible during the launching process.

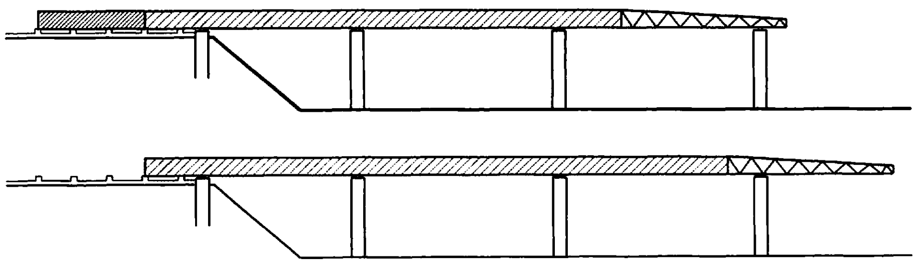


Figure 2.24: Launching nose

The launching noses are usually of steel construction, and are necessarily light and rigid compared to the remainder of the span in order to minimise the cantilever moments. The front part of the nosing is also of benefit for landing on subsequent piers.

2.4.2 Temporary Supports

The easiest solution to the problem of excessive dead load bending moments during launching is to provide temporary intermediate supports (figure 2.25). Intermediate supports placed mid-way between piers, for example, would reduce the dead load bending moments by a factor of four.

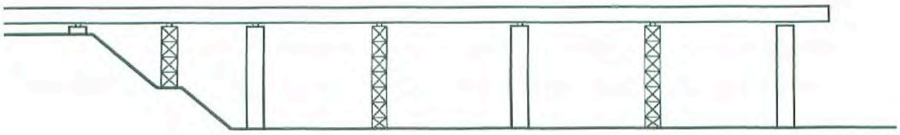


Figure 2.25: Temporary support for incremental launching

If temporary supports are used in conjunction with a launching nose, then the effect of the bending moments produced by the launching can be reduced to a level at which they may no longer be significant.

Figure 2.26: Launching nose of the Maribyrnong River bridge - Australia [25]

2.4.3 Launching. Cable-Stays

Another solution is to use an auxiliary mast for cable-stays when launching. The mast should be located approximately a span length behind the front end of the deck. Cable-stays attached to the mast support the front end of the deck and are anchored approximately a further span length to the rear of the mast (figure 2.27).

When launching, it is essential to allow the tension in the cable-stays to be varied, in order to avoid over-stressing the deck when the mast is located at mid-span.

Launching cable-stays may also be used with a launching nose, although it is usual to combine these two temporary structural elements for the longest spans only.

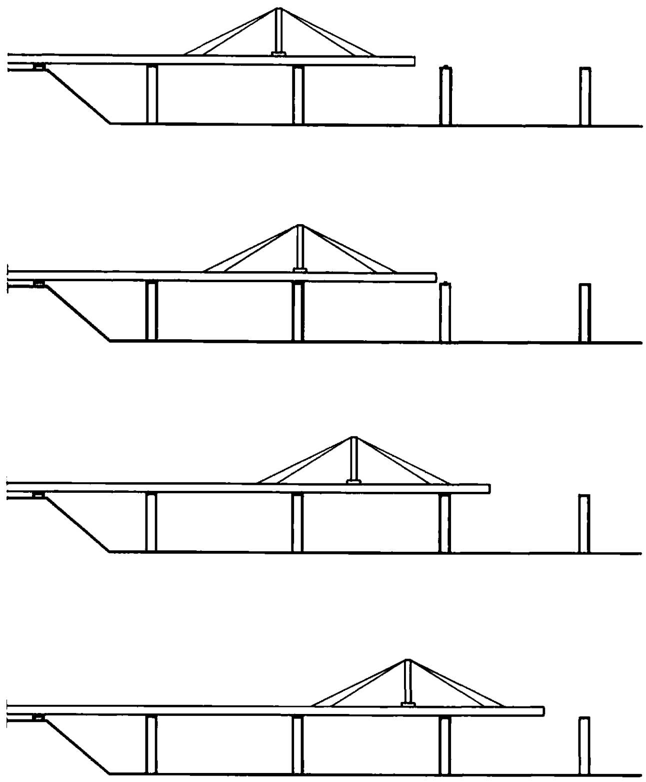


Figure 2.27: Temporary stays for incremental launching

2.4.4 Concreting and Launching Plant

Special care has to be given to the design of the concreting sequence and the layout of prestress, in order to avoid an undesirable final profile of the structure. Consideration has also to be given to the deflection of the formwork and falsework, because these can cause discontinuities in the profile which can detract from the appearance of the deck and can induce significant parasitic bending moments.

It is therefore necessary to establish, behind the abutment from which the deck will be launched, a concreting facility which includes a system of very rigid formwork mounted on temporary bearings (figure 2.28). The facility should be located far enough behind the first span in order to ensure its stability and to facilitate a suitable layout of prestressing during the first stages of launching.

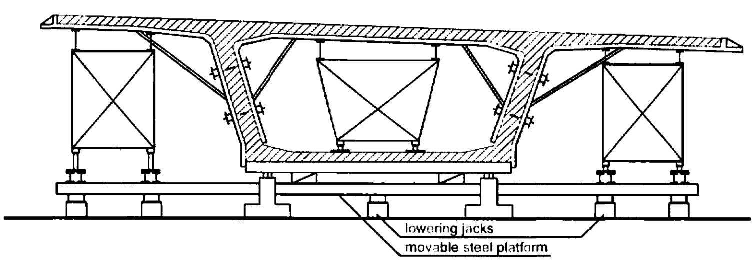


Figure 2.28: Concreting facility behind the abutment of the launched deck

In order to launch a deck weighing several tens of thousands of tons, it will be necessary to support the structure on a series of sliding bearings which offer only a minimal resistance to movement and to provide a jacking system capable of overcoming the combined effects of friction and inclination (figure 2.29).

Temporary sliding-guided bearings are provided at each of the piers and any temporary intermediate supports. These bearings usually comprise a neoprene pad to facilitate rotation of the structure and a PTFE/stainless steel sliding surface to minimise friction.

The jacking system, usually located at the launching abutment, may be one of the following types:

- either an arrangement of horizontal jacks which pull the structure by means of cables, or
- an arrangement of horizontal jacks which react against a thrust block to push the structure directly. The most well-known system, now patented, incorporates vertical jacks to support the bridge and horizontal jacks to provide the launching movement.

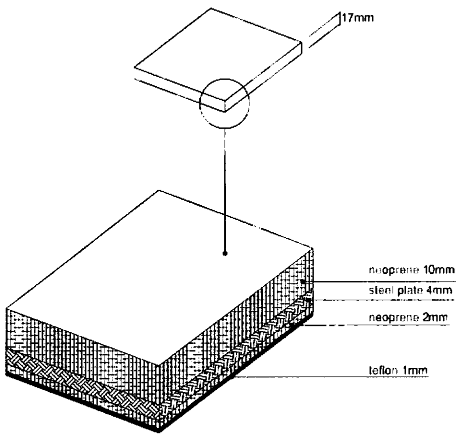
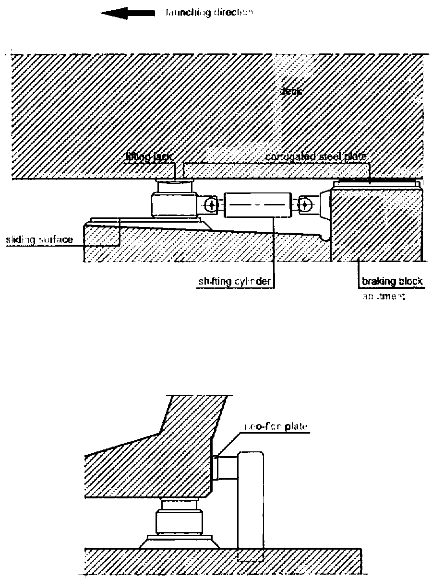


Figure 2.29: Temporary sliding bearings



Example of a jacking system

The jacking system has to be able to provide the necessary force to overcome the combined frictional resistance offered by the sliding bearings (3000 tons for the ship canal-bridge at Houdeng in Belgium) plus, or minus, any resistance arising from the longitudinal slope of the bridge. During launching the friction forces may be in the range of 1% to 3% of the vertical load, although they may be as high as 4% or 5% at the start of the process. It is therefore usual to adopt a figure of 5% when designing the jacking system and the supports for the temporary bearings.

Figure 2.30: Scardon Viaduct during launching – France, [24]

2.5 Other Methods of Movement

2.5.1 Rotation

Constructing a bridge on scaffolding can be economical, provided that it is not necessary to clear major obstacles such as rivers, railways or highways.

It may, however, be possible to avoid such difficulties by constructing either part or all of the structure on traditional scaffolding, parallel to the obstacle to be crossed. Partial prestressing may then be applied, sufficient to enable the falsework to be removed, prior to rotating the structure about one of its bearings. It is usual to rotate the bridge about the pier closest to the obstacle to be crossed, taking care to ensure that the cantilever is properly balanced.

For long spans or for bridges with three spans, a double rotation is required. One half of the bridge is built on each side of, and parallel to, the obstacle to be crossed. Each half is rotated into position and connected by means of an insitu joint (figure 2.31a).

For shorter spans or bridges with two spans, only one rotation is required. The span which is to cross the major obstacle is constructed and rotated about the main pier as described previously. The remainder of the bridge is then cast in-situ (figure 2.31b).

The span range suitable for rotation is the same range as the cantilever method. It depends on the stability of the cantilever during rotation. This can reach more than 100 metres, more than 200 metres for the whole balanced cantilever beam. For specific structures like cable stayed bridges, longer spans can be constructed by rotation (figure 2.32).

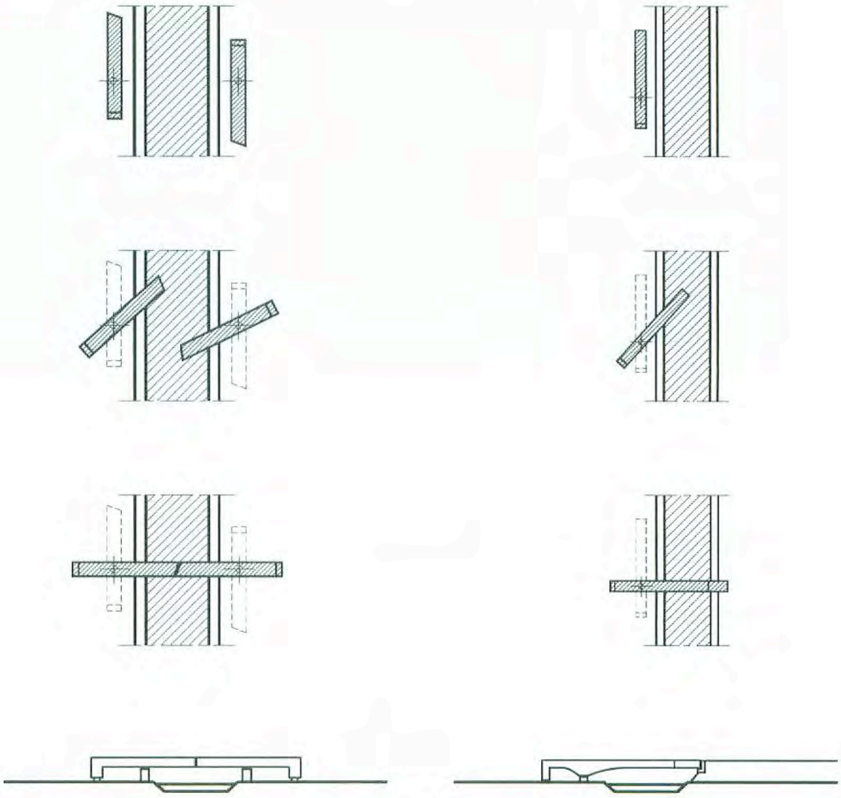


Figure 2.31: a) double rotation for a three span bridge b) single rotation for a two span bridge

Figure 2.32: "Père Pire" cable stayed bridge during rotation – Ben-Ahin, Belgium, [2]

2.5.2 Transverse Movement

I happens that a bridge has to be build to support an existing way (road or rail) without interruption of traffic or only within a very short time.

There is also an increasing requirement to replace bridges which are in service, for reasons such as an increase of loading or because they may have suffered such damage that repair is no longer possible.

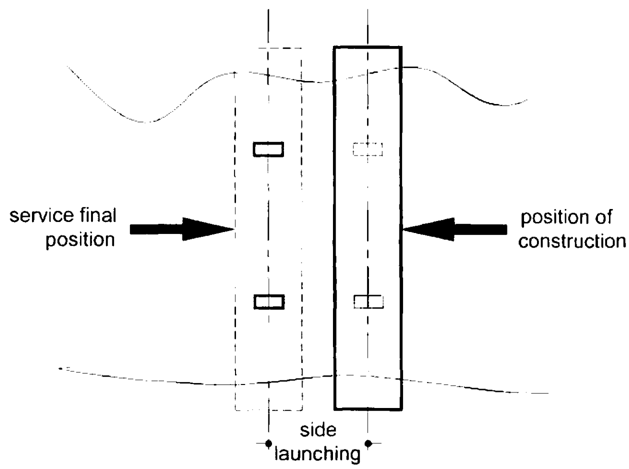


Figure 2.33: replacement of an old bridge or construction by lateral sliding

The building or the replacement of these structures is often inconvenient to users and may cause severe disruption to traffic whilst the work is being carried out. This disruption may be minimised if it is possible to build the replacement bridge alongside the existing structure. Traffic will then be disrupted only for the time required to demolish the existing bridge and slide or otherwise move the replacement structure into position. In some cases this procedure can be carried out within only a few hours (figure 2.33).

2.6 Precast Beams

The use of precast prestressed concrete beams is an economical form of bridge construction for spans up to 30m, if pretensioned, or 30m to 50m if post-tensioned. The use of precast beams will, of course, be most economical for structures such as long viaducts, where a large number of beams is required.

Viaducts with many spans may also justify the use of specialised construction techniques, as described previously. If the geometry is suitable (i.e. straight or of constant curvature both horizontally and vertically), then the launching method or the use of launching girders or gantries may be attractive alternatives.

The use of precast elements offers many advantages and makes it easier to prevent a repetition of the errors which were made in the past in the search for a short4erm economy (i.e. too thin webs, insufficient reinforcement, too little concrete cover and excessive congestion of reinforcement at the anchorages.

Firstly, the quality of concrete which is produced at a specialist pre-casting facility or at a similar installation on site will be better than that of concrete which is cast insitu on falsework. This is due, mainly, to the superior conditions within which to control the production of the material. In addition, if any concrete fails to achieve its specified strength then it is relatively easy to reject and re-cast the element. A similar occurrence with insitu concrete could result in demolition or expensive strengthening being required.

Secondly, it is easier to obtain many uses of the formwork and to produce a better surface finish. It is also possible to use fixed equipment for vibration, steam curing and other means of improving the quality of the product.

Furthermore, the use of precasting greatly reduces the congestion and obstruction to the working area, and increases the rate of construction of the bridge. The casting of the beams may commence at the same time as the construction of the substructure, provided that sufficient space for storage of beams is available.

2.6.1 Overall Design

2.6.1.1 Diaphragms

The beams are connected by the insitu concrete deck slab and by the diaphragms provided at the end of each span, close to the rows of bearings (figure 2.34). The diaphragms are designed to provide torsional restraint to the beams and to facilitate the raising of the deck by means of jacking should it become necessary to replace the bearings.

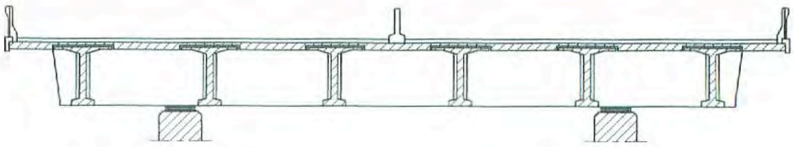


Figure 2.34: Cross section of a bridge made of precast beams with transverse diaphragms

It is no longer common practice to provide intermediate diaphragms at the quarter-span positions. Although they increased the rigidity of the cross-section and were relatively easy to design, they proved to be very difficult to construct. The main problems were the difficulty of fixing and removing the formwork once the beams were in position, and the need to provide starter reinforcement which had to project from the sides of the precast beams. When these intermediate diaphragms were omitted, however, it became necessary to increase the thickness of the deck slab in order to obtain the satisfactory transfer of load between beams.

2.6.1.2 Cross Sections

Currently, there are three common designs of cross-section of deck for girder bridges, with the choice between them being partly dependent upon the type of standard formwork favoured by the chosen manufacturer of the beams.

The first design utilises full-depth precast beams which are connected by short insitu slabs between them, requiring vertical construction joints. The second type uses relatively shallow precast beams which subsequently act compositely with the full-width insitu deck slab. This necessitates a horizontal construction joint at the interface between the top flange of the beam and the deck slab. The third type comprises precast I beams, rectangular at their ends, with narrow top flanges which support participating formwork for the full-width deck slab.

2.6.2 Placement

Precast beams are usually placed by one of two methods:

- by crane, provided that the ground beneath the structure is suitable (figure 2.35).
- This enables beams to be delivered to site and placed directly, without the need of temporary storage, or
- by launching girder, as described previously.

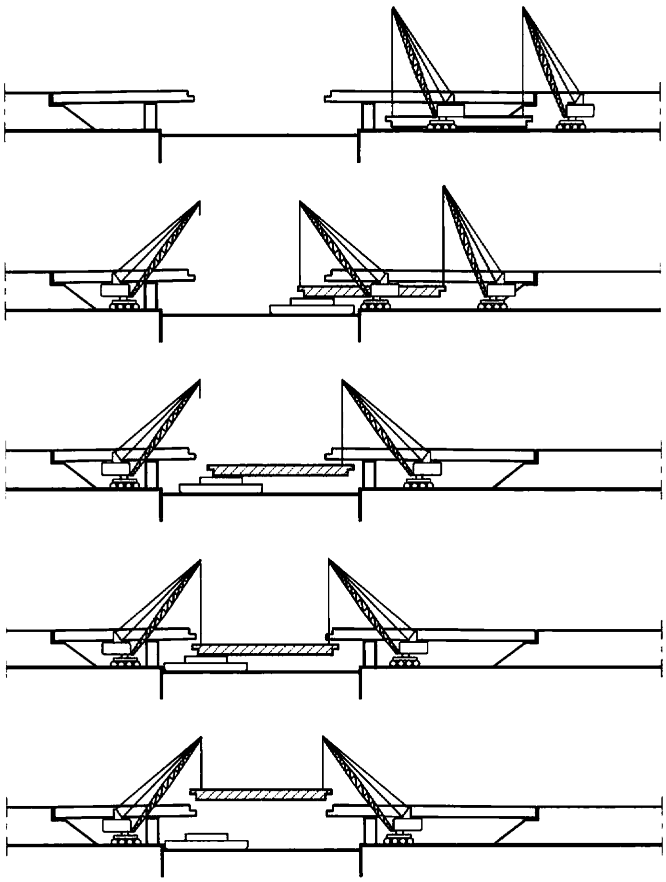


Figure 2.35: Placement of precast beams by crane

2.6.3 Prestressing

Beams may be either pre-tensioned, post-tensioned or a combination of both. An undoubted advantage of pre-tensioning is the greater degree of protection afforded to the steel by the structural concrete, compared to that afforded to post-tensioned cables by the grout injected into the ducting.

If it is necessary to apply second-stage prestressing after the insitu deck slab has been cast, then it may be necessary to anchor these tendons in the top surface of the slab if it is not possible to gain access to the ends of the beams for jacking. This is generally undesirable, because of the high risk of corrosion of these anchorages. It is also necessary to pay attention to the choice and installation of the ducts, particularly where they connect to the anchorages.

2.7 Transportation and Lifting of Heavy Loads

There have been several important developments in recent years which have had a significant effect on the construction industry. The move towards world-wide economy, the removal of the customs barriers and the increase of international exchanges are the reasons behind the rapid increase in the needs and possibilities of communication and co-operation between nations.

As a result, projects of a size which could not have been imagined twenty years ago are now being designed and constructed. Many of these projects are major bridge crossings of river estuaries or other stretches of water, whose length is measured in kilometres or tens of kilometres. Some of the most famous are:

- the Honschu-Shikoku bridge project in Japan,
- the Storebaelt and Oresund bridges in Denmark,
- the Lantau Fixed Crossing project in Hong Kong,
- the Prince Edward Island bridge in Canada.

Large consortia have been formed in response to these needs, each incorporating large financial and technical resources.

The increase in the number of these projects and the need to construct them within a relatively short period of time in order to make them profitable as soon as possible has inspired contractors to devise new methods of working and to develop mechanical equipment of phenomenal capacity.

A major objective in the design of long bridges which cross vast stretches of water is to minimise the amount of work which must be carried out on site. This objective has led contractors to devise methods of construction which require the handling of elements of ever increasing size and weight.

The methods used in the past few decades to transport and lift large steel elements such as bridges and oil platforms have, as a result of recent developments in the lifting ability of new cranes, been further developed to handle large concrete structures. Some significant examples are:

- Floating cranes or shear legs with lifting capacities up to 4000t. Combining two mast cranes, the Japanese have placed entire spans which were precast on the ground and transported to site on barges.
- In Denmark, for the Western bridge of the Great Belt, a crane with two hulls and a lifting capacity of 6000t has moved and placed over 3000 precast elements, including entire spans of 110m in prestressed concrete. This equipment has recently been adapted and is now able to lift loads in excess of 8000t.
- In Canada, the Prince Edward Island bridge comprises 44 spans, each 250m long. The precast pier and deck elements are being placed at the rate of one 250m span per week (figure 2.36).

Figure 2.36: Cantilever from Prince Edward Island bridge during placement by the Svanen barge (weight of element: 7800 tons) – Canada, [20]

2.8 Comparison of the Methods

Following the above descriptions of the various methods of constructing prestressed concrete girder bridges and before the next chapter which discusses the uses of the various types of bridge, their cross-sections and span ranges, it is worthwhile to compare the various methods available and to summarise the applications to which they are best suited (figure 2.37).

The main aspects for comparison are:

- the length of span possible,
- the speed of construction, including an average rate of construction (metres/week).

The first consideration is of great significance, because methods which use scaffolding, launching girders or gantries, precast beams or incremental launching without intermediate piers are not suitable for spans in excess of 60m or 70m. For spans in excess of 150m, the only suitable methods are insitu cantilever construction or the use of heavy handling equipment.

The second consideration is less important for bridges less than 400m or 500m in length, but becomes a major factor for structures which are several kilometres long. Even though the contract period for such structures may be 3 or even 4 years, it will almost certainly be necessary to make use of precast elements in order to achieve the required rate of construction.

There is a third consideration which has an influence on the choice of the method and the form of the cross-section of the bridge. The use of launching girders or gantries, precast beams or incremental launching are better suited to decks with only a limited variation of depth, whilst the methods based on formwork supported by scaffolding offer much more flexibility and facilitate the construction of variable sections. The methods based on the use of precast sections may be suitable when it is necessary to accommodate a moderate variation of section (particularly depth), provided that a significant number of elements are required. These methods are therefore in an intermediate category with regard to this consideration.

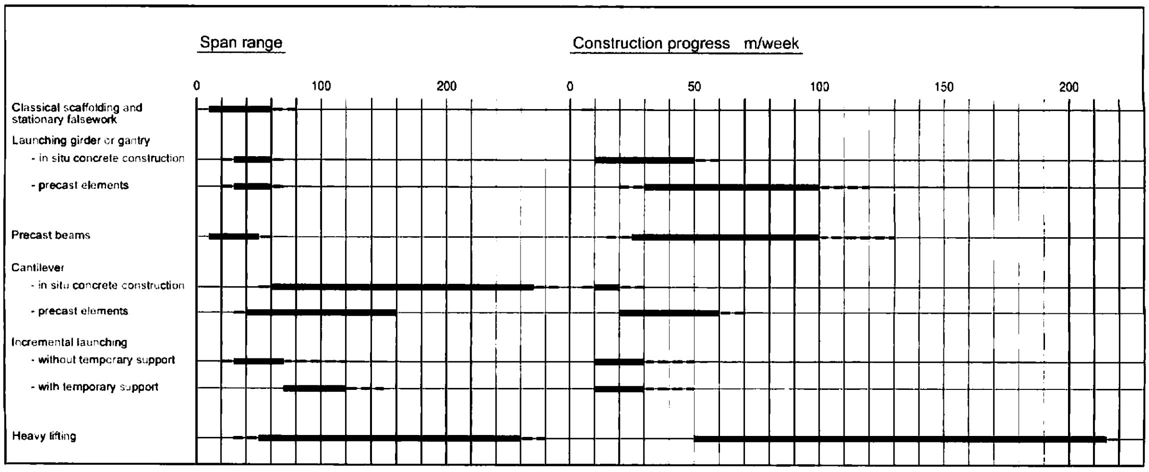


Figure 2.37: Comparison of the construction methods- erection speed vs span range

3 General comments on cross – sections

The forms of cross-section which are most commonly used for prestressed concrete bridges are:

- solid slab (either rectangular or with wide, tapering side cantilevers),
- voided slab (either rectangular or with wide, tapering side cantilevers),
- ribbed slab,
- beam and slab (which is a development of the ribbed slab, used for longer spans),
- precast beams,
- single or multiple cell box-girders.

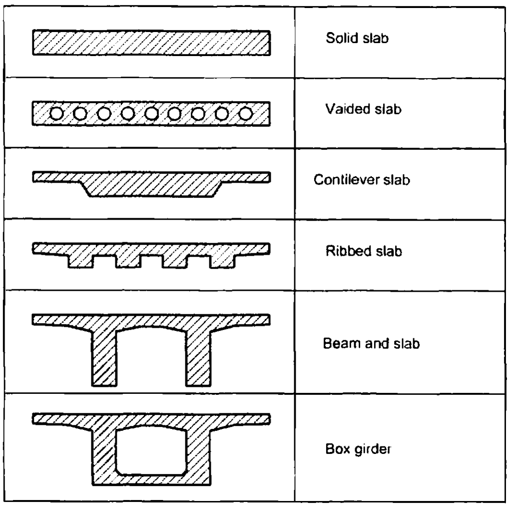


Figure 3.1: classification of cross section

3.1 Factors which Influence the Cross-section

There are many factors, technical, aesthetic and economic which have a greater or lesser influence on the choice of the cross-section.

3.1.1 Intended Use

The intended use of the bridge has the most significant influence on the essential elements of the structure, particularly the form of the deck.

For highway bridges, the width of the deck is determined by the need to accommodate the necessary width of carriageways and footways, as well as any additional width required for drainage, parapets, etc.

- For railway bridges, the width of the deck is determined by the standard track gauges and the consideration of operational and maintenance clearances, as well as the need to accommodate parapets and cable ducts.

- For footbridges and cycle bridges, the width of the deck will be dependent upon the anticipated number of users. A typical width of deck for a footbridge is 3.0m. For a combined bridge it has been found that a width of 5.0m between the railings is usually sufficient to accommodate separate lanes for the footway and cycle track.

- For special case like ship canal-bridges, the cross section has a particular U-shape adapted to this specific use. The loads are usually very important, due to the breadth (20 to 50 metres wide) and the water depth (3.0 to 5.0 metres). This leads to design very strong girders (figure 3.2).

Figure 3.2: Launching yard of the "Sart" Canal bridge – Houdeng, Belgium, [2]

- In urban areas, it may be necessary to provide structures which can accommodate roads, railways, pedestrians and cyclists. In these cases, due consideration must be given to the differences in level between the various parts of the deck as well as the need to provide adequate separation and protection to ensure the safety of the users.

- Most bridges are also required to accommodate services such as gas, water and electricity. It is therefore necessary to provide sufficient space for the various pipes and cables, and to make adequate provision for their installation, inspection and maintenance. This is most easily achieved within the section of a box-girder bridge.

- The choice of cross-section may also be influenced significantly by the consideration of accommodating additional lanes or even widening the structure in the future.

3.1.2 Overall Geometry

The overall geometry of a structure will have an influence on the cross-section, particularly if the plan curvature is such that a high torsional rigidity is required. The maximum span and the span configuration of the structure are, as described in the previous chapter, also influential.

3.1.3 Structural System

The form of the cross-section will be dependent upon the chosen structural system of the bridge, e.g. beam, portal frame, arch or cable-supported structure.

- The cross-section of a beam bridge will be influenced by the layout of the supports. Support can be provided by a single central column, double columns, leaf piers or multiple columns supporting a cross-head beam.
- The distribution of forces within a portal frame or arch bridge is dependent upon the configuration of the chosen structural system. The choice of cross-section will therefore be influenced by the layout of the structure as well as the forces acting at each section.

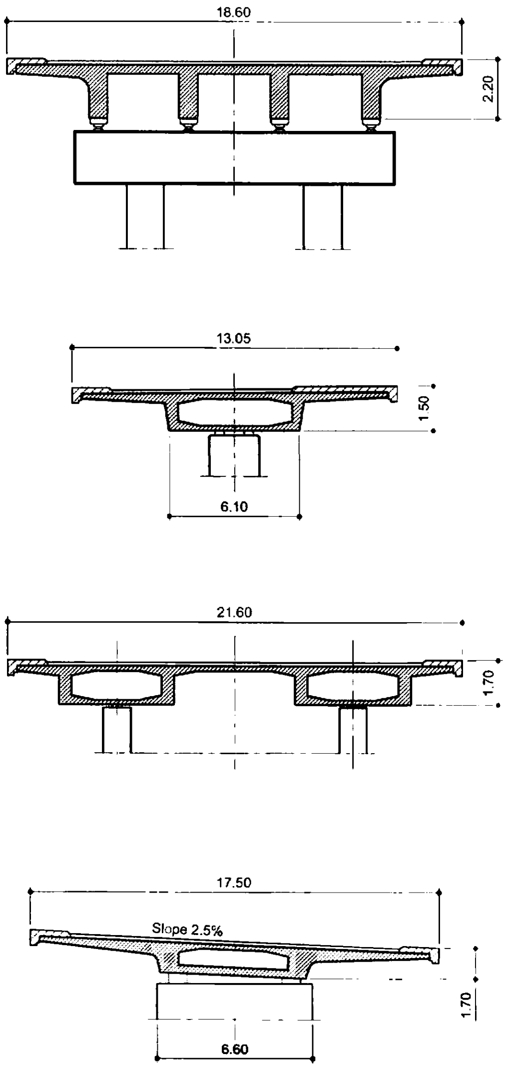


Figure 3.3: Geometry of cross sections

- For cable-supported structures or bowstring arches, the shape of the cross-section will depend upon the number of planes of supports to the deck. Such structures usually have one or two planes of support.

- If the deck is supported by a single plane of cables, usually placed centrally, then the deck will need sufficient torsional rigidity to be able to support asymmetrical loading. The provision of two planes of cables, however, reduces the need for torsional stiffness and generally permits the use of a more slender deck.

3.1.4 Architectural Considerations

Architectural considerations can also have an influence on the cross-section of the bridge, particularly if the designer is not constrained by the need to provide only an economic, functional structure.

The elegance of the structure depends largely upon the proportions of the cross-section, and may be achieved in the following ways:

- by lengthening the side cantilevers,

- by inclining the webs,

- by reducing the depth of the deck,

- by varying, when appropriate, the depth of the deck within a span, dependent upon the number of spans, the type of construction and the height of the structure above the ground.

- by considering the appearance of the structure when viewed from underneath.

Figure 3.4: Viaduc "della Biaschina" – Giornico, Switzerland [14]

3.1.5 Depth Variation

In prestressed concrete bridges, away from the bearing zones, the webs of beams or boxgirders are generally thicker than they need to be purely for carrying the applied forces. This extra thickness is required to accommodate the prestressing sheaths and to allow the concrete to be placed and compacted.

If the depth of the cross-section is decreased towards mid-span, typically following a parabolic profile, the dead load forces are reduced and, hence, the prestress required in the final condition is also reduced. A parabolic profile is particularly appropriate for bridges built by cantilevering as the variation in bending moments during construction reflects the variation in depth (see II.4.3). However, the formwork required is more complex and expensive than that of a bridge with a straight soffit.

For slab or ribbed slab bridges where successive spans are connected together at mid-span, a variation in depth is usually considered appropriate for spans over 30 to 35 metres. Box-girder bridges built by the balanced cantilever method with spans of less than 50 to 70 metres should generally employ a constant depth section. For spans over 100 metres, a variable depth girder is preferred.

Further consideration of variable depth bridges can be found in Part I, Chapter 6.

3.1.6 Deck Furniture

The need to accommodate deck furniture such as railings, parapets, lighting columns, traffic signs and overhead power cables (for railway bridges) can also have an influence on the cross-section. The location of such equipment outside the zones reserved for traffic and pedestrians requires a wider bridge and possibly a thicker deck slab. The amount of widening can be significant; for example, when lighting columns are located in the central reserve additional width may be required to accommodate two independent central safety fences with sufficient clearance from the lighting column.

The anchorage of deck furniture often requires local thickening and reinforcing of the deck slab, which increases the dead load of the bridge.

The fixing of deck furniture often introduces discontinuities in the deck waterproofing and its protective layer, which can become points of weakness where water can penetrate under the waterproofing. Damage caused to the concrete due to such leakage is frequently observed. Particular attention should therefore be given to the location of deck furniture as well as to the design, detailing and construction of any fixing details and holding-down arrangements.

3.1.7 Method of Construction

The influences of the method of construction, such as the use of fixed scaffolding, stationary falsework, launching girders, launching gantries, rotation, lifting, etc, on the cross-section are described in Chapter 4.

3.2 Economic Considerations

When determining the cross-section of the bridge, the designer has to take account of the many requirements of the structure, some of which may be contradictory.

One of his main objectives will be to devise a geometrically efficient cross-section which will enable him to make best use of the materials. This consideration may lead him, at first, to consider the use of a box-girder section rather than a relatively less efficient solid slab. Labour costs, however, have a major influence on the economy of a project, and the more simple shape of the solid slab may well provide the more economic solution when the greater labour costs associated with box-girder construction are taken into account.

For short spans, simple solid slabs generally provide the most economic solutions, despite their relatively inefficient use of material, because they are the easiest form of structure to build. For longer spans, lighter sections such as beam-and-slab and box-girders provide the most economic solutions, because the cost savings which result from the more efficient use of material (by concentrating it at the extreme fibres) out-weigh the additional labour costs.

The preceding comments are particularly true for insitu construction, because the cost of labour on site is relatively high. The choice of cross-section may be different if it is possible to make use of precast elements, for which the use of manpower is less expensive and more efficient, thereby reducing the labour time on site as much as possible.

Precasting is generally used for post-tensioned or pre-tensioned beams which are placed beneath the deck slab, or for bridges which consist of precast segments (most usually box-girder bridges). It is usually desirable to limit the weight of the precast elements or, in the case of segmental bridges, to obtain the maximum length for a given weight of unit in order to minimise the number of segments to be handled and hence reduce the construction time. In these cases it will be important for the designer to devise the most efficient form of section (i.e. minimum cross-sectional area), despite the fact that the resulting shape may be relatively complex and therefore slightly more expensive to produce.

Another method of reducing the weight of the elements is to partially precast the cross-section. It is possible to cast either the webs, diaphragms or the box-section without side cantilevers and to complete the section with insitu concrete. There are numerous possibilities for constructing a deck in this manner, provided that adequate details for the joints between the elements can be devised.

For very long viaducts for which the precasting of large elements is possible, there may be less need to minimise the area of the section (see Part 1, Chapter 6).

3.3 Ranges of Use

3.3.1 Slab Bridges

In most situations, the solid reinforced concrete slab is suitable for spans up to 15m and can be economic for spans of 18m or even 20m, particularly if the effects of self-weight can be reduced by making use of side cantilevers.

Solid slabs require greater quantities of steel and concrete than beams, but are easier to construct and require less formwork. There is also more opportunity for the formwork to be re-used. The reinforcement cage is relatively simple and can usually be fixed without the need of highly-skilled labour. Slab bridges are also shallower than beam bridges, which can be advantageous with regard to aesthetics and the quantity of earthworks required in the approach embankments.

For spans between 15m and 23m, it is more usual to prestress the solid slabs. They remain economical, however, due to their simplicity of construction and relatively low labour cost.

For longer spans, it is necessary to reduce the self-weight by introducing voids into the cross-section. These voids are created by excluding concrete from selected areas by means of formwork made from materials such as cardboard, fibre-reinforced cement, timber, expanded polystyrene or steel. The voids are usually located at the mid-depth of the slab, thereby having the effect of reducing the self-weight without significantly reducing the inertia of the section. Slabs of this type are suitable for spans up to 25m for a constant depth section, or 35m for a variable depth section (figure 3.5).

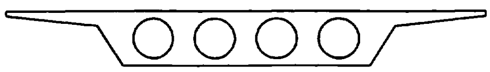


Figure 3.5: Reduction of bridge slab self-weight by introduction of voids

Voided slabs require particular attention during concreting, to avoid floatation of the void-formers due to the upward pressure of the fresh concrete. It is also necessary to consider the possibility of water entering the voids, either during concreting or whilst the deck is in service, and to provide drainage to avoid the risk of frost damage.

Spans in the range of 20m to 40m may also be accommodated by the use of ribbed slabs. These bridges are always constructed either on falsework, stationary scaffolding, launching girders or gantries. The main advantage of this type of structure is the reduction of self-weight (compared to that of a solid slab), but at the expense of a deeper section at the ribs.

Structures of this type usually have relatively shallow depths of construction. Span-to-depth ratios of 22 to 25 are typical for simply-supported spans, whilst values of 22 to 35 may be achieved for constant depth structures of reinforced concrete and prestressed concrete respectively. For voided slabs and ribbed slabs of variable depth, the span-to-depth ratios can vary from 20 to 25 at the supports and from 30 to 40 at mid-span.

3.3.2 Medium and Long Spans

3.3.2.1 Medium Spans

Medium span bridges are those with a main span in the approximate range of 30m to 80m. Possible types of construction for these spans are:

- ribbed slabs, built on fixed or movable falsework, for spans up to 60m,
- post-tensioned beams, for simply-supported spans in the range of 30m to 50m,
- launched box-girders, for spans in the range of 35m to 65m.
- balanced cantilever box-girder bridges, which more usually have spans between 50m and 80m.

Provided that there are no difficult ground conditions, viaducts comprising simply-supported spans of medium length may be constructed by using precast beams or ribbed slabs. These structures will be as economical as those constructed by launching, provided that the geometry is suitable.

Continuous box-girders with slender cross-sections are often used in urban situations, especially when a bridge with the shallowest possible depth of construction is required. The high torsional rigidity of these structures also makes then suitable for accommodating significant degrees of curvature.

3.3.2.2 Long Spans

Spans in excess of 80m usually require the use of continuous prestressed box-girders. These are most often constructed by the cantilever method, either using precast segments or cast insitu. The choice between precast or insitu is dependent upon many economical and technical considerations, including the preference and expertise of the contractor for the project.

Figure 3.6: Kochertal bridge under construction – Germany, [24]

This form of construction is generally used for spans in the range of 90m to 120m, although the longest span constructed to date is 300m. For spans up to 160m long, the use of precast segments enables a rapid rate of construction to be achieved (figure 3.6).

3.3.2.3 Depth of Beams

The depth of the beam or box-girder is determined by economic and aesthetic considerations, but can also be influenced by clearance requirements for roadway, railway or navigation.

The slenderness depends upon the shape of the cross-section, whether or not the depth is constant, the support conditions and the method of construction. For decks of constant depth, the span-to-depth ratio will normally be within the range 14 to 30.

The most economical span-to-depth ratio is approximately 15. This can give a deck a very heavy appearance if it is relatively close to the ground, but will have little influence on slenderness for taller structures. An increase in the ratio from 15 to 20 will have little effect on the cost of the structure, whereas a further increase to ratios between 25 and 30 will increase the cost to levels which can be justified only in special circumstances. For these reasons, the most usual span-to-depth ratios for decks of constant depth are in the range of 18 to 22, with the exception of launched bridges which normally require a ratio of 15.

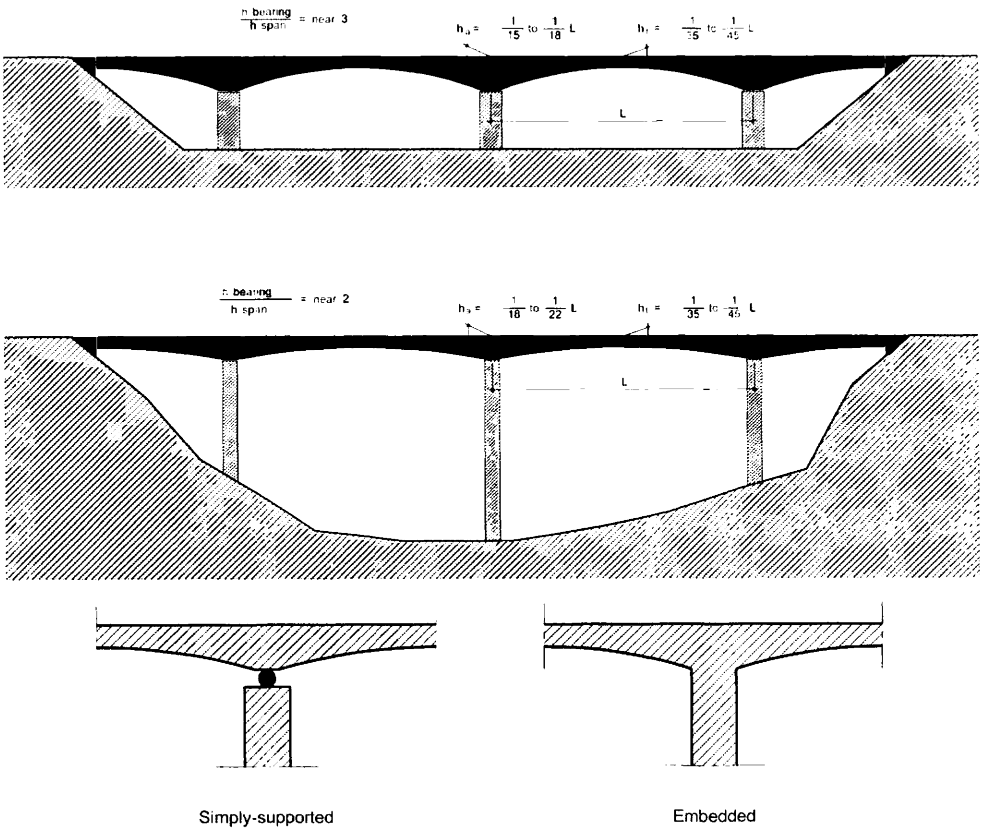


Figure 3.7: Span to depth ratio for variable depth structures

It is structurally and economically favourable to introduce a longitudinal variation of depth for spans which exceed 60m, and this should be adopted if there are no other considerations (e.g. aesthetics or clearance). For spans in excess of 150m, variable depth will be essential.

For variable depth structures, the ratio of the depth of section at the piers to that at mid-span will depend upon the height of the structure above ground. A ratio of 3 is appropriate for relatively low structures, tending towards 2 for the taller bridges. The span-to-depth ratio at the piers will be in the range of 15 to 22, with values at mid-span being in the range of 35 to 40 for continuous decks simply-supported on the piers and 40 to 45 for decks directly embedded on the piers (figure 3.7).

3.3.2.4 Ribbed and Box-Girder Bridges

As stated earlier, the most usual cross-sections for medium and long spans are those with T-shaped beams and box-girders. Box-girders have greater rigidity and strength, both torsional and flexural, than T-beams, and are usually easier to build except when precast beams or ribbed bridges can be built on launching girders or gantries.

In cold countries, the box-girder offers some protection to the underside of the deck slab which reduces the potential for damage due to frost action. Box-girders also enable easy access to services, compared to the underside of a ribbed or precast beam bridge

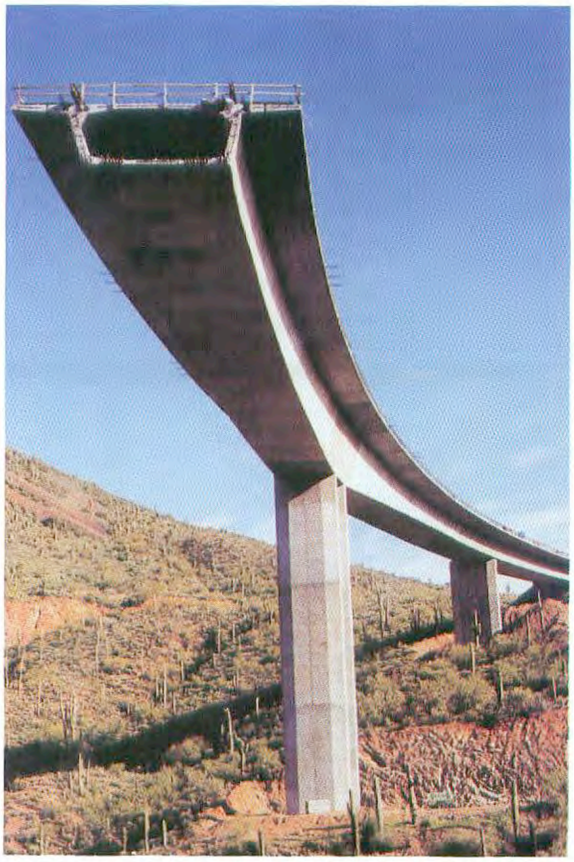


Figure 3.8: Lower Screwtail Bridge – USA, [24]

3.3.2.5 Number of Webs

Except for solid slabs, which are the most rudimentary type of cross-section, the width of the deck has considerable influence on the design of the cross-section.

Voided and ribbed slabs, with two or more ribs, vary the deck width in one of the following ways:

- the number of ribs is increased,
- the ribs are spaced more widely and the deck slab between is thickened,
- transverse beams are introduced, spanning between the ribs.

Of the first two, the current tendency is to limit the number of ribs and increase the slab thickness. The introduction of transverse beams makes the construction more complex, but is particularly suited to construction using precast elements.

In general, the number of webs in a ribbed or box-girder bridge should be as few as possible. The longitudinal flexural strength and rigidity are greatest for a given depth when the total width of the webs is least (figure 3.9).

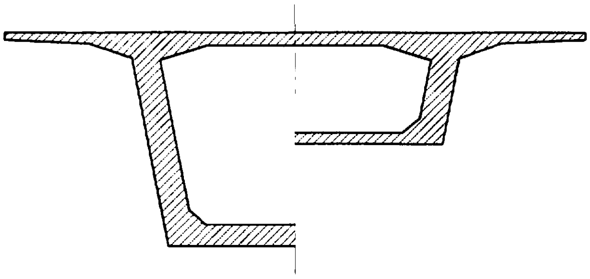


Figure 3.9: Elegance of a single box girder

In the last twenty years there has been a noticeable evolution in the design of bow-girder bridges. Previously the designer's objective was to design as light a bridge as possible in order to limit the quantities of materials and therefore produce an "economic" bridge. This resulted in multi-web box-girders with thin deck slabs (figure 3.10). Unfortunately, the durability of these bridges has often proved to be less than desired and they have not proved to be economic when measured over the whole lifetime of the bridge.

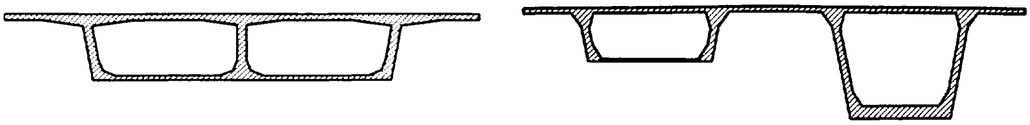


Figure 3.10: Examples of multi-web box girders

Bridges with more than two webs in the cross-section have exhibit a complex structural behaviour and the distribution of forces between the webs has often been incorrectly evaluated as it requires complex and expensive analysis. Furthermore, site labour costs have risen much faster than the cost of materials so that there is less benefit to be gained in reducing material quantities at the cost of using more human resources. Therefore, despite the sophistication of modern analysis tools which allow better modelling of the actual structural behaviour, the current tendency is to limit the number of webs to two, placed further apart, if necessary, leading to longer transverse spans and cantilevers. This results in a thickening of the deck slab and an increase in reinforcement quantities and, for wide bridges, transverse prestressing will be necessary.

In order to limit the thickening of the deck slab, transverse beams or external inclined struts can be introduced to support the cantilevers. For very wide box-girders, inclined struts can be provided within the box in order to support the deck slab. The struts can be either concrete or steel. In some applications precast slabs have been used for the struts, placed continuously so that they appear as an additional web.

As well as traditional systems, prestressed concrete three-dimensional trusses have been used successfully. The present trend in this form of construction is to combine concrete upper and lower slabs with steel or concrete truss webs and external prestress.

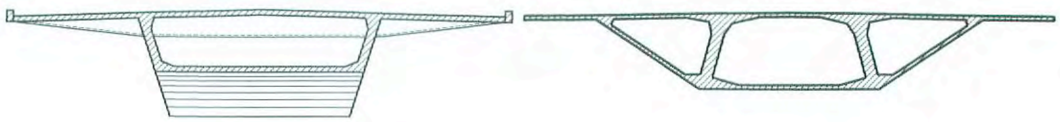


Figure 3.11: Transverse beams or inclined strut to support lateral cantilevers of wide bridges

Figure 3.12: "Eschachtal" Bridge – Rottweil, Germany [24]

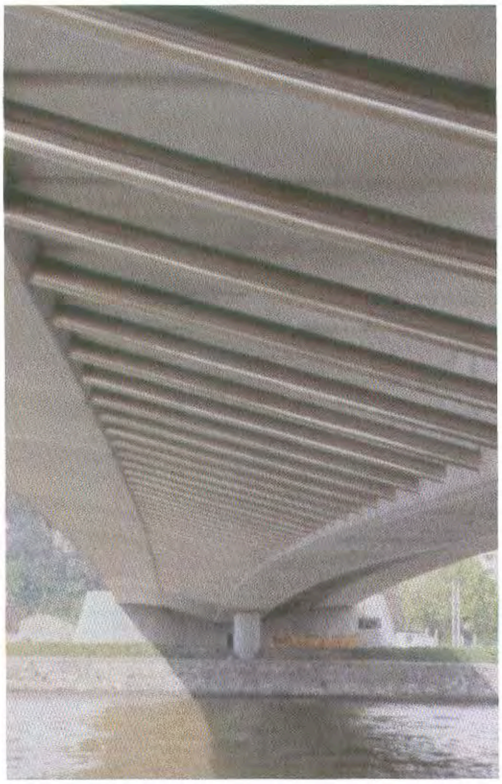


Figure 3.13: Stainless steel struts for the "Val-Benoit" Bridge – Liège, Belgium [2]

4 Influence of the method of construction on design

4.1 Construction on Scaffolding or Stationary Falsework

The use of scaffolding or stationary falsework is undoubtedly the best basic method of construction, and it offers the designer the greatest scope for the shape of the structure. The method is suitable for the most modest as well as the largest and most complex structures, including portal and arch bridges (e.g. Plougastel arch by Freyssinet). Since 1945, however, developments in prestressing have brought about great changes to the methods of construction, as a result of which the use of scaffolding and falsework is now limited to the more modest structures.

4.1.1 Conditions of Use

This method may be used economically in the following situations:

- structure is to be built over land, when the ground conditions are good,
- structure is relatively close to the ground,
- structure is of modest length,
- little settlement of the soil expected due to dead load of the concrete,
- no major obstacles to be crossed by falsework,
- suitable for span-by-span construction.

4.1.2 Influence on the Prestressing Tendons

It is most usual to construct the bridge on a span-by-span basis.

As soon as the concrete in the span under construction has attained adequate compressive strength, sufficient prestress is applied to enable the structure to support its self weight and to allow the falsework to be removed.

The falsework is then moved forward to prepare for the construction of the next stage. The process of concreting, prestressing and moving the falsework continues until the structure is complete. Construction joints between stages are usually located within a span, often near the point of contraflexure (figure 4.1).

In order to limit the effects of dead load, it will be beneficial if, at the end of construction, the resulting bending moments are as close as possible to those which would have existed if the entire structure had been supported on falsework and then stressed in one operation. Careful consideration should therefore be given to the length of the segments and construction stages.

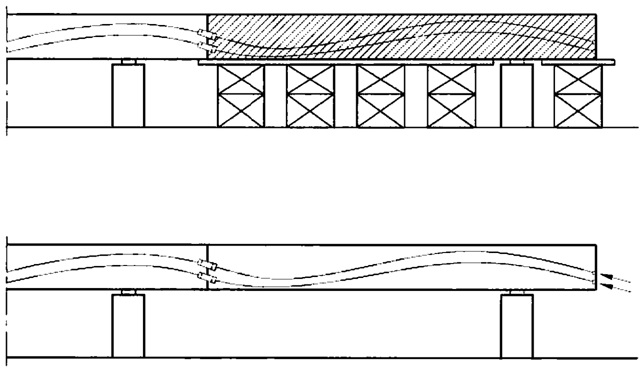


Figure 4.1: Prestressing geometry for span by span construction

There are several methods of prestressing these structures, as follows.

- The first method is to apply the full prestress to the span being constructed as soon as the concrete has attained the necessary strength. In order to achieve this, it is necessary to connect the prestressing cables to those of the preceding stage by means of couplers located at each construction joint. The space required to accommodate these couplers can, however, cause some problems when it comes to detailing, and the resulting congestion can make it difficult to achieve adequate compaction of the concrete in the vicinity of the construction joints.

- Another possible method, suitable for slab bridges, ribbed slabs or bridges with wide webs, is to apply the prestress in two stages. The first set of cables is stressed as soon as the span has been constructed, in order that the structure can carry its self weight and allow the falsework to be removed. The second set of cables overlaps the first, and is stressed as soon as the next span has been constructed (figure 4.2). The second set is usually anchored in recesses or pockets in the deck surface, which is acceptable only if adequate precautions are taken to ensure that no water can enter the ducts at these locations and is not generally recommended.

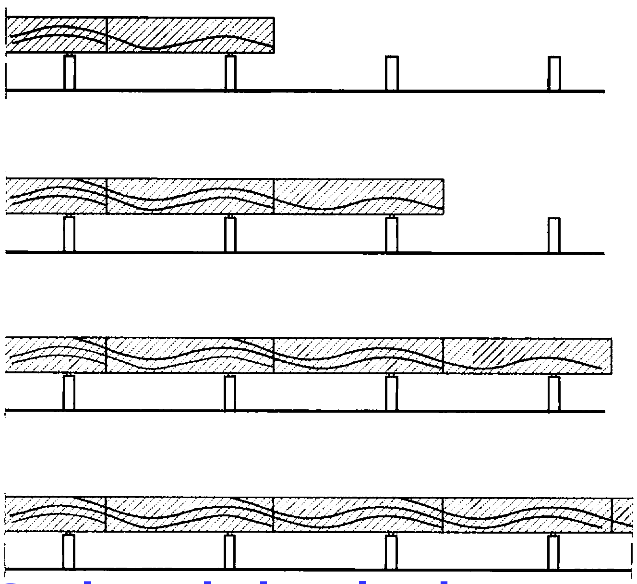


Figure 4.2: Prestressing in two stages for span by span construction

- The design of prestress is easier in the case of box-girders because it is possible to anchor the second set of cables at blisters located within the box section, which greatly reduces the risk of corrosion (figure 4.3).

- A further method is to stress one set of cables at the time of construction of the span, with a second set being provided locally over the piers and stressed as soon as the structure is complete.

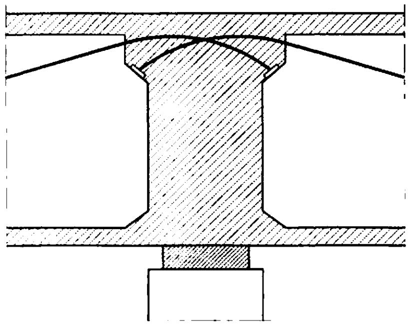


Figure 4.3: Anchoring of prestressed cables at blisters in a box girder

4.2 Construction on Launching Girders or Gantries

This method is applicable to the construction of beam decks, which are usually continuous and of constant depth. The launching girders and gantries are designed to support a span whilst it is being constructed and then to be moved forward to the next position once the span has been completed. Construction joints are usually positioned at approximately one fifth of the span from the pier

4.2.1 Conditions of Use

The use of launching girders or gantries can be considered for spans in the range of 30m to 60m, although they are most often used for spans of 40m to 50m.

They are expensive items of equipment to construct and operate, and it is necessary to consider depreciation, transport, assembly, dismantling and storage when assessing their economic viability. They can be considered only for long bridges with many spans, preferably of equal length. Their cost should be redeemed on the bridges for which they have been designed, and it should not be assumed that they may be of use on another project without the need of very expensive modifications.

Once in place, a launching girder or gantry should enable a rate of construction of one span per week to be achieved, particularly in the case of ribbed slabs.

4.2.1.1 Launching Gantry

A launching gantry may be used with many types of deck cross-section, although it is best suited to box-girder construction. There are no particular requirements for the shape of the pier head. The use of the gantry, however, necessitates the use of an intermediate bearing at each pier, requiring the construction of the deck at those congested and highly stressed locations to be completed at a later date (figure 4.4).

The system has the following advantages:

- no problems of clearance,
- facilitates the construction of a bridge which is curved on plan,
- the gantry may accommodate any handling equipment required to transport materials,
- has no influence on the shape of the pier head.

There are, however, a number of disadvantages, as follows:

- the need to suspend the formwork from hangers which have to pass through the deck, and which have to be moved out of the way of the piers when the gantry is launched to its next location,
- the deflection of the supporting structure for the formwork reduces the clearance beneath the structure,
- the heavily reinforced section of the deck over the piers has to be completed once the temporary support to the gantry has been removed.

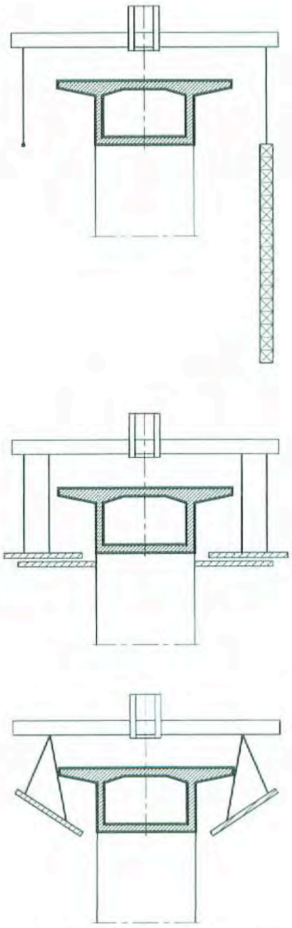


Figure 4.4: Construction sequences with a launching gantry

4.2.1.2 Launching Girder

The launching girder is particularly suited to the construction of ribbed slabs, and it causes fewer problems during the launching operations. The girders are usually supported on temporary bearings mounted on brackets at the piers, with one beam located beneath each side cantilever. The head of the pier often has a cut-away portion to allow passage of the central beams. Alternatively, these central beams may be moved transversely prior to launching.

Launching girders have the following advantages:

- the surface of the deck is entirely clear, thereby allowing a free working area and easy access to the handling equipment,
- there are no hangers to be removed prior to launching.

There are, however, the following disadvantages:

- due to its low position, a launching girder will generally be heavier than a launching gantry, although this will be partially offset by the fact that the formwork will contribute to the bending strength of the supporting members,
- temporary bearing brackets are required at the piers,
- not suitable for bridges which are curved in plan,
- cannot support handling equipment,
- can encounter problems of clearance beneath the structure.

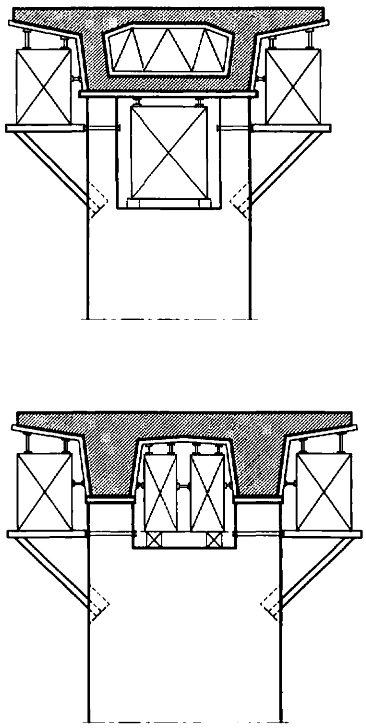


Figure 4.5: Construction sequences with a launching girder

4.2.2 Influence of the Tendons

The usual method is to connect the tendons in adjacent spans by means of couplers, as described in the previous section for bridges built on scaffolding or stationary falsework.

The main disadvantages of this system are the potential for a local weakening of the section at the construction joint and problems which can occur with the couplers. In addition, the efficiency of the prestress is limited at that section due to the effect of concentrated forces.

The layout of the prestressing tendons has to be modified as described for bridges built on scaffolding and stationary falsework.

4.2.3 Use of Precast Elements

The examples in Chapter 2 illustrate the advantages of using launching girders to support precast elements.

Launching gantries or launching girders may be used, either separately or in combination where an assembling girder is lifted into position from a gantry. Such a system is easily capable of achieving a rate of construction of one span per week, and rates of one span per day have already been recorded (figure 4.6).

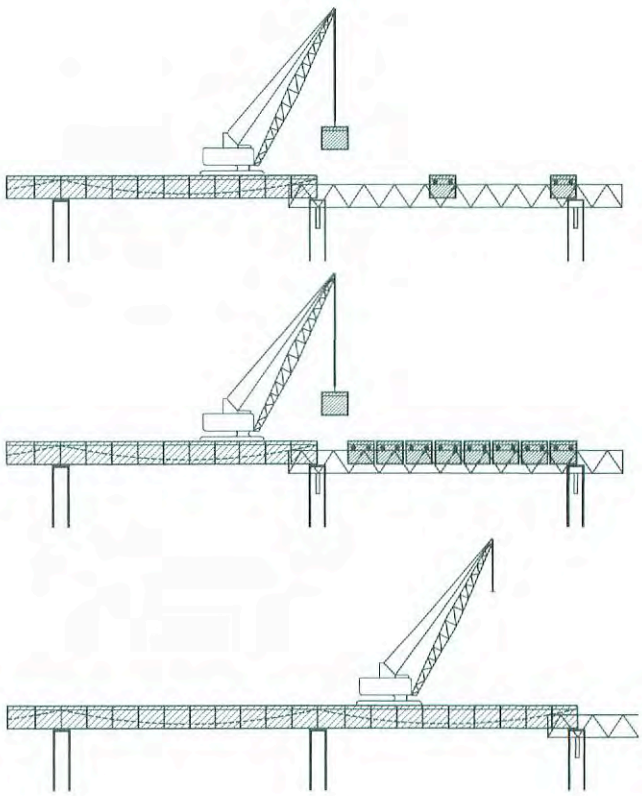


Figure 4.6: examples of using precast elements together with a launching equipment

4.3 Cantilevers

The most important types of bridge to be built by the cantilever method are those which, when complete, become continuous structures. These bridges are usually constructed from the piers, adding cantilevered segments, prestressed to their predecessors, which may be either precast or cast insitu on moveable formwork.

For balanced cantilevers, the length of the cantilevers will be approximately half the span length on either side of the pier.

4.3.1 Distribution of the Span Lengths

The final length of structure which is cantilevered from each pier is therefore equal to the sum of the half-spans on either side. It would obviously simplify construction if the end span were to be half the length of the adjacent span, but this configuration would result in uplift of the bearings at the end of the bridge due to the action of live loading. It is therefore necessary to increase the length of the end span by means of one or more elements either precast or placed on falsework.

If the deck has a variable depth (usually following a parabolic profile), then the end span should be 0.6 x the adjacent span to avoid uplift. This ratio should be increased to 0.7 if the deck is of constant depth. In some cases, however, it may be necessary (for geometrical, economical or architectural reasons) to provide an end span which is less than the optimum length. In these cases, it will be necessary to prevent uplift by one of the following means:

- by providing additional weight to the end span, or
- by anchoring the end of the deck to the abutment, or
- a combination of both.

For each solution the balance of the bridge has to be stable, in order to ensure that there is no reversal of the direction of the reaction on the abutment. Some bridges have, in the past, been designed in such a way that it was necessary to provide bearings which were capable of resisting vertical loads in either direction. This resulted in some very complicated details, and the bearings were soon found to be defective.

Figure 4.7: Nutagawa bridge before final keying – Japan, [24]

4.3.2 Typical Cross-sections and Ranges of Span

The decks of bridges built by the cantilever method are usually box-girders. Beam and slab bridges have been constructed this way in the past, but they have the following disadvantages:

- the geometry of the cross-section, with the neutral axis close to the top fibre, is not suitable for resisting the predominantly hogging bending moments,
- the sections do not possess the torsional rigidity of box-girders,
- there is very little saving of material, especially in the webs, compared to a box-girder,
- when in service, such bridges are much more difficult to inspect.

Decks may be of constant or variable depth. A constant depth is suitable for spans up to 60m or 70m, for which other forms of construction (such as launching or the use of precast beams) may be equally appropriate. For spans greater than 70m, a variable depth becomes more economic and more aesthetically pleasing.

It is possible to construct long span bridges with constant depth decks, but they require a considerable amount of prestressing.

Variable depth decks are economical for spans in the range of 70m to 250m, with the greatest span being 300 m long. The variable depth allows the sections to be proportioned with regard to the hogging bending moments which result from this method of construction.

There are several ways in which the depth of a deck may be varied, the most usual of which is the traditional parabolic profile. Some designers have achieved a pleasing appearance by providing a constant depth of deck for the central part of the span, with a linear increase of depth towards the piers. Other structures have been designed as constant depth for the majority of the span, with a parabolic or cubic variation in depth locally at the piers.

4.3.3 Bearing Systems

In most cases the decks transmit vertical loads only to the piers, usually by means of a single line of bearings. The behaviour is therefore similar to that of a continuous beam on simple supports. However, it may be possible for the piers to carry some of the bending moments due to live load which would otherwise be transmitted from one span to another. This would require the deck to be partially or totally restrained at the piers.

Partial restraint is often provided by means of two lines of laminated rubber bearings, for which the amount of restraint is proportional to the vertical load upon them. The effect of this partial restraint is to reduce the transmission of bending moments from one span to another, and therefore to reduce the required depth of construction at mid-span. The characteristics of the bearings, however, are known to vary with time, which will have the effect of reducing the certainty of the distribution of loading throughout the structure. It must also be remembered that the replacement of those bearings may be a most difficult operation.

Total restraint of the deck at the piers can be very beneficial, and should be considered whenever possible. The main advantages are that it provides a simple solution to the problem of stability of the cantilevers during erection of the segments and, by transferring a proportion of the bending moment to the piers, will reduce the bending moments within the span.

It will be necessary, however, for the piers to be sufficiently flexible in longitudinal bending, in order to avoid inducing stresses in the deck which would arise from the restraint of temperature and shrinkage. In addition, if the bending stiffness of the piers is too great, then the piers will resist the application of the continuity prestress and there will be insufficient compression in the deck.

It will also be important for the piers to possess adequate torsional rigidity, in order to prevent the cantilevers rotating due to wind action during erection. This requirement is based on the consideration of comfort of the workforce, as well as the need for accurate alignment when making the connection between the tips of the cantilevers. It has been found that piers comprising two parallel columns satisfy these requirements.

4.3.4 Considerations for the Design of the Cross-section

As stated previously, the cantilever method is most commonly used for the construction of box-girders. In recent years there has been a tendency towards the use of single cell box-girders, even for relatively wide structures. For these sections, the shear forces can be distributed equally between the two webs only when the bridge is straight. Single-cell box-girders have the advantage, however, of being easier to construct than multiple-cell box-girders and they require fewer bearings.

When designing these wide structures, it is necessary to give careful consideration to the effects of distortion and transverse bending. The effects of distortion can be reduced by the use of external continuity prestressing, because of the stiffening effect provided by the deviators which are required to deflect the tendons. With internal prestressing, the additional width of web required will offer some resistance to distortion, although it may also be necessary to provide diaphragms at quarter-span points in the case of long spans.

The consideration of transverse bending is most significant for the top slab of a box-section which has to be designed for the local effects of heavy wheel loads. The bottom slab may also be subject to some transverse bending, particularly in the case of a variable depth girder which incorporates a bottom flange which is curved in elevation, thereby creating a radial component of the longitudinal force in the slab. It is therefore necessary to ensure that the slabs are of adequate thickness, or to consider the use of transverse prestressing or transverse stiffening beams.

4.3.5 Considerations for the Design of the Prestress

The cantilever method of construction requires the use of two sets of prestressing tendons, viz. the cantilever tendons and the continuity tendons (figure 4.8). The cantilever tendons are required primarily to resist the effects arising from the construction stages, which constitute the greater part of the hogging bending moments at the piers. The continuity tendons are required to ensure adequate compression at the joint between the tips of the cantilevers, as well as to provide the necessary resistance to superimposed dead and live loading. The current tendency is to provide the cantilever tendons within the concrete, with the continuity tendons being partially internal and partially external to the section.

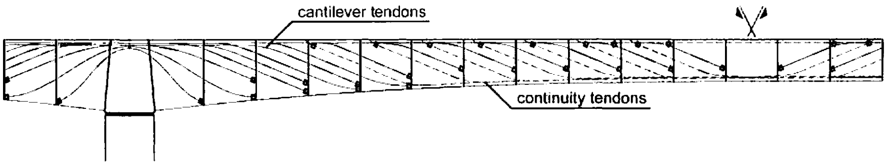


Figure 4.8: Double set of prestressing tendons for bridges built by the free cantilevering method

The contribution made by the external tendons will depend upon the preference of the designer. In some cases the decision will be made to include the minimum number of tendons only within the concrete section, in order that more resistance may be provided by the external tendons which can usually be stressed in a more controlled manner. In other cases, the external tendons will be designed to carry live loading only (figure 4.9).

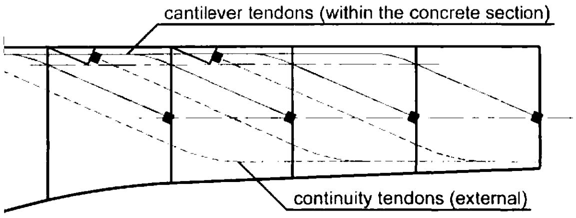


Figure 4.9: Splitting between internal and external tendons

When designing the bridge, it may be desirable to make provision for additional (empty) prestressing ducts to be installed. These would then be available to accommodate additional tendons in the event that, during construction, the stressing records indicated that insufficient prestress had been applied (e.g. due to exceptionally high friction in the ducts).

4.3.6 Design of the Pier Segments

The pier segments are the first to be placed in the cantilever method of construction. These segments are usually very heavy, have a complicated geometry and are very heavily reinforced. In most cases, it is not possible to position the bearings directly beneath the webs of these segments, due to their size. It is also not possible to apply the bearing loads or, at times of bearing replacement, the jacking loads directly to the bottom slab due to its limited capacity for transverse bending and shear. It is therefore necessary to provide a substantial diaphragm over the piers, which also serves to provide torsional rigidity and prevent distortion of the cross-section.

4.4 Incremental Launching

Launching is suitable for decks of constant depth only. Any variation of depth would be virtually impossible to accommodate at the positions of the roller bearings. It is also necessary for the alignment of the bridge to be straight, or on a constant circular curve (in any plane).

Detailed geometric studies have shown that a bridge may be launched, without inducing parasitic moments, provided that the soffit has the shape of a spiral line or is part of a frustum of a cone. It is therefore possible for a launched bridge to be curved both horizontally and vertically provided that, for a given highway alignment, the shape of the soffit is chosen to match, as near as possible, that of a frustum of a cone.

During construction, it will be necessary to provide sufficient working space behind the abutment to facilitate construction of the deck prior to launching. In most cases, this space should be long enough to accommodate between one and two spans, if the bridge is to be launched from one side.

Figure 4.10: General view of the "Sart" Canal Bridge during launching – Houdeng, Belgium, [2]

4.4.1 Choice of Deck Type

Incremental launching can be used for bridges of many types including slabs, ribbed slabs, single or multiple box-girders. There are references in the bibliography to each of these.

Wen launching slab decks, whether they be solid or voided, rectangular or with side cantilevers, the maximum span should normally be limited to 20m or 25m. They may be used for longer spans, e.g. when it is necessary to cross a live highway or railway, but they will generally be less economic that ribbed slabs.

Despite being easy to construct, ribbed slabs are no longer used for spans over 40m. These longer spans are usually constructed using box-girders, because of their high torsional stiffness and structurally efficient cross-sections.

Box-girders have therefore been found to be the most suitable sections for resisting the reversible effects encountered during construction, especially if launching is carried out from one side (figure 4.11).

Box-girders also offer the best solution with respect to ease of inspection and maintenance, as well as providing a suitable means of accommodating services.

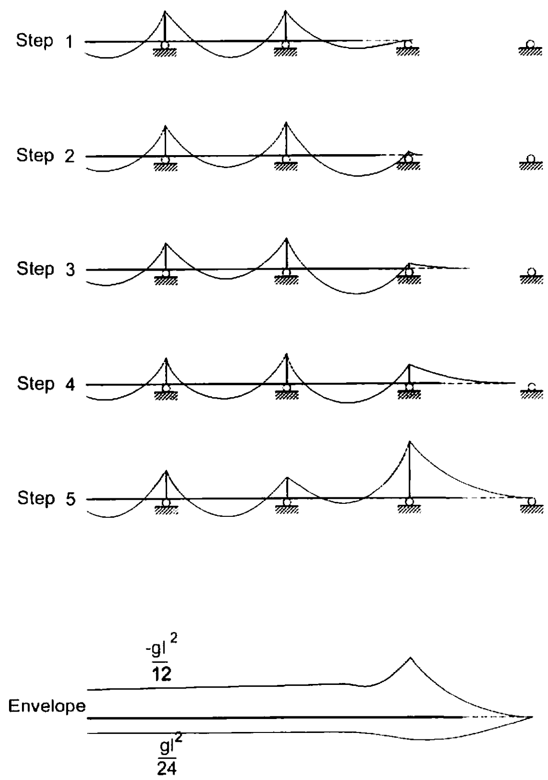


Figure 4.11: Bending moments envelope during launching process

4.4.2 Range of Use and Typical Spans

For bridges launched from one side, the method requires the construction of the deck, in successive segments of equal length, at a fixed construction facility located behind an abutment on the line of the bridge. As soon as the concrete in a new segment has attained sufficient strength, partial uniform prestress is applied and the entire structure is moved (launched) forward in order to free the working area for the construction of the next segment. The process is repeated as many times as necessary to complete the bridge.

For practical and economic reasons it is desirable for the spans to be as similar as possible and, preferably, not greater than 60m. The most usual length of span for launched bridges is between 35m and 65m, but spans of 80m are not unknown. It is possible, however, to launch decks with one or more spans considerably longer than the others, by making use of temporary supports. For example, the Our Viaduct (Steinebruck, Belgium) has a central span of 104.4m, with spans of 90m on either side and other spans of 64.55m, for which temporary supports were required at the centre of the three main spans. It is also possible to create long spans by launching the structure from both sides and by providing a central connecting section.

The use of temporary supports, however, can create serious technical problems if they need to have a significant height, and serious consideration must therefore be given to the matter of their horizontal deflection. The vertical settlement of the temporary supports can also be much greater than that of the permanent piers, in which case it will be necessary to take account of the resulting bending moments in the deck or compensate these settlements by means of hydraulics jacks.

It is usually necessary to apply uniform, temporary, prestress to the section, to enable it to resist the effects of the reversal of bending moments which is associated with the launching process. The importance and cost of this prestressing are more significant for the longer spans, for which the dead load is large in relation to the subsequent live loading to be supported.

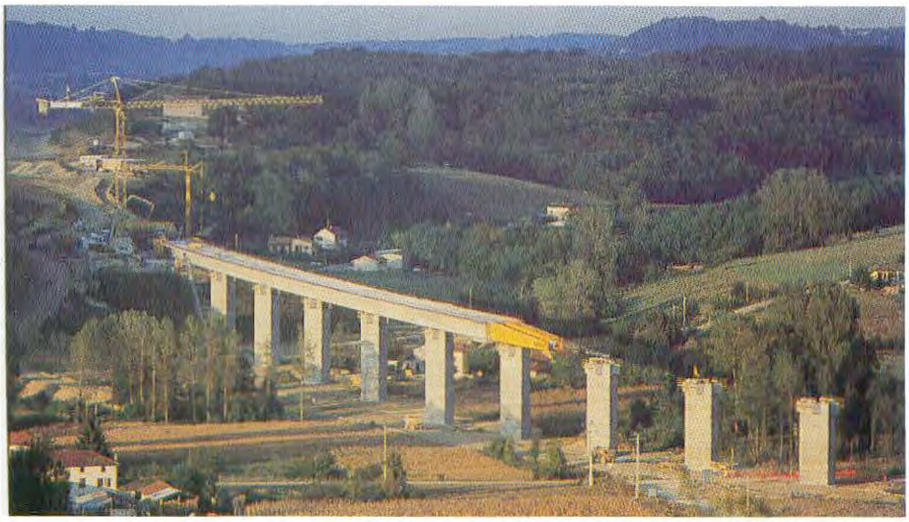


Figure 4.12: General view of the Meyssiez TGV viaduct during launching – France, [x]

4.4.3 Design of the Prestress

The profile of the tendons in a prestressed bridge can be relatively complex. The position of the tendons will vary between the upper part of the deck over the piers and the lower part at mid-span. For a launched bridge, however, each section of the deck will experience the effects at both mid-span and piers (and all intermediate locations) as it traverses the span during the launch. Each section is therefore subjected to a reversal of the sign of the bending moments and shear forces, for which the most appropriate prestress is uniform compression. This is achieved by means of prestressing tendons whose centroid is located at, or close to, the neutral axis of the section.

4.4.3.1 Traditional Design

In a traditional design, there are two main stages of prestressing.

- The first stage tendons have a straight profile and are located in the top and bottom slabs. They apply the uniform prestress, and are therefore called the launching tendons.
- The second stage tendons have a variable profile and are located within the webs. They apply the variable prestress to enable the structure to resist the applied service loading, and are therefore called the continuity tendons.

4.4.3.2 Use of external prestress

The development of external prestressing has, in recent bridges, had the effect of improving the tendon profiles and reducing the self-weight of the structures. The use of external prestress, however, varies considerably; some designers use it for the continuity tendons only, whilst others use only external tendons in their structures.

For recent bridges, the design of tendons in a span is based on the following principles:

- the launching tendons provide a prestress force acting as close as possible to the neutral axis of the section,
- upon completion of the structure, some of the launching tendons remain stressed and therefore contribute to the strength of the bridge, whilst others are de-tensioned in order to avoid unfavourable stresses at some locations,
- a variable number of the continuity tendons are provided externally to the concrete section.

4.4.4 Design of the Cross-section

Besides the slab and ribbed bridges mentioned above, the majority of launched bridges have a single box-girder cross-section. The width of the bottom slab (or, more precisely, the distance between the webs) for launched bridges should, ideally, be constant. It is possible, however, to accommodate some minor variation of this dimension, provided that the bearings on which the structure is launched can tolerate some lateral movement of the structure.

The magnitudes of the bending moments which occur during launching are such that it is usual to require a greater depth of construction than if the bridge had been constructed on falsework. Common span-to-depth ratios are 14 to 17 for highway bridges and 12 to 15 for railway structures.

The amount of ordinary reinforcement required is usually within the range of 130 to 150 kg/m^3, whilst the amount of prestress varies between 40 to 70 kg/m^3, dependent upon the type of loading to be carried (i.e. road or rail).

4.4.5 Advantages of the Method

Despite the specific requirements of the structure which are imposed by the launching process (i.e. the shape of the soffit, the need for a constant depth of construction, the requirements of the prestress and the desirability of equal spans), this method of construction has a number of advantages which justify its use.

- The establishment of a concreting facility at a fixed location, behind the abutment, allows the structure to be built in factory-like (covered and heated) conditions. There are opportunities to use rigid, mechanically-handled formwork, as well as to establish efficient systems for placing the reinforcement, concreting and prestressing. These all contribute to the achievement of a fast rate of construction, as well as improving the quality of the structure.
- Good control over the geometry of the structure.
- A reduction in the number of construction joints. All work carried out under factory-like conditions, enabling good quality concrete and construction joints to be achieved.
- A minimal amount of additional land required to accommodate the concreting facility, because it will usually be located within an area which will ultimately be required for the approach embankment.

4.5 Rotation

This method of erection is suitable for bridges of all types, because construction takes place on conventional scaffolding. The problems of the method, however, are related to the stability of the cantilevers during the rotation. It can be a satisfactory method of construction when the incremental cantilever method is too expensive, provided that the topography is suitable.

The method is used mainly for bridges having either two or three spans. It facilitates the construction of structures which have a significant architectural qualities without introducing excessive technical constraints, and is particularly suitable for the erection of cable-stayed bridges where most of the dead load is supported by the pylons.

The rotation method of construction does, however, impose some constraints on the design of a bridge. The first is the need to have sufficient space beneath the deck within which to provide a suitable scaffolding. The second is the need to ensure adequate stability of the cantilevered structure during rotation, for which the considerations will be similar to those of incremental cantilever construction.

The need to rotate a cantilever about the pier closest to the obstacle to be crossed is a disadvantage, because this necessitates the pier to be set-back from the obstacle by at least half the width of the deck, thereby increasing the span. The topographical conditions (such as the slope of a bank of a river) or the need to maintain the operation of a road or railway may increase the set-back even further. The construction conditions will also have an influence on the length of the main span of the bridge, particularly if the deck is wide.

It is, of course, possible to rotate the structure about a pier (or abutment) other than the one closest to the obstacle to be crossed. This, however, is much more difficult and expensive to achieve, because a significant proportion of the self-weight has to be carried on temporary supports during the rotation.

4.6 Precast Beams

Precast beams are most often used in simply-supported spans, usually supported on laminated rubber bearings. This configuration is very simple and is not sensitive to settlements of the bearings. If there are several spans, however, it can be economic to introduce some continuity, either by complementary post-tensioning or, more simply, by the provision of ordinary reinforcement.

Continuity is most usually achieved by means of ordinary reinforcement, using one of the following methods:

- to provide reinforcement within the slab only,
- to provide, over the pier, a length of insitu concrete deck which connects both the slab and beam components of the cross-section. This solution, however, has led to problems when the continuity has been achieved by means of ordinary reinforcement alone. Post-tensioning provides a more satisfactory connection, but is more difficult to achieve and is more expensive.

Connection of the spans by these means enables a continuous deck surface to be achieved, but the bridge will still behave as a series of simply-supported spans. The advantages are therefore related to the reduction in the number of movement joints required, an improvement in the comfort of the passengers and an increase in the life of the structure. The continuity slab must be sufficiently flexible to accommodate the end rotations of the beams, and must have adequate capacity to resist the punching effects of local wheel loads. As a result, the slab will usually be reinforced with a large number of small diameter bars.

4.6.1 Cross-section

Many different types of cross-section have been used for precast beams in the past, including inverted T-beams or double-T (TT) beams placed next to each other, with only a nominal joint between them, in order to provide a continuous deck slab. Other sections include trapezoidal shaped beams (wider at the top) and U-beams. T and TT beams have often been used, but they are relatively heavy and can be placed only with the use of high-capacity cranes. It can also be difficult to obtain a satisfactory continuous deck, because of the possibility of there being a difference in deflection (due possibly to the use of beams of different ages, concreting or storage conditions) between adjacent beams when delivered to site.

It is now more usual to use precast beams of a rectangular or I section, placed at 0.6m to 4.0m apart, which act compositely with an insitu concrete deck slab. The formwork for the slab usually consists of fibre-reinforced panels or thin concrete slabs.

Rectangular section beams, which are easy to construct and require only simple formwork, are usually economical for spans up to 15m. For longer spans it is more economical to use I-beams, although these will need to be of rectangular cross-section at their ends in order to accommodate the anchorages for the post-tensioned tendons. Pre-tensioned beams, however, may be of the same cross-section throughout their length.

The deck slab, which connects the precast beams and carries the live loading, is usually made of reinforced concrete. When the beams are relatively close, the formwork for the slab usually consists of sheets of fibre-reinforced cement. For spacings of beams up to 4.0m, the use of participating formwork comprising reinforced concrete slabs is often the most economical.

The effective thickness of the slab should not be less than l&m, and will generally not exceed 25cm. The choice of thickness will be require a compromise between the effect of the self-weight of the slab and its ability to distribute loads between the beams.

4.6.2 Depth of the Beams

The engineer has some freedom, when it comes to determining the number of beams and their spacing, to configure the bridge to suit the requirements of the project. Because these bridges are usually constructed with precast beams, the slenderness, defined as the ratio of the span length to the total depth of construction (beam and slab), has to be at the lower end of the scale defined in the previous chapter.

For beams which are 2m or 3m apart, a slenderness of 18 is typical. A values of 25 is possible for spans which have been made continuous, thereby allowing a reduced depth of construction when clearance is a major consideration. A slenderness of 15 will produce the most economic solution, although it is unlikely that the resulting structure would be aesthetically pleasing.

4.6.3 Prestressing

Precast beams may be pretensioned or post-tensioned, or a combination of both.

Pretensioning, where the prestressed stands are embedded directly into the structural concrete, provides better protection of the steel against corrosion than post-tensioning, where the tendons are located in ducts which are subsequently injected with grout.

If it is necessary to carry out second stage prestressing after the deck slab has been cast, then these tendons will often need to be anchored in recesses set into the top of the slab. This situation usually arises when it is not possible to access the ends of the beams for the purposes of anchoring the second stage tendons. The position of these anchorages makes them particularly vulnerable to corrosion and they should, whenever possible, be avoided. However, if they are unavoidable, special attention has to be given to the choice of system and the method of installation in order to ensure that the tendons are adequately protected.

4.6.4 Limits of Use

The use of precast beams facilitates the construction of wide bridges which require a great number of beams. The main disadvantage of their use is related to the form of the substructures which are required to accommodate their bearings. These are usually either solid leaf piers or portal frames comprising cross-head beams on a number of columns, neither of which are aesthetically pleasing. A better situation can, however, be created if the cross-head beam can be located within the depth of the superstructure, and the number of piers kept to a minimum.

Precast beams are not suitable for bridges which are curved on plan. Bridges which have been constructed by using a series of straight spans to try to follow a curved alignment, do not have a pleasing appearance and are suitable for use only in a poor quality industrial environment.

Precast beams should also be avoided if the vertical alignment requires the structure to be curved in elevation. In any case, the appearance of the outer I-beams is not particularly attractive, especially where it is necessary to thicken the section at the ends of the beams in the vicinity of the bearings. A common solution is to mask the outer beam by attaching full-depth precast panels to its edge, but this has the effect of making the beam appear to be particular heavy.

4.7 Transportation and Lifting of Heavy Loads

The need to reduce the time required for construction (of bridges which may be many kilometres in length) has, for a long time, been a major influence on the development of new methods of construction and on the design of new bridges.

The precasting of heavy elements has become the most appropriate solution, in order to satisfy the economic requirements and to achieve the required rate of construction. The construction methods are based on those developed within the

Japanese steel construction industry, using high-capacity floating cranes to place elements weighing several thousand tonnes.

The design of these bridges is governed by the following considerations.

- It is necessary to provide an adequate number of movement joints.
- With the increase in the capacity of the lifting equipment, it has become possible to construct spans of even greater length e.g. 250m for Prince Edward Island Bridge in Canada.
- In order to achieve a rapid rate of construction as well as maintaining the stability of the precast elements, placed one after another, it is beneficial to work on an entire span at one time.

For long viaducts, the following conventional Systems are used.

- Statically determinate structures, consisting of several beams built by the balanced cantilever method and connected at mid-span by simply-supported beams (Barheim Coastway).
- Statically determinate beams with a constant depth, placed directly on bearings and made continuous (Second Tagus Crossing).
- Several variable depth beams, placed and balanced on the piers until being made continuous by the provision of insitu connections at mid-span.
- Several 200m long variable depth beams, fully restrained at the piers, connected on one side by additional 50m long elements to form a portal, and by simply supported beams on the adjacent span (Prince Edward Island Bridge).

For most of these bridges, the substructures (foundations and piers) are also precast and placed with the same lifting equipment.

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