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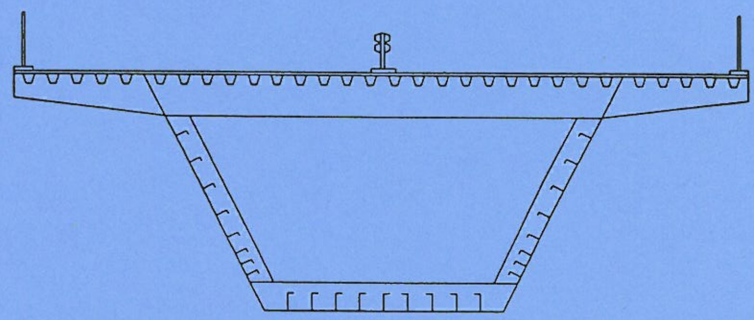

TEKSTHÆFTE

STORE STÅLBROER - Bjælkebroer samt skråstags- og hængebroer
ESDEP-UDDRAG v/A. ALBERTSEN
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Niveau A:	Grundkurser for ingeniørstuderende
Niveau B:	Kurser for studerende med speciale i stålkonstruktioner
Niveau C:	Efteruddannelseskurser

I dette kompendium tilhører afsnit 8.5.1 niveau A, medens alle øvrige tilhører niveau C.

Materialet foreligger indtil videre kun på engelsk.

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INDHOLDSFORTEGNELSE FOR KOMPENDIET *

NB: Figurerne til teksten i dette hæfte findes i FIGURHÆFTE, note U9408.

*) De afsnit fra kapitel 15B, der *ikke* er medtaget i dette uddrag, er:

15B.7 Arch Bridges
15B.11 Splices and Other Connections in Bridges
15B.12 Introduction to Bridge Construction

OVERSIGT OVER ESDEP-MATERIALETS INDHOLD:

CONTENTS:

ESDEP WG 15B
STRUCTURAL SYSTEMS: BRIDGES

Lecture 15B.1: Conceptual Choice

OBJECTIVE/SCOPE

To introduce the designer to the principal conceptual choices that have to be addressed for the design of successful, economic steel and composite bridges.

PREREQUISITES

None

RELATED LECTURES

Lecture 1B.6.1: Introduction to Design of Steel and Composite Bridges Part 1.
Lecture 1B.6.2: Introduction to Design of Steel and Composite Bridges Part 2.

SUMMARY

This lecture emphasises the importance of the correct conceptual design of bridges. After a brief introduction on the different types of bridge, it highlights the influences that the bridge function and other factors may have on the selection of correct structural form. It also addresses the more detailed choices that have to be made as the design is developed.

1. PRELIMINARY

"If you get the concept right, the design will be right". A trite statement, but one which contains a considerable element of truth (provided, of course, that the development of the concept is carried out correctly). If the concept is wrong, it will lead at best to a less than optimum design, or at worst to much abortive work or a design quite unsuited to its location. Conceptual design does not involve detailed calculations; indeed, in most circumstances, an experienced designer would probably be able to produce a safe and economic design from previous experience and would only use detailed calculations as a final check or for "fine tuning".

The purpose of this lecture is to give an inexperienced designer some guidance on the conceptual choices which have to be made. It is deliberately couched in general terms and makes no specific use of particular national or international standards for bridge design. In addition, many of the concepts described could be designed in either steel or concrete; in such cases, the emphasis is placed on steel construction.

2. FUNDAMENTAL BRIDGE FORMS

2.1 Introduction

Before attempting to describe how a designer approaches his task, it is useful to distinguish in very broad terms between a number of bridge types. (A more detailed catalogue of the types of bridges is given in Lecture 1B.6.1: Introduction to the Design of Steel and Composite Bridges Part 1. Their design is discussed in the remaining Lectures 15B.

2.2 Bridges which Carry Loads Mainly in Flexure

By far the majority of bridges are of this type. The loads are transferred to the bearings and piers and hence to the ground by slabs or beams acting in flexure, i.e. the bridges obtain their load-carrying resistance from the ability of the slabs and beams to resist bending moments and shear forces. Only for the very shortest spans is it possible to adopt a slab without any form of beam. This type of bridge will thus be referred to generally as a girder bridge. As can be seen in Lecture 1B.6.1, a wide range of structural forms is possible. Figure 1 indicates a typical elevation of a girder bridge with a number of terms being defined.

2.3 Bridges which Carry their Loads Mainly as Axial Forces

This type can be further subdivided into those bridges in which the primary axial forces are compressive (arches) and those in which these forces are tensile (suspension bridges and cable-stayed bridges). Such forces normally have to be resisted by members carrying forces of the opposite sense. Figures 2a to 2d show the basic structural systems for some typical layouts.

It must not be thought that flexure is immaterial in such structures. Certainly, in most suspension bridges, flexure of the stiffening girder (see Figure 2c) is not a primary loading in that overstress is unlikely to cause overall failure; however, in cable stayed bridges (particularly if the stays are widely spaced) flexure of the girder is a primary loading. Similarly, in arch bridges, non-uniform loading of the rib can cause primary bending moments to be developed in it and may well govern the arch design.

2.4 Truss Bridges

Truss bridges are not specific bridge forms in themselves - rather trusses are used to perform the functions of specific members in one of the types above. For example, a girder in flexure or an arch rib in axial compression may be designed as a truss rather than as a solid web plate girder. A truss used as a girder in flexure carries its bending moments by developing axial loads in its chords, and its shears by developing axial loads in its web members. Definitions can become somewhat blurred, e.g, a tied arch (see Figure 2b) can sometimes be considered

to act as a truss girder, particularly if the hangers are inclined to form a triangulated system.

3. THE PURPOSE AND FUNCTION OF A BRIDGE

3.1 Introduction

A bridge has to carry a service (which may be highway or railway traffic, a footpath, public utilities, etc.) over an obstacle (which may be another road or railway, a river, a valley, etc.) and to transfer the loads from the service to the foundations at ground level. The functional considerations that have greatest influence on conceptual choice are:

- The clearance requirements (both vertically and horizontally) and avoidance of impact
- The type and magnitude of the loading to be carried
- The topography and geology of the site

It is not possible to place these considerations in any particular order of importance; the relative importance is likely to vary from project to project and each must be considered on its merits.

3.2 Clearance Requirements

All bridges must be designed to ensure, as far as is possible, that they are not struck by vehicles, vessels or trains which may pass below them. This requirement is normally met by specifying minimum clearances. It must be remembered that designed values must take into account deflections due to any loading that may occur on the bridge structure. Clearance requirements may thus determine the span of a bridge and also have a significant bearing on the construction depth. Whilst the requirements will not normally determine precisely the type of bridge, it may well eliminate some possibilities.

Typically, for example, a bridge over a major highway would be expected to have a minimum vertical clearance of about 5,3 metres; even this may not protect it from accidental impact (e.g. cases have occurred of the jibs of cranes being moved on transporters becoming free and rising). In addition, pier positions must be such that the likelihood of impact from errant vehicles is minimised, both to protect the pier and the vehicle itself. This requirement is usually achieved by setting the pier back a reasonable distance from the edge of the carriageway.

Strict rules for vertical and lateral clearance over railways are laid down by all railway authorities, and must be complied with.

Navigation authorities specify clearances over rivers, to allow not only for the mast height and width of vessels below the bridge, but also for particular requirements for piers in the waterway (or on a flood plain) to avoid excessive flow velocity and scour of river banks.

In considering vertical clearance, a designer must bear in mind the problems of attaining them. The approach gradient for a highway bridge should not normally exceed about 4% and a railway bridge much less. This is of great importance when comparing fixed with moving bridges.

3.3 Loading

The type and magnitude of loading has a significant bearing on the form of bridge. Highway loading by its nature is impossible to determine exactly, either in disposition or in magnitude. For obvious reasons, a highway bridge requires a deck on which the traffic can run and (unless the span is so short that a simple slab is adequate to span between abutments) the deck must be strong enough to distribute the loading to the main girders. Traditionally, railway bridges were designed without full decks since the position of the load was determinate and the bridge could be constructed with rail tracks running directly on the girders. However, modern railway bridges, particularly in certain environments, have decks to support the ballast. The latter is necessary to give satisfactory noise reduction. A service bridge, e.g. a pipeline, may similarly dispense with a deck.

Every country has its own specification for the magnitude of loading on highway and railway bridges. Eventually, in the European Community, these specifications will be replaced by a standard European loading specification, but until then the national codes will continue to be used. For highway bridges most national codes have in common a uniform loading together with a line load (or series of point loads) to represent isolated heavy axles. In many codes, the uniform load is of decreasing intensity as the length of bridge increases, to allow for the reduced probability of a concentration of heavy lorries. Furthermore, there are rules for multiple lane loadings, frequently assuming that not more than two lanes are fully loaded at any one time, again based on a probabilistic approach. Many authorities also specify checks for a single very heavy abnormal vehicle. In many codes, the effect of impact (dynamic magnification) of highway loads is implicitly taken into account by the static load specification.

Railway loadings are more nearly deterministic since the loads of the heaviest trains are reasonably well known. However, most codes for railway loadings require an explicit calculation for the impact effect.

Additionally, forces arising from braking or acceleration of vehicles, centrifugal effects on curved bridges, temperature effects and wind have to be taken into account where relevant.

Whilst the details of applied loads are appropriate to the detailed, rather than the conceptual design of a bridge, certain aspects enter into the concept. For example, where heavy abnormal vehicles are specified, the bridge will require good transverse load distribution. This requirement may eliminate certain forms of construction. Temperature effects are significant for bearing layout and structural articulation, and wind loading plays a dominant part in the conceptual design of very long spans, even though it may be insignificant for short spans (except, possibly, for the foundations).

Lecture 15B.2 gives a detailed introduction to bridge loading.

3.4 The Topography and Geology of the Site

Sometimes this aspect alone determines the structural form. For example:

- The overall topography of the site will probably determine the line of the road or railway. Not infrequently this may mean that bridges will have to cross other roads, railways or rivers at a substantial angle, resulting in skew spans (Figure 3). The road may be on a curve; whilst it is possible to curve a bridge to follow this, it is frequently expensive and structurally inefficient, usually dictating the use of torsionally stiff girders even for short spans. If the curve is slight, it may be preferable to construct the bridge as a series of straight spans.

- Poor foundation conditions will favour fewer foundations and hence longer spans. A diagrammatic representation of the costs of a bridge is given in Figure 4. A balance has to be found between the cost of foundations and superstructure to minimise the total cost.

- Sometimes topography alone will point to a particular solution; the classic case is a deep, rocky sided gorge which is ideally suited to a fixed arch bridge (Figure 2a).

  Generally, the bridge site is fixed by the geometry of the obstacle and local terrain.

- However, where possible, it is well to consider the siting carefully. There is often the possibility to reduce the size of the span, to avoid placing the bridge on a curve, to reduce the angle of skew, or improve the construction depth.

4. OTHER FACTORS INFLUENCING CONCEPTUAL CHOICE

4.1 Introduction

In addition to considerations of the purpose and function of the bridge, there are several other important factors which can have a significant influence on the conceptual design of a bridge.

4.2 Methods of Erection

It has long been appreciated that a designer must consider at the design stage the method by which a bridge will be erected. Indeed it is not infrequently the case that such consideration should be made even at the time of conceptual choice, since it can happen that the superficially most attractive design is impossible to erect in a particular location. For example, a design that relies on being erected in large pieces (such as a major box girder), may be ruled out because of the impossibility of transporting such pieces to a remote site with inadequate access roads.

Frequently, particularly on large structures, it is possible to adjust the distribution of moment and forces in a structure by choosing a particular erection sequence. This possibility can affect the conceptual choice, e.g. the designer of a major three span estuarial crossing with a central span of 200m, may consider that the best conceptual choice is a steel box girder haunched at the internal piers thus carrying high hogging bending moments at these piers and comparatively low sagging moments at midspan. However, the most convenient erection method for this site may be to float out and lift most of the central span in one piece, thus causing high dead load sagging moments at midspan. By building the end supports high and jacking the ends of the bridge down after the connection of the mainspan is made, an overall hogging moment can be induced to counter the unwanted sagging moment. Certainly this solution requires very careful analysis, and calculation of fabrication and precambering dimensions to obtain the correct carriageway profile, but at least the concept will be right!

Many methods of erection of steel bridges exist; five typical ones are:

- Assembly in situ
- Launching
- Lifting
- Cantilevering
- Sliding

Combinations of these methods are possible.

4.2.1 Assembly in situ

This method involves assembling the bridge from its individual components or sub-assemblies in its final position, usually on falsework or some other form of temporary support, making the site splices and removing the falsework. Adequate cranage must be provided to cover the whole of the deck area. The presence of falsework may temporarily block a road, railway or river over which a bridge is built. Because large individual pieces are not normally involved, it is a method which may be practicable when access to a site is difficult. Assembly in situ may be used in conjunction with other methods of erection.

4.2.2 Launching

This method involves assembling a bridge on rollers or skates on its final alignment but at the side of the obstacle to be crossed. When complete it is pushed or pulled forward to cross the obstacle and land on bearings on the far side (Figure 5).

Whilst simple in principle, launching requires a site where large pieces of the bridge can be constructed in line with the final position but on the shore. The operation also requires very careful control and detailed analysis since, at various stages, bridge sections may be subjected to loadings differing greatly from those in service.

4.2.3 Lifting

This method involves lifting a self-supporting part or the whole of a bridge into or near to its final position (Figure 6). Pieces lifted can vary from a small footbridge weighing a few tonnes, to a large section of a major crossing weighing over 1000 tonnes. Lifting may be a complete operation in itself, or part of a cantilever erection scheme.

Lifting plant may range from small cranes for minor bridges, to very large floating cranes for major parts of estuarial bridges; alternatively winches or jacks on the already erected part of the bridge may be used. Hence the position and topography of the site will have a significant effect on the conceptual choice.

4.2.4 Cantilevering

This method involves constructing a bridge, normally continuous over several spans, progressively from one or both abutments, by attaching sections to the end of already erected portions (Figure 7). An anchor span is lifted or assembled in situ, and sections then cantilevered from this by either lifting from ground level, or running along the deck and lowering from the end. The position of the site and the access to it will determine the size of the pieces erected and this in turn will have a bearing on the original choice of the structural concept. Cantilevering is an ideal method for erecting cable-stayed bridges, using the stays as supports for the cantilever as work progresses.

4.2.5 Sliding

This method involves building the bridge offset laterally from the final site and then jacking it sideways into its final position. It is typically used for replacing an existing bridge which cannot be taken out of service for a long period. For obvious reasons it is only possible to use it for a very strictly limited type of site.

4.3 Local Constructional Skills and Materials

It should go without saying that a bridge should be suited to local technology. It is not sensible to specify a sophisticated design in welded high tensile steel if all the material and labour has to be imported. This consideration applies not only to minor bridges - the new 460 m span Hooghly Bridge in Calcutta, under construction in 1992, was designed as a rivetted cable stayed bridge; rivetting skills were still available in India and local steels could have given problems with site welding; the design proved an economic solution whereas in Europe there are probably no riveters still in existence and weldable steel is the norm.

4.4 Future Inspection and Maintenance

The importance of future inspection and maintenance cannot be overemphasised, both at the conceptual design and the detailed design stages. There is little doubt that too little attention has been given to it in the past, with the result that many bridges, otherwise satisfactory, have deteriorated because of difficulty in inspection and maintenance. It is particularly important that in locations where access is difficult (either physically or because it would cause disruption of services) details which deteriorate should be avoided as far as possible. This will be considered further in various respects, for example whether a bridge should be a series of simple spans or should be continuous.

At the conceptual design stage, a designer should consider whether it would be appropriate to use a material such as weather resistant steel or perhaps whether the structure should be fully enclosed with a non-maintenance material to protect it and give access for inspection. Either of these expedients could result, for example, in a requirement for greater clearance of the bridge above the obstacle being crossed. They might have to be considered together with the layout of the overall scheme. For example, the topography might show that a minor change of alignment could accommodate the greater clearance at little or no extra expense.

4.5 Aesthetic and Environmental Aspects

The appearance of bridges has in recent years become a matter of considerable importance. Frequently, a scheme takes a road or railway through an area of great natural beauty and it is important that any structures are in keeping with these surroundings and do not adversely affect them. Many authorities consider that there are some bridges which actually contribute to the environment by giving an interesting focus to an otherwise empty scene; the Severn Bridge is a typical example. Regrettably, however, there are many bridges for which the contrary is true. Not infrequently, the problem could have been avoided by following some simple rules.

For example, it is commonly accepted that a bridge is more aesthetically pleasing with an odd number of spans than an even number. In addition, a degree of deepening at piers can add to the attraction. Figure 8 shows a typical location for a highway overbridge, with a number of possible solutions, any of which is normally structurally viable. There is little doubt that the 3-span structures are more attractive than the two span ones. Hence, unless there are other contra-indications, the conceptual choice should probably tend towards a 3-span solution.

In Section 3.3 it was pointed out that on a railway bridge, it is possible to mount the rails directly on the main girders. Unfortunately this arrangement gives rise to a high level of noise emission and so would not normally be acceptable, particularly in an urban area. Provision of a concrete deck slab together with the use of ballast and possibly elastomeric rail mountings can produce a dramatic improvement and shows where an "obvious" choice can be modified by environmental considerations.

5. DETAILED CONSIDERATIONS - GIRDER BRIDGES

5.1 Introduction

In Sections 3 and 4 consideration was given to a number of general aspects which have to be taken into account by a designer in making a conceptual choice. In the current section, attention will be focused on some more detailed matters as an introduction to the particular forms of construction which are covered in other lectures. As the large majority of bridges are the simple girder type, attention will be concentrated on these.

5.2 The Deck

A typical cross-section of a short span highway bridge is shown in Figure 9. For obvious reasons such a bridge requires a deck on which the traffic can run. The deck must be strong enough to distribute the local loads to the main girders. For such multi-girder structures there is little conceptual choice for a designer - it has been found by experience that a reinforced concentre slab between about 200 and 300mm thick, supported at about 3 - 3,5m centres is suitable for most purposes. For large span structures, twin girder solutions become more attractive and either a thicker deck, probably with varying depth, is required or cross girders have to be introduced. Only in bridges where weight is at a real premium (e.g. long span bridges, or moving bridges) is it normally necessary to think further than this.

One possibility for reducing the deck weight is the use of lightweight concrete (an example is the 174m main span Friarton Bridge in UK where the deck has been constructed as a lightweight reinforced concrete slab). However, a more normal alternative to an RC slab is an orthotropically stiffened steel plate deck. Many layouts have been tried, some of which have suffered from premature fatigue from the repeated stresses from traffic. There now seems to be general agreement within Europe that the cross-section shown in Figure 10 is the "state of the art" solution for a steel deck in 1992.

Finally, it must be emphasised that in modern designs the deck, whether of reinforced concrete or stiffened steel plating, will invariably be connected to the girders below it so that it acts compositely with them in carrying the bending moments imposed on them. In the case of concrete decks this connection will be made using shear connectors (see Figure 9) and in the case of steel decks by a direct connection (normally welding or high strength friction grip bolting).

5.3 Typical Layouts of Short and Medium Span Bridges

Figures 8, 11 and 12 show a number of typical layouts for bridges of this type with indications of dimensions. Comment has already been made in Section 4.5 on the aesthetic aspects. In the present section some technical questions regarding the alternatives are addressed. For example:

- What are the relative merits of making the slab span transversely between the main girders (Figures 11 a, b, c, d and f) or making it span longitudinally between transverse girders spanning between the longitudinal girders (Figure 11 e)?

- Should the slab provide the sole distribution medium to transfer the traffic loads from the roadway surface to the main longitudinal girders, or will additional transverse girders and/or transverse bracing be used?

- Should the slab be of constant or variable depth?

- Should the main longitudinal girders be fabricated from rolled sections or made up (e.g. plate girders or box girders)?

- Should the main girders be designed as compact or non-compact?

- Should the main girders be of constant or variable depth?

- Should the main girders be continuous over the piers or not? If continuous, into how many spans should the crossing be divided?

- What is the likely depth of the main girders?

- Is there any merit in using construction other than simply supported girders, e.g. portal frame.

- How are environmental loads, e.g. wind, temperature, catered for?

The bridges shown on Figures 5, 8, 11 and 12 are all real structures; Figure 11, in particular, identifies six types of cross-section for highway bridges, all of which have been used successfully. In the end, the test at the detailed design stage is which layout is the most economic for a particular site. To resolve this question may require a significant amount of trial and error calculation. It is useful, however, to lay down a few guidelines at the conceptual design stage (not necessarily in the same order as the questions above!):

- If a slab is made to span transversely between main longitudinal girders, the girder spacing is limited to 3 - 3,5m (unless a thicker slab is used; such slabs are likely to be of variable depth). Hence if the carriageway is wide, particularly in the case of long spans, an uneconomically large number of main girders may be required. On the other hand, widely spaced main girders require the use of transverse girders which do not contribute to carrying the longitudinal bending moments. Hence for narrow carriageways, particularly on short spans, this arrangement is unlikely to be economic.

- When a concrete slab spans between transverse girders the local bending effects cause stresses in the same direction as the overall bending stresses. Hence the effects are frequently additive and the slab has to be designed to allow for this.

- Steel plate decks will almost always be designed to span between transverse girders since, if they form part of the overall compression flange of the main girders, they will in any case require longitudinal stiffening.

- An alternative to transverse girders, to reduce the number of longitudinal girders but still allow the slab to span transversely, is shown on Figure 11d, where a longitudinal small section stringer supports the slab and is supported in turn from the main girders.

- Normally in the sort of spans where multiple longitudinal girders with a transversely spanning slab is the economic solution, it will be found that the slab will be adequate for distributing longitudinal bending moments among the main girders without added bracing (such bracing may, however, be required during erection to stabilise the system).

- Generally a constant depth slab is much cheaper than a haunched one, but is normally limited to a span of about 3,5m between supporting beams.

- Rolled sections are significantly cheaper per tonne than fabricated sections. However, they are of a limited depth so the maximum span for which they can be used is limited.

- Fabricated sections can be made more "efficient" structurally than rolled, since the material can be concentrated where it is most needed, i.e. in the flanges, and more particularly, in the lower flange when the deck forms a significant part of the upper flange. Hence for the same span they can be made lighter but this advantage may be offset by higher unit fabrication costs, see Section 5.5.

- Rolled sections are almost invariably Class 1. Hence advantage can be taken of designing them to allow for the full plastic moment of resistance in calculating their strength.

- Fabricated sections designed as compact are seldom economic.

- The conclusion is that rolled sections are usually economic for spans up to about 25m if simply supported and 30m or so if continuous. Plate girders and box girders can be used for spans up to 300m.

- Box girder construction for short span bridges is not normally economic unless it is necessary for a specific purpose, e.g. where high torsional rigidity is necessary, such as in curved bridges.

- Variable depth longitudinal girders are more expensive per tonne than constant depth ones, but can offer significant weight savings in continuous spans and frequently are aesthetically more pleasing.

- There are arguments both for and against continuity in short to medium spans. Some of these are listed below:

	For		Against
1.	Items difficult to maintain, such as expansion joints, can be minimised.	1.	Compression in bottom flange near piers; hence potential stability problems.
2.	Advantages of reduced depth of construction	2.	Composite sections much more efficient in sagging than in hogging.
3.	May be essential for bridge erection and/or launching.	3.	Hyperstatic structure - indeterminate - problems with differential settlement, concrete shrinkage and temperature gradient.
4.	The reduction in maximum moments during concrete placing is useful.		

Whilst other forms of construction, e.g. cable stayed, have been used for short span bridges in the past, they are normally adopted only in cases where special conditions govern (e.g. moving bridges, severe restrictions on headroom, etc.) or where the undeniable aesthetic attraction of such bridges is an important consideration. Footbridges frequently fall into the latter category.

5.4 Long Span Girder Bridges

Long span girder bridges are normally developments of the plate or box girder forms described in the previous section. They will usually be continuous over two or more spans and will frequently be haunched. Normally the span limit is about 250 metres clear (although longer examples exist, e.g. Rio Niteroi). A typical elevation of such a girder bridge is shown in Figure 13.

As with shorter spans, consideration has to be given to the cross-section (number of main girders, etc.) and the form of deck - normal reinforced concrete, lightweight reinforced concrete, orthotropically stiffened steel plate, etc. A very typical cross-section is the twin box girder with transverse girders as shown in Figure 14, although when the carriageway is comparatively narrow a single large box girder, frequently with an orthotropic steel deck, is quite common (Figure 15).

A form of construction used frequently in the USA and Canada, although not common in Europe, is the "open-top" composite box girder in which the reinforced concrete slab, placed in-situ, forms the complete top flange (Figure 16). The main problem with this form occurs during construction, when the tops of the webs need stabilising until the slab is placed.

5.5 Minimum Cost or Minimum Weight?

Any modern bridge designer must recognise that the ratio of material to labour costs has changed considerably within the last few decades. Depending on local conditions, 1 man hour now costs the same as 30 to 70kg steel. In the past, material costs were relatively greater and detailed designs close to practical minimium weight were also likely to be minimum cost designs.

In current conditions, it is frequently the case that the most economic design is one where the labour content of the fabrication has been minimised by careful design, where necessary at the expense of material weight. Figure 17 shows two examples where modern economic design is considerably simpler than earlier detailed designs. In the plate girder example shown, the stiffened girder would be 230kg lighter if it was 10m long but the seven stiffeners would each take at least ¾ hour to position and weld, a total time of 5¼ hours. Thus the heavier girder is cheaper in all fabrication shops where a man hour costs more than 230 ÷ 5¼ = 44kg of steel.

5.6 Design For Construction

Design for minimum fabrication can influence directly the choice of structural form. Table 1 indicates the contributions to total cost of these different types of bridge girder, taking the total cost of a plate girder construction as 100 units.

Bridge Type	Rolled Section	Plate Girder	Box Girder
Steel	30	30	30
Fabrication	30	30	70
Corrosion Protection	10	10	15
Erection	10	10	15
TOTAL	80	100	130

The influence of fabrication content is readily seen. Clearly a rolled section bridge can still be the cheapest solution even if it is significantly heavier than a box girder alternative.

The detailed economic comparison will vary considerably with local conditions but local fabricators will generally be only too ready to advise on the relative economics of different forms of construction.

Fabrication can be considerably facilitated by detailing which maximises repetition and also makes the bridge easier to construct. For example, incorporation of bolted joints for some connections introduces a tolerance to the structure which is not available in all-welded construction. Carefully positioned, they may minimise the use of expensive full penetration butt welds and may also reduce the risk of lamellar tearing. Once again, fabricators are usually pleased to advise designers on such matters.

6. CONCLUDING REMARKS

The range of types of bridge and methods of erection open to a designer is huge and sometimes daunting. There is no doubt that the hardest task that an inexperienced engineer will have to face is that of the best conceptual choice from among the available alternatives. This lecture has attempted, in outline, to give some guidance on which type is appropriate for a particular purpose and site, but it has to be pointed out that that often several will have to be tried to decide the best. Sometimes, indeed, it may be necessary to obtain tenders for two or more alternatives, since they will be so close that the designer cannot, with confidence, decide between them but has to rely on a contractor do so. Indeed, different contractors may put the schemes in different orders of cost.

At least an understanding of the basics should avoid the problems of trying to build a totally unsuitable design.

7. CONCLUDING SUMMARY

- If the concept is right, the design will be right' is at least as true for bridges as it is for other types of structure.

- Initial conceptual choice should take account of:

  - clearance requirements and the avoidance of impact damage
  - type of loading
  - topography and geology of the site
  - possible erection methods
  - local skills and materials
  - future inspection and maintenance
  - aesthetic and environmental aspects

- The design development needs to make the correct choices for:

  - deck structure
  - layout i.e. spans and structural arrangements
  - continuous or simple construction
  - proportions, i.e. span/depth ratios
  - reducing fabrication labour to a minimum
  - design for ease of construction

8. ADDITIONAL READING

1. B.H.V. Topping (ed) Developments in Structural Engineering. Proc Forth Rail Bridge Centenary Conference 1990, Spon, London.

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ESDEP WG 15B
STRUCTURAL SYSTEMS: BRIDGES

Lecture 15B.2: Actions on Bridges

OBJECTIVE/SCOPE

To identify the principal actions on bridge structures and to describe how they are considered in design.

PREREQUISITES

None.

RELATED LECTURES

Lecture 1B.2:	Design Philosophies
Lecture 1B.3:	Limit State Design Philosophy and Partial Safety Factors
Lecture 1B.4:	Background to Loadings

All other Lectures 15B.

SUMMARY

This lecture begins by explaining the reasons why actions on bridge structures are considered with considerable precision. It identifies the principal actions for highway and railway bridges. It discusses the means by which considerations of individual actions are combined to ensure that an economic design of adequate reliability is achieved.

1. INTRODUCTION

It is customary to devote considerably greater attention to the assessment of loads for bridges than for many other types of structure. There are several reasons for this greater precision:

- Bridges, particularly larger structures, are substantial investments of public funding for which a high level of safety is required.
- Loads may be determined with greater precision than with many other types of structure.
- Load paths are usually well defined - some bridge structures are effectively iso-static.
- Strength, static or fatigue, is more frequently the governing design condition.
- The primary structure is a much higher proportion (typically > 80%) of the total investment than is the case in, for example, a commercial multi-storey building (frequently < 20%).

In the future the Eurocode for the Basis of Design and Actions on Structures [1], will identify the loads and other actions which need to be considered in bridge design, and define their characteristic values. Parts 2 of Eurocode 3[2] and Eurocode 4[3] will provide detailed guidance for the design of bridges and plated structures. The combinations of actions which need to be considered, will be defined in these documents and appropriate partial safety factors will be presented [4].

None of these documents will be available for some time. This lecture is written in general terms. Representative examples of practice in defining actions on bridges are provided to illustrate general principles and also to provide indications of orders of magnitude.

2. HIGHWAY DESIGN LOADINGS

2.1 Dead Load

Dead load on bridges includes the weight of structural materials (self-weight) and also the so-called superimposed dead load (surfacing, finishes, etc.), Figure 1.

The weight of the surfacing generally has a large variation during the life of a bridge and so particular care must be taken to assess its design value. It is customary to adopt a conservative estimate of initial thickness to determine the characteristic loading and then to apply a high partial factor.

2.2 Traffic Loads

Traffic loads on bridge decks are used to simulate the effects of vehicles and/or pedestrian loads. Some traffic loads represent the weight of real vehicles that can travel over the bridges; other values and distributions are chosen in such a way that they produce maximum internal forces in bridge structures similar to the ones produced by real vehicles.

Four types of loads are specified in the European national codes:

- Uniform distributed loads
- Knife-edge load
- Single wheel loads
- Truck load

Figure 2 shows the combination of knife-edge load and uniformly distributed load on a 3-span structure which gives the greatest value of mid-span moment.

Impact effects (dynamic effects) of traffic loads are in general specified in the codes. For highway bridges an enhancement of up to 25% of the static load is often used to take impact into account.

a) Uniform distributed load (Figure 3).

This load simulates the effects of normal permitted vehicles. In some national codes its value is constant and independent of the loaded area. In other codes the load value decreases with the area occupied by the load, see for example Figure 4. Distributed load is applied on the traffic lanes and over the lengths that give the extreme values of the stress resultant (or internal force) being considered. It may be continuous or discontinuous.

b) Knife edge load

This load (Figure 5) is usually associated with the uniform distributed load. It does not represent a single axle load, but is a device to ensure that, together with the uniform distributed load, the vertical shear and the

longitudinal moments that may occur in real bridge elements are produced.

c) Single wheel load

Some national codes specify the application of a single heavy wheel load placed anywhere on the carriageway, with a circular or rectangular contact area (Figure 6).

d) Truck load

This load is intended to represent the extreme effects of a single heavy vehicle. In some countries it consists of a specified number of wheel loads and arrangements, Figure 7. Other codes indicate only the distances between axles, the spacing of wheels in each axle, and the minimum number of axles.

2.3 Longitudinal Tractive Forces

These forces (Figure 8) result from the traction or braking of vehicles and they are applied to the road surface, parallel to the traffic lanes.

2.4 Centrifugal Forces

Curved bridges are subject to centrifugal forces applied by the vehicles that travel on them. These forces are related to the traffic loads by a coefficient, α, whose value depends on the radius of the curve, R, and on the design speed, v by:

$\alpha = CV^2/r$

where

C is a constant

Some codes consider a uniform distributed radial load and others divide it into concentrated loads, Figure 9.

2.5 Sidewalks and Parapets

Many highway bridges, in urban and non-urban areas, have sidewalks (footpaths) for pedestrian traffic and/or cycle tracks. On these areas a uniform distributed load is usually considered, Figure 10. Some codes indicate also that one wheel load applied on the sidewalks should be considered.

Parapets of footpaths and cycle tracks that are protected from highway traffic by an effective barrier are designed to resist horizontal distributed force applied at a height of 1m above the footway. The nominal value of this force is about 1,5 kN/m, Figure 11.

When footways and cycle tracks are not separated from the highway traffic by an effective barrier, design loads have to recognise the need to contain traffic in the case of an accident. These loads are considerably higher and include an alternative concentrated load.

3. RAILWAY DESIGN LOADINGS

3.1 Dead Load

Superimposed dead loads on railway bridges usually include the rails, the sleepers, the ballast (or any other mean for transmission of train loads to the structural elements), and the drainage system, Figure 12.

3.2 Train Loads

Typical train loads on bridges consist of a number of concentrated loads preceded and followed by a uniformly distributed load. Both loads are equally divided between the two rails, Figure 13.

Some national codes specify the criteria to be used for the distribution of concentrated loads by the adjacent sleepers and the dispersal through the ballast onto the supporting structure, Figure 14.

3.3 Dynamic Effects (Impact)

Train loads specified in the codes are equivalent static loadings and should be multiplied by appropriate dynamic factors to allow for impact, oscillation and other dynamic effects including those caused by track and wheel irregularities.

Values of dynamic factors depend on the type of deck (with ballast or open-deck) and on the vertical stiffness of the member being analyzed. For open-deck bridges values of dynamic factors are higher than for those with ballasted decks.

Consideration of the vertical stiffness is made by adopting formulae in which the dynamic factor is a function of the length, L, of the influence line for deflection of the element under consideration. Some codes use different formulae for impact factors associated with bending moment and vertical shear.

3.4 Longitudinal Tractive Forces

These forces, which are a percentage of train loads, are considered as acting at rail level in a direction parallel to the tracks, Figure 15.

3.5 Centrifugal Forces

The nominal centrifugal load is applied corresponding to with the train loads and acts radially at a height of 1,8m above rail level. Its value is obtained by multiplying the train loads by a coefficient, $\alpha = cv^2/r$ (as in 2.4) which is proportional to the square of the greatest speed, v, envisaged on the curve and increases with the inverse of the radius, r, of curvature.

3.6 Lateral Forces From Loads

A single nominal load, acting horizontally in either direction at right angles to the track at the top of the rail is taken to provide for the lateral effect of the nosing of equipment, such as locomotives, Figure 16.

This force, usually with a value of about 100 kN, should be applied at a point in the span to produce the maximum effect in the element under consideration.

The vertical effects of the horizontal force on secondary elements such as rail bearers should be considered.

4. OTHER LOADS ON BRIDGES

4.1 Wind Loads

The wind actions on a bridge depend on site conditions and geometrical characteristics of the bridge. The maximum pressures are due to gusts that cause local and transient fluctuations about the mean wind pressure. Design gust pressures are derived from the design wind speed defined for a specified return period.

A reduced wind pressure can be used in erection calculations to allow for the much shorter period during which the relevant structure is at risk.

The design wind load, normally taken as horizontal, and acting at the centroids of the exposed areas is given by

$$P_t = \frac{1}{2}\rho v_c^2 A_t C_D$$

or

$$P_t = q A_t C_D$$

where:

ρ is the density of the air (1,226 kg/m^3 under normal conditions)
V_c is the design gust speed
A_t is the solid area in normal projected elevation
C_D is the drag coefficient (defined in the codes)

$q = \frac{1}{2}\rho v_c^2$ is the dynamic pressure

The area A_t should include the exposed area of the traffic where appropriate. Drag coefficients should be considered accordingly.

Exposed area of traffic on bridges has the length corresponding to the maximum effects and in general a height of 2,50m above the carriageway in highway bridges and 3,70m above rail level in railway bridges, (Figure 17).

4.2 Thermal Effects on Bridge Structures

Daily and seasonal fluctuations in air temperature cause two types of thermal actions on bridge structures, as referred in the codes:

a) Changes in the overall temperature of the bridge (uniform thermal actions)

b) Differences in temperature (differential thermal actions) through the depth of the superstructure, Figure 18.

The coefficient of thermal expansion for steel structures may be taken as 12×10^{-6}°C

These two types of thermal effect produce different types of response in a bridge. The overall change in temperature causes overall changes in bridge dimensions in an unrestrained structure (or so-called thermal stresses if these potential changes in dimension are resisted by the supports). Usually the structure is allowed to expand with minimal restraint by the provision of expansion joints and sliding bearings.

The non-linear temperature distribution in Figure 18 lead to self-equilibrating stresses on all cross-sections, even in an unrestrained structure. One way to understand these stresses is to consider the structure initially to be fully restrained and subjected to these non-linear temperatures, as shown in Figure 19(i). It is then progressively released to give the final stress distribution of Figure 19(i) d). It is noteworthy that the releases have led to both axial strain and curvature.

If the structure is simply supported on sliding bearings, it is free to expand and curve as shown in Figure 19(i)c). However, a continuous bridge will not be able to curve freely and will develop secondary "thermal" moments, M_t, and reactions shown in Figure 19(ii)b).

4.3 Shrinkage of Concrete

In composite girders the effect of concrete shrinkage should be considered.

In principle the shrinkage gives a stress independent of the strain in the concrete. It is therefore equivalent to the effect of a differential temperature between concrete and steel.

Generally, shrinkage effects are only taken into account when the effect is additive to the other action effects.

4.4 Settlements of Foundations

The settlements of foundations determined by geotechnical calculations should be taken into account during design of the superstructure.

For continuous beams the decisive settlements are differential vertical settlements and rotations about an axis parallel to the bridge axis.

For earth anchored bridges (arch bridges, frame bridges and suspension bridges) horizontal settlements have to be considered.

Where larger settlements are to be expected it may be necessary to design the bearings so that adjustments can be made, e.g. by lifting the bridge superstructure

on jacks and inserting shims. In such a case the calculations should indicate when adjustments have to be made.

4.5 Earthquake Actions

In European countries located in seismic areas, i.e. in southern Europe, earthquake actions should be considered in bridge design.

The behaviour of a structure during an earthquake depends on its dynamic behaviour, namely its natural vibration modes and frequencies, and damping coefficients.

When the bridge has a simple dynamic behaviour, for instance when the first vibration frequency is much lower than the other ones, the seismic action may be reduced to an equivalent static force, see Figure 20.

4.6 Forces due to Water Currents or Ice

All piers and other portions of the bridge should be designed to resist the forces induced by flowing water, floating ice or drift.

4.7 Collisions

In structures where essential load-carrying elements may be subject to impact by vehicles, ships or aircraft, the consequences should be considered as accidental load cases - unless the risk of such collisions is evaluated as being so small that it can be neglected.

It is necessary in many cases to allow partial destruction or damage of the element which is directly hit. This element then has to be repaired after the collision. It should, however, be shown that the partial destruction of a single element will not lead to a total collapse of the entire structure.

To reduce the consequences of collisions it may be necessary to limit the movements of movable bearings so that only the movements due to temperature effects can take place without restraint.

4.8 Friction in Bearings

It should be checked whether the unavoidable friction in bearings can induce forces or moments that have to be considered in the design of the structural elements.

Modern sliding bearings are characterized by a coefficient of friction of approximately 0,03 if the sliding surfaces are absolutely clean. However, to take into consideration some deterioration in the sliding surfaces as well as tolerances in the positioning of the bearings it is recommended that the design is based on a coefficient of friction of 0,05.

In a continuous beam with a fixed bearing at the centre and longitudinally movable bearings on either side, expansion (or contraction) of the beam induces symmetrical frictional forces. These forces are in horizontal equilibrium if a constant coefficient of friction is assumed, and they normally result in moderate axial forces in the main girders. However, to take into account the uncertainty in the magnitude of frictional forces it may be reasonable to assume full friction in the bearings on one side of the fixed bearing and half friction on the other side.

4.9 Construction and Erection Loads

Erection loads are especially important for the design of composite and long-span bridges.

In long-span bridges the internal forces existing when the construction is completed are frequently adjusted by movements of supports or, in the case of cable stay bridges and suspension bridges, by adjustment of the cable forces.

In composite bridges the formwork for the deck is usually supported by the steelwork alone, and is not removed until after the deck becomes composite. The stresses induced in the composite deck by the removal of the formwork may be small enough to neglect, but in principle, they are a form of permanent prestressing, which can be considered in load combinations.

5. CRITICAL LOAD CASES FOR DESIGN

5.1 Load Combinations

Limit state design is probabilistic and load factors are determined to ensure that the probability of exceedence of a particular set of design actions (i.e. characteristic actions times partial safety factors) is satisfactorily small. The probability of two different actions, e.g. limiting live load and limiting wind load, occurring simultaneously is clearly less than that for either individual occurrence. It is therefore desirable to achieve the same reliability for different combinations of actions by adopting different partial safety factors on the individual components. Table 1 below shows the different factors that are used in the UK for two different combinations of action (load). Combination 1 is dead load plus live load. Combination 2 is dead load plus wind load plus live load.

Load	γ_{fL} At ultimate limit state combination	
	1	2
Dead	1,05	1,05
Superimposed Dead	1,75	1,75
Wind		
with Dead + S.Dead		1,40
with Dead + S.Dead + Live		1,10
Vertical Live Load		
HA alone	1,50	1,25
HA with HB or HB alone	1,30	1,10

Table 1 **Combination of Actions (Loads)**

(Source: BS 5400: Part 2)

5.2 Modelling the Construction Process

For simple bridges which are constructed from Class 1 or Class 2 sections, it is not necessary to consider the detailed build-up of stresses in the bridge as it is constructed. Such structures have sufficient ductility to redistribute stresses within the cross-section. It is therefore only necessary to check the adequacy of the structure at each stage in construction and to ensure that the completed structure can carry both fixed and variable actions, i.e.dead plus live loads.

Larger structures are usually constructed of Class 3 or 4 sections and it is not safe to assume that redistribution of construction stresses can occur. It is therefore necessary to model in some detail the build up of construction stresses throughout the construction process. Figure 21 shows the sequence for a typical three-span composite highway bridge. Each case is analyzed elastically and the stresses are summed at critical cross-sections.

5.3 Variable Actions on the Completed Structure

Considerable skill and computational effort are required to determine the critical variable action effects for all relevant cross-sections in the bridge.

The analysis has to determine governing values of the following global effects:

- positive longitudinal moments within the span
- negative longitudinal moments at internal supports
- greatest longitudinal moments at changes of girder cross-section
- maximum shears at supports
- maximum shears at changes of web resistance
- maximum reactions
- critical combinations of moment and shear (usually at supports)
- maximum torsions (usually most critical for box sections)
- maximum moments, shears and torsions on cross girders, cross bracing and slabs

In addition it is necessary to determine governing values of load moments, shears and torsions on the concrete slab or orthotropic deck.

National practice for this analysis varies considerably - at least partly in response to variations in national load specifications. However, it is possible to identify some general trends.

- most global analysis is carried out by grillage analysis
- influence lines are still used; sometimes just to identify critical locations for heavy vehicles and knife-edge loading and sometimes for the determination of numerical values. They may be developed by the use of coefficients for transverse distribution or they may be determined by grillage analysis

- most countries have one or two heavy vehicles, usually with defined axle and wheel layouts. They govern global effects for medium and short span bridges. They are applied at specific positions on the structure. These positions may be determined by general inspection, or by examination of influence lines. Some modern computer programmes have automatic load stepping facilities, both along and across the bridge with search routines to determine relevant maxima and minima.
- knife edge loads are applied at specific locations, usually at midspan or close to supports.
- distributed loads are applied over the full lengths of positive, or negative, influence lines. For example, both neighbouring spans are loaded to determine governing support moments, only one span is loaded to determine governing mid-span moment
- software routines for automatic summation are becoming more popular to determine governing values of action effects.
- local slab and deck analysis is carried out separately. This is discussed in Lecture 15.3.

6. CONCLUDING SUMMARY

- Loading on bridges is treated with much greater precision than on many other types of structure.
- Specified highway bridge loadings comprise heavy vehicle loads, uniformly distributed loads, knife edge loads and single wheel loads.
- Specified railway loadings comprise concentrated loads, preceded and followed by uniformly distributed loads.
- Dynamic effects are usually incorporated by quasi-static loadings.
- Wind, thermal, seismic and construction actions also need to be considered.
- Different partial safety factors are used for different combinations of actions to achieve the greatest uniform reliability, with economy.
- For most bridges, construction effects need to be summed throughout the construction process.
- A wide envelope of variable action effects needs to be considered carefully to determine the governing moments, shears and torsions on critical cross-sections.

7. REFERENCES

[1] Eurocode 1: "Basis or Design of Actions on Structures", CEN (in preparation).

[2] Eurocode 3: "Design of Steel Structures" Part. 2: Bridges and Plated Structures, CEN (in preparation).

[3] Eurocode 4: "Design of Composite Steel and Concrete Structures" Part. 2: Bridges, CEN (in preparation).

[4] Sanpaolesi, L., Sedlacek, G., Merzenich, G.M. "The Development of European Codes and Supporting Standard for the Design of Bridges,". ECCS 2nd International Symposium on Steel Bridges, Paris, April 1992.

ESDEP WG 15B
STRUCTURAL SYSTEMS: BRIDGES

Lecture 15B.3: Bridge Decks

OBJECTIVE/SCOPE

To introduce the types of bridge deck used for highway and railway bridges and their methods of design and construction.

PREREQUISITES

Lecture 15B.1: Conceptual Choice

RELATED LECTURES

Lecture 15B.4: Plate Girder Bridges
Lecture 15B.5: Truss Bridges
Lecture 15B.6: Box Girder Bridges
Lecture 15B.7: Arch Bridges
Lecture 15B.8: Cable Stayed Bridge
Lecture 15B.9: Suspension Bridges
Lecture 15B.10: Bridge Equipment (Bearings, Parapets)

SUMMARY

Bridge traffic is carried directly on the bridge deck. In highway bridges, the concrete slab is a very common deck type, and its usefulness is further increased when composite methods of construction are used. Recent bridges which have adopted the integral steel deck-and-flange system have stiffened the deck by longitudinal stiffeners comprised of either simple vertical members or torsionally strong U sections. The longitudinal stiffeners function partly as stringers and partly as plate stiffeners. In railway bridges, the rail track is either carried on cross ties which may be directly on the girders or on sleepers which rest on ballast. The ballast helps to reduce noise and vibration and allows continuous track maintenance.

1. INTRODUCTION

The principal function of a bridge deck is to provide support to local vertical loads (from highway traffic, railway or pedestrians) and transmit these loads to the primary superstructure of the bridge, Figure 1(1). As a result of its function, the deck will be continuous along the bridge span and (apart from some railway bridges) continuous across the span. As a result of this continuity, it will act as a plate (isotropic or orthotropic depending on construction) to support local patch loads.

Continuity ensures that whether or not it has been designed to do so, it will participate in the overall structural action of the superstructure.

The overall structural actions may include:

- Contributing to the top flange of the longitudinal girders, Figure 1(2).
- Contributing to the top flange of cross girders at supports and, where present in twin girder and cross girder structures, throughout the span, Figure 1(3).
- Stabilising longitudinal and cross girders, Figure 1(4).
- Acting as a diaphragm to transmit horizontal loads to supports, Figure 1(5).
- Providing a means of distribution of vertical load between longitudinal girders, Figure 1(6).

It may be necessary to take account of these combined actions when verifying the design of the deck. This is most likely to be the case when there are significant stresses from the overall structural actions in the same direction as the maximum bending moments from local deck actions, e.g. in structures with cross girders where the direction of maximum moment is along the bridge.

The passage of each wheel load causes a complete cycle of local bending stresses. The number of significant stress cycles is, therefore, very much higher for the deck than for the remainder of the superstructure. In addition, some of the actions of the deck arising from its participation in the overall behaviour are subject to full reversal; an example is the transverse distribution of vertical load between girders. For both these reasons, fatigue is more likely to govern the design of the bridge deck than the remainder of the superstructure.

2. HISTORICAL DEVELOPMENT

Modern decks consist of concrete slabs or orthotropic steel decks. Despite the different materials, it is possible to identify common themes in their development.

2.1 From Separation to Integration of Functions

Partly because of limited understanding of behaviour and methods of analysis, and partly because it suited historical methods of construction, early decks were separated from the remainder of the superstructure. The steel "battledeck" shown in Figure 2(a) comprised plate panels welded to rolled beams as stiffeners that were supported by and spanned simply between cross-girders which, in turn, spanned between the principal girders. The deck construction was relatively deep but could still fit within the overall depth of the truss. A similar approach can be seen in the concrete deck slab in Figure 2(b). The slab acts compositely with the stringers but does not contribute to overall bending.

Although this separation reduced the overall efficiency of the design, it is noteworthy that it does assist bridge repairs. For example, the entire deck of the Golden Gate Bridge in San Francisco was replaced during night time possessions, permitting the bridge to continue to be used during the day.

Modern decks in both materials are fully integrated into the overall superstructure as shown in Figure 3. These integrated decks improve the economy of the primary structure considerably. In all - steel construction the cross girders and main girders do not need separate top flanges. With a concrete deck, rolled sections (used for cross girders and main girders for short spans) will be considerably lighter. The top flanges of plate girders will typically be half the cross-section that would have been needed for non-composite construction.

The disadvantage of integrated construction is that repair or replacement of the deck is difficult and usually requires prolonged closure of the bridge.

2.2 Greater Simplicity

The increasing ratio of labour to material costs has encouraged the development of simpler forms of construction. Simplification has been considerably assisted by the development of modern welding techniques.

For example, early attempts to arrange stringers and cross-girders at the same level required the bolted or riveted connection shown in Figure 4a. Its modern equivalent in Figure 4b is readily accomplished with reliable welding.

2.3 Evolution of the Stringer in Steel Decks

A very important aspect of the historical development of steel decks is the evolution in form of longitudinal stiffeners or stringers. Initially, only open stiffeners shown in Figure 5a were utilised. Flats (i) and (ii) are simple to work with but are relatively inefficient in bending; bulb flats (iii) are more efficient in bending but are prone to lateral instability; tees (iv) and angles (v) offer a good combination of longitudinal bending strength and resistance to lateral buckling. All these open stiffeners have the basic disadvantage that they are flexible in torsion. Their use leads to a panel that is strongly orthotropic with little torsion stiffness (Dx >> Dy or Dxy). Such panels are inefficient as transverse distribution of local loads leading to a narrow effective width in bending and high longitudinal stresses under patch loading.

It would be possible but expensive to introduce local transverse stiffeners to increase Dy, but it is feasible to increase Dxy, and thereby improve transverse distribution, by using closed stiffeners. Figure 5b shows the closed stiffeners that have been developed. Initially, the "wineglass" stiffener(i) was developed for the early post war Rhine bridges in Germany. This stiffener gave a good combination of torsional and bending stiffness but was expensive to fabricate. Subsequently, the Vee and the trapezoidal stiffener were developed. The latter gives better bending resistance than the former, although it loses some torsional stiffness from cross-section distortion.

The earliest welded battledecks were detailed with continuity to the web of the cross girder, for example (a)(i). This created a very poor fatigue detail for the stringers. Subsequently, it became common practice to slot the web and have continuous stringers, for example (a) (iv) + (v) and (b) (ii) and (iii). With suitably rounded openings in the web, no fatigue problem is created in that element. It is noteworthy that the extreme fibres of the stringers are also not welded, thereby improving their fatigue performance.

3. MODERN HIGHWAY BRIDGE DECKS

3.1 Reinforced Concrete Slabs for Composite Bridges

3.1.1 Spans and depths

Reinforced concrete slabs are used for a wide range of composite bridges. Where they are supported at close centres, i.e. up to 3,5m, as shown in Figure 6a, they will usually have a uniform depth of 220 to 250 mm.

Concrete slabs are also used for widely spaced girders, as shown in Figure 6b. In such cases, the economies that can be achieved by reducing dead weight justify the extra cost of using variable depth slabs, as shown.

3.1.2 Methods of construction

It is possible, and is still quite common, to cast the concrete slab in situ on conventional formwork. However, considerable ingenuity has been devoted to improving this form of construction. Examples include:

- Use of full depth pre-cast units, with pockets to accommodate the shear connections. Grouting is used to complete the shear connection and complete the concrete between neighbouring slabs, Figure 7a.
- Use of full depth pre-cast units, with high-strength friction-grip bolts, Figure 7b and c.
- Use of glass reinforced plastic permanent formwork.
- Use of stiffened steel plate as external reinforcement. The plate is attached to the concrete by conventional shear connectors to form a composite slab.
- Use of pre-cast planks as permanent formwork. Depending on the detail of the reinforcement within these planks, they may or may not contribute to the resistance of the completed slab.

3.1.3 Methods of analysis and design

Apart from certain types of slab with participating pre-cast planks, all reinforced concrete slabs are isotropic and may be analysed by simple plate methods. Analysis needs to consider the various modes of behaviour of the slab.

- *Contribution to the overall longitudinal bending of primary girders*

  The effective breadth of the slab is included in the modulus of the girders. The stress resultants, compression at midspan, tension at supports, can readily be determined from the global analysis. The midspan regions are likely to be satisfactory in compression; the support regions usually require additional reinforcement, which should be placed within the effective breadth of the slab.

- *Contribution to the overall bending of cross-girders (where they are present)*

  The treatment is similar to that for the longitudinal girders.

- *Contribution to the overall behaviour of structure, e.g. transverse distribution of local loads from one girder to its neighbour*

  This effect should also be modelled within the global analysis. Usually the slab is replaced by equivalent beams in the grillage analysis, as shown in Figure 8. Not less than eight beam strips are recommended for each span to ensure adequate modelling of the structure. Calculation of the bending stiffness of the beam strip is straightforward - the slab is assumed to be uncracked and fully effective. It is also necessary to model the torsional stiffness of the slab - this is best done by distributing the total torsional rigidity equally between the transverse beam strips and the longitudinal twin girders, i.e. assign $\frac{d^3}{6}$ per unit width to both directions, where d is the depth of the slab.

- *Local bending action to transfer wheel and other local loads to the main superstructure*

  Analysis is required only of the slab local to the wheel load. Most practical situations can be reduced to standard cases and evaluated using standard influence charts [1]. Figure 9 shears schematically a typical chart (Pucher Influence chart). The patch leads are applied to the chart in a way that maximises the volume under the influence surface. The volume is then evaluated numerically. The simplification of support conditions to permit use of standard charts normally leads to a conservative assessment of worst moments.

Once the methods of analysis have determined the overall combination of moments, axial forces and shears on the slab, its adequacy in compression and shear can be checked and the reinforcement detailed in the conventional way. It is customary for some compression reinforcement to be required in the regions of highest moment.

3.2 Orthotropic Steel Decks

3.2.1 Introduction

Orthotropic steel decks have been subjected to considerable practical optimisation. The principal motivation for optimisation has been to develop the cheapest deck that will achieve a satisfactory fatigue life. There have been substantial problems with fatigue cracking, both at the stiffener/deck welds and the stiffener/crossgirder connections. The former has led to the use of thicker deck plate and more fatigue resistant welds; the latter has led to a particular form of welded connection.

The outcome of this practical optimisation has been the development of the standard "European" orthotropic deck that is described below.

3.2.2 Structural Behaviour of Orthotropic Steel Decks

Although the functions and the resulting stresses of the component parts of a steel-deck bridge are closely interrelated, it is necessary for design purposes to treat separately the three basic structural systems, as follows [2]:

System I. The main bridge system, with the steel deck acting as a part of the main carrying members of the bridge.

In the computation of stresses in this system in girder-type bridges the entire cross-section area of the deck, including longitudinal ribs, may be considered effective as flange.

System II. The stiffened steel-plate deck, acting as the bridge floor between the main members, consists of the ribs, the floor beams, and the deck plate as the common upper flange.

The major contribution of Pelikan and Esslinger [3] is in the prediction of the behaviour of System II, the continuous orthotropic plate on flexible supports.

System III. The deck plate, acting in local flexure between the ribs, transmitting the wheel loads to the ribs. The local stresses in the deck plate act mainly in the direction perpendicular to the supporting ribs and floor beams and do not add directly to their other stresses.

The governing stresses in the design of the deck are obtained by superposition of the effects of Systems I and II.

3.2.3 The "European" orthotropic deck and methods of construction

Figure 10a shows the basic cross-section of this deck which is judged to provide the most cost-effective and fatigue resistant design.

The most important detail of construction is the deck/stiffener weld. For a reliable fatigue life a sound full penetration weld is essential, Figure 10b. This can be achieved by a square cut to the end of the stiffener, providing that qualified weld procedures are adopted and a consistently good fit is achieved between the two plates. The latter requires careful manufacture of the trough stiffener and adequate jigs and clamps on the panel welding line.

Another important detail is the stiffener to cross girder (or diaphragm) connection shown in Figure 10c. The stiffener is continuous through an opening in the cross girder to ensure full continuity. Only the webs of the stiffener are welded to the cross girder; this improves the fatigue performance of the stiffener. The upper and lower 'mouse holes' have corner radii to minimise stress concentrations in the cross girder.

3.2.4 Methods of analysis and design verification

The detailed analysis of the orthotropic deck is well documented [2, 4]. The most practical method of analysis is that of Pelikan and Esslinger. This method is based on the application of Huber's equation. It assumes that the deck system is a continuous orthotropic plate, rigidly supported by its main girders and elastically supported by the floor beams.

The design procedure is divided into two stages:

- In the first stage it is assumed that the floor beams, as well as the main girders, are infinitely rigid.
- In the second stage, a correction is applied, considering the floor beams as elastically supported. The reactions of the plate on the floor beams are replaced by a load proportional at each point to the deflection of the floor beam. The total moments are found by superposition, due to the influence of dead and live loads assuming rigid supports and live loads assuming elastic floor beams.

Points of particular note are :

1. The effective breadth of a discretely stiffened plate is less than that of a fully continuous orthotropic plate. Its determination must recognise that only the deck plate is continuous.
2. Trough shaped stiffeners are not fully effective in torsion because of cross-section distortion. Guidance is available on suitable methods of accommodating this reduction in rigidity.

In principle, the design of the deck should be verified separately for static strength and fatigue resistance. For static strength the individual components of the deck need to be checked for the following stresses, in combination:

1. Longitudinal stresses from participation in overall bending of the superstructure.
2. Transverse stresses from participation in bending of the cross girder.
3. Longitudinal stresses and shear stresses from bending of the stiffened plate between cross girders.
4. Transverse bending of the deck plate between trough webs.

For fatigue loading the critical regions are those identified in 3.2.3 above. In practice, adequacy is demonstrated by experience rather than by calculation of the very complex elastic stress fields.

Usually, the flanges of bridge cross-sections are relatively wide with respect to their spans. The effects of shear lag need, therefore, to be included in the bending analysis.

Shear lag effects cause the stress distribution over the cross-section to be non-linear. The maximum stress values occur at the flange-to-web junctions. The effective width is defined by the condition that the stresses at the flange-to-web junction, according to engineering bending theory, must be identical to the maximum stresses calculated by applying the mathematical theory of elasticity.

The effective width b_m is defined as the width of a rectangular area of height $(\delta x)_{max}$, and having the same area as that enclosed by the distributed curve stress area. The effective width, Figure 11a is calculated by the following equation.

$$b_m = \frac{\int_0^b \delta_x \, d_y}{(\delta_x) \,|_{y=b}}$$

To solve this equation, it was necessary to establish simultaneous differential equations for determining both the deflection of the girder and the axial displacements at any point of the plate.

Most practical situations can be reduced to standard cases and evaluated using standard charts and tables, Figure 11b.

4. MODERN RAILWAY BRIDGE DECKS

4.1 Replacement Structures

Most modern designs for railway bridges are for replacements to existing, worn out structures. The design is dominated by the need to complete the replacement and re-open the railway line within a very limited possession, usually a single weekend.

4.2 New Alignments

Where a new line or a realignment is being developed, the most important criterion is an unusual one for steel structures, that of noise. Some earlier steel bridges were constructed with rail bearers that provided more or less direct support to the rails (Figure 12a, b and c). Traditional ballast was eliminated. These structures have proven to be very noisy - certainly too noisy for situations near residential accommodation and probably too noisy for passengers. This problem has been overcome by reintroducing the ballast throughout the bridge, Figure 12d and e. The ballast has the additional benefit of ensuring that track maintenance and line alignment are similar both on and off the bridge.

5. CONCLUDING SUMMARY

- Bridge Decks:
    - Provide support to local loads.
    - Contribute to overall longitudinal and transverse bending.
    - Stabilise the primary structure.
    - Act as diaphragms.
    - Assist in the transverse distribution of load between primary girders.
- Bridge decks are susceptible to fatigue.
- Modern bridge decks are integrated into the overall behaviour of the bridge.
- Reinforced concrete slabs, often with various forms of permanent formwork, are widely used in composite bridges.
- A standard European orthotropic deck has been developed.
- Modern railway bridge decks are:
    - Designed for fast construction during bridge replacement.
    - Designed to support track ballast to minimise noise.

6. REFERENCES

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7. ADDITIONAL READING

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   SCI Publication 084, 1991.

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ESDEP WG 15B
STRUCTURAL SYSTEMS: BRIDGES

Lecture 15B.4: Plate Girder and Beam Bridges

OBJECTIVE/SCOPE

To introduce the design of plate girder and beam bridges for highway, railway and footbridge applications.

PREREQUISITES

None

RELATED LECTURES

Lecture 8.4.1:	Plate Girder Behaviour and Design I
Lecture 8.4.2:	Plate Girder Behaviour and Design II
Lecture 8.4.3:	Plates Girder Design - Special Topics
Lecture 15B.1:	Conceptual Choice
Lecture 15B.2:	Actions on Bridges
Lecture 15B.3:	Bridge Decks
Lecture 15B.10:	Bridge Equipment
Lecture 15B.11:	Splices in Bridges
Lecture 15B.12:	Introduction to Bridge Construction

SUMMARY

This lecture identifies the principal types of composite and non-composite plate girder bridges that are used for highway, railway and pedestrian bridges. It discusses overall layouts, types of continuity, girder proportions, longitudinal and cross girder spacings and choice of deck slabs. It gives guidance for initial sizing of the most popular forms of construction. It discusses the means by which girders can be stabilised against lateral-torsional and distortional buckling. It offers guidance for detailed design from global analysis to important details.

1. INTRODUCTION

1.1 General

The simple beam is perhaps the most basic though not necessarily the most efficient form of bridging member. Unlike the arch, the beam supports applied loads primarily in flexure and associated shear. The distribution of material within the beam or plate girder cross-section is carefully selected to meet this requirement: material required to carry bending stresses is located at the upper and lower extremities of the cross-section for maximum efficiency, whilst the (usually deep) web panel separating the flanges is normally assumed to resist the full shear load applied to the section.

Depending on the spans involved, the loading intensity, costs of steelwork fabrication and any particular geometric and/or aesthetic requirements of the structure, a decision must be taken as to whether commercially available rolled beam sections or fabricated girders are to be used.

Plate girders and beams are utilised in a range of bridge forms [1-4]. Figure 1 shows the basic types of composite bridges. In Figure 1a the closely spaced main girders support a deck of uniform thickness directly. This form of construction is very simple and is widely used. However, its economy is reduced because there is much more shear capacity than is required, i.e. there are too many webs. If the webs are reduced in thickness so that they are more highly stressed they will require considerable, expensive stiffening.

In Figure 1b the economy in shear has improved considerably because there are only two girders, the minimum number possible. Theses are now much more widely spaced, and the slab will usually be haunched to provide sufficient transverse bending resistance.

At larger girder spacings the required slab thickness is increased beyond its economic limit. Cross girders are therefore introduced, as shown in Figure 1c. There is effectively no upper limit to the width of this form of construction.

As an intermediate solution the twin girders with intermediate stringer cross-sections shown in Figure 1d has also been developed. The presence of the stringer reduces the slab support spacing to that for multiple girder bridges. The stringer, much shallower than the main beams, has to be supported by robust cross bracing. The longitudinal shear resistance from the two main girders is now limited to the minimum required. However this system does require substantial cross-bracing which considerably reduces the overall economy.

Non-composite bridges can adopt any of the structural forms shown in Figure 1 with the concrete slab replaced by an orthotropic steel deck. Such structures can only be justified where there is an overriding need to minimise the structural weight.

More commonly non-composite plate girders are used for the half-through or through girder bridges shown in Figure 2. The deck may either be concrete, supported by and usually acting compositely with steel cross-girders, or orthotropically stiffened steel.

1.2 Types of Application

Composite girder bridges are primarily used for highway or footbridges. Non-composite steel plate girder and beam bridges can be used to carry highway, railway or pedestrian loadings. One of the more specialist applications is in the design of ramps for access to roll-on/roll-off ferries.

1.3 Range of Application

Composite bridges with Universal Beams can span up to 30 m for simple spans and up to 35 m for continuous construction. However, Universal Beams can only be used for spans close to the upper end of these ranges if girder spacing are reduced considerably. In many such cases it will be more economic to use plate girders.

Composite plate girders can be used for the great majority of medium span bridges. When spans exceed 80 to 100 m it is likely that box girders will be favoured, because of their improved torsional and aesthetic properties.

Non composite girders can be used for spans between 20 and 100 m.

1.4 Types of Through Girder Bridges

It is necessary to define the terms "through girder" and "half-through girder" bridges. These terms are common in the United Kingdom but appear to have no direct equivalents in many other countries. Figure 2 shows these two cross-section types and defines the essential difference between them. Two important points should be noted:

- plate girders or beams are unlikely to be used in through girder construction on aesthetic grounds and also because plate girders of the necessary depth (for example 6-6, 5 m to accommodate typical highway loading gauges) are impracticable. Trusses or arches would probably be used instead of through girder construction.

- any form of main girder (plate girder, beam, box girder, truss or arch) can be used in half-through girder construction.

For plate girder and beam bridges, therefore, it is possible to say that the through girder form will not be encountered in structures carrying highway or railway traffic.

2. SPAN ARRANGEMENTS

2.1 Continuous or Simple Spans

All girder bridges can be used for the range of longitudinal arrangements shown in Figure 3. For viaducts, the most economic arrangement is one in which all internal spans are of equal length L and the two end spans are each approximately of length 0,8L. Clearly, specific constraints imposed by the particular bridge site may prevent the use of such an arrangement.

The use of a continuous girders rather than a number of simply supported girders on a multiple span structure will prove to be more efficient structurally and generally therefore also more economic. There is also a potential saving arising from the reduced number of deck joints required. Fewer deck joints give a longer term saving in maintenance costs as the need for repair to both the superstructure and pier heads at intermediate pier positions as a result of leakage through failed deck joints is reduced or entirely eliminated.

The use of cantilever and suspended span construction results in a determinate structure for which the global analysis is straightforward. As in the case of multiple simply supported spans, the cantilever and suspended span configuration can prove attractive where there is a likelihood of significant differential settlement between supports, e.g. in areas of mining subsidence. However, the need for both half joints in the main girders at points of support of the suspended spans and deck joints at these same locations results in increased long term maintenance costs (for similar reasons to those discussed in the previous paragraph).

2.2 Proportion of Main Girders

Figure 4 shows the span-to-depth ratios that experience has demonstrated are likely to be the most economical for various types of girders. It is of course possible to adopt shallower construction to meet the constraints of a particular site but the weight and cost of the superstructure will thereby increase.

Figure 5 shows the two alternative ways of varying the depth of plate girders. The haunches of Figure 5a are most suitable for arrangements where the deck is above the main girders. Figure 5b shows the arrangement that is more suitable for half-through and through bridges.

3. INITIAL DESIGN OF COMPOSITE GIRDER BRIDGES

3.1 Girder Spacing and Deck Slab Thickness

The deck slab has to distribute wheel leads to the main girders and also to transfer some load from more highly loaded girders to adjacent ones. The spacing of main girders thus affects the design of the slab as well as the number of girders required.

For closely spaced girders, wheel loads determine the design of the slab including its reinforcement. The minimum thickness of slab is about 220 mm, on the basis of typical shear and crack width requirements. The total transverse moments in the slab are not very sensitive to girder spacing in the range 2,5 to 3,8 m because the increase in local moments as the spacing increases is almost balanced by the reduction in moments from the transfer of load between girders. The optimum slab thickness is typically 230 - 250 mm.

In selecting a suitable girder spacing, it is important to ensure that the cantilevers at the edges of the slab are limited to avoid overstressing the slab or forcing too much load onto the outer girders.

From the preceding discussion the following proportions for the cross-section in Figure 1a naturally emerge:

- slab depth of 230 to 250 mm
- girder spacing of 2,5 to 3,8 m
- cantilevers of not more than 1,5 m if they carry traffic or about 2,5 m if they carry footways that are protected by crash barriers to avoid local wheel loading.

This form of construction has proved to be economic for shorter span composite bridges. It is used throughout Europe for spans up to about 35 m. In the UK it is regularly used for spans up to around 60 m and exceptionally up to 100 m. At the longer spans economy has been achieved because the market for major plate girders has been met by specialist fabricators who have invested in semi-automatic fabrication lines.

Elsewhere in Europe the cross-section shown in Figure 1b has generally been adopted for structures of modest width, up to 12 m, and spans of over 35 m. The slab is usually of greater, and variable depth to allow the girder spacing to increase. Typical proportions are:

- a slab depth of 250 mm, increasing to 350 mm over the girders
- girder spacing of 6 or 7 m
- cantilevers that carry traffic of up to 2 m, extending to 3,5 m where there are footways protected by crash barriers.

Where the bridge is wider than about 12 m, it is usual to introduce cross girders as shown in Figure 1c. It follows from the earlier discussions that the optimum proportions are:

- slab depth of 230 to 250 mm
- cross girder spacing of 3,5 to 4,0 mm
- girder spacing - as wide as is necessary
- cantilevers will usually be smaller, no more than 1,5 m, because the penalty of increasing the girder spacing is only to increase cross girder size. Greater girder spacing decreases the proportion of an eccentric load that is carried by one girder.

3.2 Initial Selection of Flange and Web Sizes

Experience and a few empirical rules can be used for the approximate analysis that proceeds the initial selection of sizes. The following illustrates the process for a multi-girder bridge. Further guidance can be found in [5].

Governing shear occurs when the design "heavy vehicle" is placed directly over a girder adjacent to a support. If the girder is an edge girder, approximately 85% of the total shear will be carried by the girder under consideration. If it is an internal girder a more appropriate proportion is 70%.

Governing moments usually occur when the design "heavy vehicle" is directly over the girder at midspan. Approximately 75% of the vehicle is carried by an edge girder; for an internal girder the proportion approximately 50%. Simple moment distribution or other manual method of analysis will give realistic estimates of support and midspan moments.

Most designers size the web first so that it can carry 150% of the governing shear (the reserve is valuable in contributing to bending resistance). If the bottom flange is inclined, it will carry some of the shear, and the web can be reduced in thickness accordingly. For girders up to 1,5 m depth, including rolled sections, the web is usually proportioned so that it does not require any stiffening except at supports. In the range 1,5 to 2,5 m the optimum web is likely to require vertical stiffening possibly with horizontal stiffening near internal supports where more of the web is in compression. Above 2,5 m it is likely to require both vertical and horizonal stiffening.

The bottom flange is sized next to provide the required modulus. Consistent with the availability of standard plates and flats, it is usually made as wide as possible within codified limits on outstands. These proportions give the highest possible lateral inertia to the girder, minimising bracing requirements and assisting stability during erection.

At internal supports the top flange is usually made half the area of the bottom, additional tensile resistance being provided by slab reinforcement.

At midspan the top flange should only be reduced to 50% of the bottom if this is not to increase stability problems during erection. It will often be necessary to increase the top flange size for the construction condition.

3.3 Economic and Practical Considerations

3.3.1 General considerations

Clean lines to the overall appearance and minimum use of complex details are most likely to lead to an economic and efficient bridge structure, though external constraints often compromise selection of the best structural solution.

The fabrication of the basic I-section is not particularly expensive, especially with the use of modern semi-automatic girder welding machines (T and I machines). It is of the same order of cost as the material used. With the widespread use of computers in CAD and in control of fabrication shop machines, geometrical variations, such as curved soffits, varying superelevation and precambering, can be readily achieved with almost no cost penalty. Much of the total cost of fabrication is incurred in the addition of stiffeners, the fabrication of bracing members, butt welding, the attachment of ancillary items, and other local detailing which leads to a significant manual input to the process. The designer can thus exercise freedom in his choice of overall arrangement but should try to minimise the number of 'small pieces' which must be dealt with during the fabrication process.

The use of wide flats for flanges eliminates a cutting-out operation from the fabrication process, thus reducing costs. The rolled edge to the flat is to be preferred over the sharper flame-cut edge. It also allows the use of automatic saw-and-drill machinery for cutting to length, drilling of the holes and marking of shear stud positions. The rolling tolerances of wide flats have recently been brought into line with those of plates, and this makes them an attractive option for flanges. It is recommended that designs be tailored to permit their use, though some fabricators will still prefer to cut from plate, on account of the very good tolerances on width and straightness achieved with modern cutting equipment, which suits girder welding machines.

Expert advice should be obtained from fabricators to assist in the choice of details at an early stage in the design. Most fabricators welcome approaches from designers and respond helpfully to any interest shown in their fabrication methods.

The form of the substructure at intermediate supports, whether for reasons of appearance or of construction, often has a strong influence on the form of the superstructure. For example, a low clearance bridge over poor ground might use multiple main girders on a single broad pier, whereas a high level bridge of the same deck width and span over good ground might use twin main girders, with cross girders, on individual columns.

Highly skewed bridges are sometimes unavoidable, but it should be noted that the high skew leads to the need for a greater design effort, more difficult fabrication and more complex erection procedures. In particular, the analytical model, the detailing of abutment trimmer beams, precambering and relative deflection between main beams must all be considered carefully.

3.3.2 Construction considerations

Construction of a composite bridge superstructure usually proceeds by the sequential erection of the pieces of the main girders, usually working from one end to the other, followed by concreting of the deck slab and removal of falsework. However, situations vary considerably and the constraints on access will be a major influence on the erection sequence for any bridge. In some cases they might determine the form of the bridge. Before proceeding to detailed design, at least one erection scheme should be considered, and the requirements for it included in the detailed design.

In some circumstances, where access from below is difficult or impossible, launching from one or both ends may be appropriate. If so, this is likely to have a significant effect on girder arrangements and detailing. Advice should be sought from an experienced contractor.

Stability of girders during erection and under the weight of wet concrete will have a significant effect on the size and bracing of the top flange in midspan regions. Temporary bowstring bracing to individual girders may need to be provided if they are too heavy to be erected as pairs.

Site splices between the main girder sections are commonly connected with high strength friction grip (HSFG) bolts. Welded joints are more expensive and prove more onerous on quality control on a small job but should be considered on larger jobs and where their better appearance is warranted. One method or the other should be adopted throughout the bridge; it is uneconomic to use both methods.

4. INITIAL DESIGN OF NON-COMPOSITE PLATE GIRDER BRIDGES

4.1 Bridge Cross-Section

Figure 6 shows the basic types of cross-section for non-composite plate girder bridges.

If minimum construction depth is required either for aesthetic or economic reasons, the half-through cross-section, e.g. Figures 6a, 6b, 6e and 6f, will be the most appropriate solution for highway, railway or footbridges. This arrangement is commonly used in railway bridges where even maximum permissible approach gradients for the track are very small and where the minimum effective construction depth (Figure 2a) afforded by the half-through arrangement is important to minimise the cost of earthworks and land purchase on the approaches to the bridge. The half-through form does however have important implications for compression flange stability. These implications are discussed in greater detail in Section 5.

If construction depth is unlimited then a deck-type cross-section as shown in Figure 6c can be considered. It must be stated, however, that the use of a steel/concrete composite girder deck is generally much more economic than the orthotropic deck arrangement shown in Figure 6c. Only where minimum weight is the overriding design consideration will the orthotropic deck be an attractive solution.

An open grid steel deck arrangement for a railway structure results in a cross-section as shown in Figure 6d or a variant of this. The open deck form is however now almost unused for new railway structures in Europe particularly because ballast is used on nearly all modern railway structures and therefore some form of "closed" deck is required for ballast retention. In this "open" arrangement the main girders also act as rail bearers. One variant, discussed in greater detail in Section 5, is that in which the rails are supported on longitudinal stringers, themselves rigidly connected to flexurally stiff cross girders spanning between the two main girders. For all but the shortest spans this form of construction would almost certainly demand some form of wind bracing in plan as, unlike the previous cases, there is no deck plate to provide a horizontal diaphragm. Such plan bracing would be connected directly to the main girders.

4.2 Main Girders

The use of fabricated girders rather than rolled beam sections for the two main girders gives the designer freedom to select the most economical girder cross-section for the structure. However, where spans are relatively short and/or live loading intensity is low, rolled sections of suitable proportions are normally available.

Where parallel flanged main girders are used, i.e. where overall depth of girder remains approximately constant over the full span, a more interesting appearance can often be obtained by the introduction of a noticeable degree of precamber.

The degree of precamber which will be visually and geometrically acceptable in any particular situation depends on the nature of the crossing, e.g. road, rail or pedestrian traffic, and the interaction of the structural form with its surroundings.

Variable depth plate girders offer considerably more scope for a satisfactory final appearance. Clearly, however, they demand the use of fabricated rather than rolled beam sections. Deepening of the girder at intermediate support positions by the introduction of a curved soffit profile (Figure 5a) is one method of achieving varying depth. It should be noted however that a half-through arrangement combined with this form of haunched main girder is impracticable.

An alternative form of variable depth girder is one in which the lower flange remains almost horizontal in the final profile whilst the upper flange is gently curved in elevation with maximum overall girder depth occurring at midspan on the central span. An example of this form is shown in Figure 5b taken from [1]. This arrangement can readily be used in conjunction with the half-through form. It is probably fair to conclude that a variable depth girder of this type provides more satisfactory appearance in a multiple than in a single span configuration.

Some rolled beam sections and almost all plate girders of normal proportions require some form of web stiffening (either transverse or longitudinal or both). The functions of the various types of stiffeners are described in Lecture 8.4.3. The attachment of intermediate stiffeners to the exposed outer faces of the girders is often avoided for aesthetic reasons, although there may be little alternative to the provision of bearing stiffeners on both sides of the web at support positions.

Where bending moments increase to the extent that local thickening of flanges is required, the thickening can be achieved either by the attachment of flange cover plates or by the use of tapered plates. The latter, where required, have traditionally been machined from flat plate; however, tapered plates are now commercially available from some steel producers.

4.3 Deck

Three basic forms of deck [7] are shown in Figure 6: orthotropic (Figure 6c), modified filler beam (or composite transverse beams) (Figure 6a), and the steel plate type (Figure 6e). Of these three types, the orthotropic deck although lightweight, is the most expensive whilst the basic steel plate is only generally suitable for use in footbridges. Subsequent maintenance costs will also be greatest for the orthotropic deck.

Where the diaphragm action provided by an orthotropic deck or modified filler beam deck is used to provide resistance to transverse loads, e.g. wind, any resulting additional stresses must be allowed for in the design.

The use of permanent soffit formwork in the case of the composite transverse or modified filler beam deck can speed construction. This possibility is of particular importance in bridge construction or replacement over railway tracks or busy motorways.

Other aspects of these and other bridge deck types are discussed in Lecture 15B.3.

4.4 Initial Sizing of the Main Girder

Initial estimates of main girder proportions are generally made on the basis of experience or rules of thumb such as those given below. Such estimates of girder size then permit better estimates of dead load of the structure to be calculated. Additional guidance on proportioning plate girders is given in Lecture 8.4.1.

For highway and railway bridges, common proportions for main girders (where L_o is the length between points of zero moment) are:

Overall depth, h:	$L_o/18 \leq h \leq L_o/12$	(highway)
	$L_o/10 \leq h \leq L_o/7$	(railway)
flange width, b:	$0{,}25h \leq b \leq 0{,}35h$	
flange thickness, t_f:	$b/25 \leq tf \leq b/10$	
web thickness, t_w:	$t_w \approx h/125$	

These values should be regarded as indicative only.

Assuming the web carries approximately 20% of the factored bending moment, M, then a better approximation of the required flange cross-section area may be determined from:

$$A_f = 0{,}8 \frac{M}{h \cdot \sigma},$$

although it must be noted that the value of σ used will be dependent on girder effective length; consequently, an estimate of σ must be made which reflects the degree of restraint provided to the main girder.

Refinements are then carried out as part of the detailed design process to maximise girder efficiency whilst also satisfying any other stability, stiffener, fatigue or dynamic criteria which may be relevant.

In terms of final cost, it is frequently more important to design a girder for minimum labour input rather than for minimum steel tonnage (or at least to swing the balance in this direction). For example, the labour costs associated with fabrication of stiffeners for a highly stiffened, thin web plate are often considerably greater than the additional cost of material associated with the provision of a thicker, more lightly stiffened web. The usual solution is generally a compromise between minimum labour input and minimum tonnage.

5. GIRDER STABILITY AND BRACING

5.1 Introduction

Plate girders have a very low torsional stiffness and a very high ratio of major axis to minor axis second moment of area [1-3]. Thus, when bent above their major axis, they are very prone to lateral-torsional instability, Figure 7a. Adequate resistance to such instability has to be provided during construction.

In the completed structure one flange is usually stabilised by the deck. If the unrestrained flange is in compression, distortional buckling, Figure 7b, is a possible mode of failure and has to be adequately considered in design.

5.2 Composite Plate Girder Bridges

Figure 8 summarised the types of bracing and other forms of transverse structure that commonly occur within composite plate girder bridges. Figure 9 shows some typical arrangements of bracing systems.

Within a span the most convenient form of stabilising bracing is torsional bracing, Figure 8a. Not less than three such bracing lines are usually placed in each span, Figure 9a and b showing typical arrangements. The convenience of this form of bracing primarily arises because of the simple way that it ties two girders together into a stable substructure. Given adequate cranage this substructure can be assembled on the ground and lifted into position by a single operation. By tying pairs of girders together in this way their torsional displacements are suppressed and therefore, providing the system is sufficiently stiff, lateral-torsional buckling of the overall girders is prevented during construction. In the completed structure the concrete slab, restraining the top flange, prevents any instability in positive bending. For regions near internal supports and subject to negative moments, the bottom flange continues to need restraint (now against distortional buckling rather than lateral-torsional buckling). This restraint is effectively provided by torsional bracing. [Note that for shorter span structures, even when account is properly taken of pattern loading effects, the length of bottom flange in compression is usually so short that no such bracing is required for the completed structure. Where the web is stocky inverted U-frame action can be mobilised to assist with negative moment stability.] Modern practice is usually just to tie pairs of girders together as shown in Figure 9a. The discontinuity in the transverse bracing ensures that it has a low overall transverse bending distribution in the completed structure. Where full transverse bracing is provided it attracts considerable, and fully reversing, loads both to itself and the stiffeners to which it is attached. Such elements are therefore prone to fatigue damage. (There have been several such failures in North America.) Where discontinuous temporary bracing is used it may be safely left in for the service life of the structure; if continuous bracing is adopted it must be recognised that it will participate significantly in the structural behaviour of the completed bridge. It should either be designed accordingly for fatigue or removed after construction.

Plan bracing may also be provided near the top flange for the construction condition, Figure 7a. However, it is likely to interfere with the slab construction and is of negligible benefit in stabilising negative moment regions.

Plan bracing may sometimes be required in the completed structure near the bottom flange, Figure 8a. It may be used just near internal supports to stabilise the bottom flange in compression. It may also be needed in more major structures which could be prone to aerodynamic instability. One way of preventing such instability (flutter) is to separate the torsional and vertical natural frequencies of the structure. Lower flange plan bracing, effectively turning the pairs of plate girders into cells, increases the torsional stiffness sufficiently to achieve the desired effect.

At most supports there are separate columns for each girder, Figure 8b or all the girders sit on a substructure cross head. In such cases, support bracing, is required to:

- provide torsional restraint to the girders
- transfer wind and other transverse forces to the bearings and hence into the substructure.

Where it is not possible to provide direct support to each longitudinal girder, as shown in Figure 8c, a cross girder is required to transfer the girder vertical reactions to the supports. It clearly also will provide the bracing functions outlined above.

5.3 Non-Composite Plate Girders

Where the plate girders support the deck at or near the top flange, as shown in Figure 6c, d and e, considerations are broadly similar to the composite girders discussed in Section 5.2.

However, the half-through or through girder arrangements of Figure 6a, b and f cannot adopt any form of triangulated bracing because it would interfere with the bridges function. The deck can usually be designed as a horizontal beam and provides translational restraint at its level but the flange remote from the deck can only be stabilised by using U-frame action.

The form of the U-frame action can either be continuous or discrete, depending on the form of the deck structure and the slenderness of the webs, as shown in Figure 10. The degree of restraint provided to the compression flange depends directly on the stiffness of the three main U-frame components: the transverse member, the two webs of the main girder (including any associated vertical stiffeners) and their connections. The effective length of a compression flange restrained by U-frame action is usually calculated by recourse to the theory of beams on elastic foundations [6], the spacing supports being provided by the U-frames.

6. DETAILED DESIGN

The detailed design stage confirms or refines the outline design produced in the initial design stage. It is essentially a checking process, applying a complete range of loading conditions to a mathematical model to generate calculated forces and stresses at critical locations in the structure. These forces and stresses are then checked to see that they comply with the 'good practice' expressed in the code. The detail of the checking process is sufficiently thorough to enable working drawings to be prepared, in conjunction with a specification for workmanship and materials, and the bridge to be constructed.

6.1 Global Analysis

A global analysis is required to establish the maximum forces and moments at the critical parts of the bridge, under the variety of possible loading conditions. Local analysis of the deck slab is usually treated separately from the global analysis; this is described in Lecture 15B.3.

It is now common practice to use a computer analysis, and this facility is assumed to be available to the designer. Programs are available over a wide range of sophistication and capability. The selection of program will usually depend on the designer's in-house computing facilities. However, for a structure as fundamentally simple as a beam-and-slab bridge, quite simple programs will usually suffice.

The basis of most commonly used computer models is the grillage analogy. In this model the structure is idealised as a number of longitudinal and transverse beam elements in a single plane, rigidly interconnected at nodes. Transverse beams may be orthogonal or skewed with respect to the longitudinal beams.

Each beam element represents either a composite section (e.g. main girder with associated slab) or a width of slab (e.g. a transverse element may represent a width of slab equal to the spacing of the transverse elements). Figure 11 shows examples of typical grillages.

6.2 Actions and Combinations

Because many different load factors and combinations are involved in the assessment of design loads at several principal sections, it is usual for each load to be analysed separately and without load factors. Combination of appropriate factored load cases is then either performed manually - usually by presentation in tabular form - or, if the program allows, as a separate presentation of combined factored forces. Since so many separate load cases and factors are used to build up total figures, the designer is advised to include routine checks (such as totalling reactions) and to use tabular presentation of results to avoid errors. The graphical displays and printout provided now by analysis and spreadsheet software can also be recommended for checking results.

The object of the analysis is to arrive at design load effects for the various elements of the structure. The most severe selection of loadings and combinations needs to be determined for each critical element. The main design load effects which are to be calculated include the following:

- Maximum moment with co-existent shear in the most heavily loaded main girder: at midspan; over intermediate support; and at splice positions.
- Maximum shear with co-existent moment in the most heavily loaded main girder: at supports; and at splices.
- Maximum forces in transverse bracing at supports (and in intermediate bracing if it is participating).
- Maximum and minimum reactions at bearings.
- Transverse slab moments (to be combined with local slab moments for design of slab reinforcement).
- Range of forces and moments due to fatigue loading (for shear connectors and any other welded details which need to be checked).

In addition, displacements and rotations at bearings will need to be calculated.

The total deflections under dead and superimposed loads should be calculated so that the designer can indicate the dead load deflections on his drawings.

Selection of the most heavily loaded girder can usually be made by inspection, as can the selection of the more heavily loaded of intermediate supports. Influence lines can be used to identify appropriate loaded lengths of the maximum effects. If cross-sections vary within spans, or spans are unequal, then more cases will need to be analysed to determine load effects at the points of change or in each span.

The effects of differential temperature and shrinkage modified by creep are calculated in two parts. The first is an internal stress distribution, assuming that the beam is free to adopt any curvature that this produces (primary effects). The second is a set of moments and shears necessary to achieve continuity over a number of fixed supports. These moments and shears give rise to further longitudinal and shear stresses (secondary effects).

6.3 Element and Connection Design

Detailed design of the plate girder is discussed in Lectures 8.4.

Detailed design of splices and other connections are discussed in Lecture 15B.11 and in Lectures 11.

6.4 Effects Peculiar to Steel Open Grid Deck Configurations

Sections 6.4.1 and 6.4.2 describe situations in which additional stresses arise in longitudinal stringers and cross girders in steel open grid deck arrangements. Whilst the use of this type of deck is now uncommon in Europe for the reasons stated in Section 4.1, this form of construction nevertheless highlights two aspects which are of structural importance and which also serve to illustrate a broader structural principle.

It is assumed that twin longitudinal main girders are connected by relatively stiff cross girders at appropriate intervals; longitudinal stringers are assumed to be rigidly connected to these cross girders. For deck type railway structures the stringers would be located at the top of the section to act as rail bearers. This then gives rise to a mismatch in neutral axis levels between the main girders and stringers, Figure 12.

6.4.1 Bending of the Stringers

In the deck cross-section shown in Figure 12, Δh represents the difference in level between the neutral axes of the main girders and longitudinal stringers. The rigid connections between stringers and cross girders ensure that the curvature of the main girders is also imposed on the stringers. Expressing curvatures in terms of M/EI for each element and equating, leads to:

$$M_{st} = \frac{I_{st}}{I_{mg}} M_{mg}$$

where

M_{st} is the bending moment in the stringer

M_{mg} is the bending moment in the main girder

I_{st} is the second moment of area of the stringer

I_{mg} is the second moment of area of the main girder

M_{st} can be reduced by making the final connection of stringer to cross girder after the bridge carries its own dead load, thus ensuring that M_{st} arises purely from live load effects.

6.4.2 Weak axis bending of the end cross girder

Assume ϕ is the absolute value of the rotation (in the plane of loading) of the ends of the main girders at supports and I_y is the second moment of area of one flange of the cross girder with respect to the minor axis of the section. Neglecting second order effects, the lengths of the neutral axes of the stringers do not change (i.e. shortening is assumed to be negligible). The displaced arrangement is then as shown in Figure 13.

This displacement of the end of the top flange of the cross girder, Figure 13b, is approximately:

$$\delta = \Delta h \, \phi$$

As the stringers suffer no shortening they serve as fixed supports to the cross girder flange and as a result this flange deforms in plan.

The displacement δ can be assumed to be the result of application of a tip force F to the flange, the force-displacement relationship being:

$$\delta = F \left[\frac{a^3}{3EI_y} + \frac{a^2 d}{2EI_y} \right]$$

in which a, d are the dimensions indicated in Figure 13a and I_y is the second moment of area of one flange of the cross girder with respect to the minor axis of the cross girder section.

The maximum bending stress in the cross girder flange arising from the above effect is therefore:

$$\sigma_f = \frac{3 \, \Delta h \, \phi \, b \, E}{a \, [2a + 3d]}$$

where b is the flange width of the cross girder.

The resulting bending stresses in the flange are not insignificant. This fact is demonstrated by an example in which the following values are assumed:

Δh	=	600 mm
ϕ	=	0,003 rad
b	=	300 mm
a	=	1000 mm
d	=	1500 mm,

giving

$$\sigma = 52 \text{ N/mm}^2$$

Clearly the additional stress disappears when $\Delta h = 0$ and, although reducing b is beneficial, varying flange thickness theoretically has no effect. Reductions in transverse dimensions a and d have an adverse effect.

7. CONCLUDING SUMMARY

- Beams and plate girders are widely used for bridges spanning between 20 and 100 m.
- Several forms of construction have developed to meet specific needs for highway, railway and pedestrian bridges. The most common are:

  Composite multi-girder bridges
  Composite twin girder bridges with haunched slabs or cross-girders
  Half-through and through girder bridges.
- Experience has defined limited ranges of layouts for each of these bridges that are effective and economic.
- Simple rules may be used for the initial sizing of most girder bridges.
- Plate girders are prone to lateral-torsional buckling. They need to be stabilized by the deck slab and/or bracing and/or U-frame restraint.
- The bridge is usually modelled as a grillage for global analysis with separate analysis for local deck moments.
- Simple, practical details can be defined for all parts of plate girder bridges, maximising their economy and hence justifying their popularity.

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ESDEP WG 15B
STRUCTURAL SYSTEMS: BRIDGES

Lecture 15B.5: Truss Bridges

OBJECTIVE/SCOPE

This lecture gives information about the design and detailing of truss bridges. It is intended for engineers with some experience.

PREREQUISITES

Lecture 15B.2: Actions on Bridges

RELATED LECTURES

Lecture 15B.1: Conceptual Choice
Lecture 15B.3: Bridge Decks

SUMMARY

The history of truss bridges is reviewed and different configurations are described. Design principles are presented, e.g. span ranges, span-to-depth ratios and arrangement of diagonals. Different sections for chords and diagonals are shown and discussed together with the connections between the members. Analysis of trusses is treated in a general way and recommendations are given on what has to be considered and what can be neglected.

1. INTRODUCTION

The configurations of truss beam bridges are shown in Figure 1. Underslung trusses are rarely used in modern construction.

Through and semi-through truss bridges are used when the depth of deck construction is very limited, for instance, when a highway or a railway crosses a canal.

It is unusual for through-trusses to be economic for highway bridges, except for very long spans. With less severe restrictions on the gradients on the approach embankments it is, in addition, much easier for a road to gain the height required for a deck bridge than for a railway to do so.

Semi-through trusses tend therefore to be used for highways while through and semi-through trusses are still used for railways.

The principle of a truss is simple. The structure is composed of top and bottom chords triangulated with diagonals and/or verticals in the webs so that each member carries purely axial load. Additional effects do exist but in a well designed truss they will be of a secondary nature and may be neglected.

Global moment on a truss is carried as compression and tension in the chords as shown in Figure 2a. Global shear is carried as tension or compression in the diagonal and vertical members. In the simplified case, where the joints are considered as pinned, and the loads are applied at the nodes, the loading creates no bending moment, shear, or torsion in any member. Loads applied in such a way as to cause bending, shear, or torsion usually result in inefficient use of material.

The saving of material compared to a plate girder is clear when the webs are considered. In a truss the webs are mainly 'fresh air' - hence less weight and less wind pressure.

A truss can be assembled from small easily handled and transported pieces, and the site connections can all be bolted. Trusses can have a particular advantage in countries where access to the site is difficult or supply of skilled labour is limited. Undamaged parts of a truss bridge can easily be re-used after an accident or the effects of war.

2. DIFFERENT TYPES OF TRUSS

2.1 Historical Background

The truss as a structural form dates back to Roman times. A bronze truss was used in the Pantheon.

In the nineteenth century, the United States can claim to have created the greatest number of different types of truss. Their use of timber and their enthusiastic pioneering spirit created some unlikely looking structures, but nevertheless firmly established the truss as the ideal form of bridge at that time for medium spans.

Eiffel built lattice trusses in France (Figure 2f). Fowler and Baker, however, introduced a major innovation by adopting steel tubular sections as the main compression members for the Forth Bridge which is well known throughout the world for its grandeur. Modern truss bridges also use box sections for the compression members.

The Hungarian architect Virgil Nagy built the very aesthetic Ferenc Jozsef truss girder bridge in Budapest over the Danube in 1892. The bridge is supported by Pratt-truss girders of variable height (Figure 2e). The central span is 175m long with an isostatic central part of 47m.

For most modern bridgework the Warren truss (with its modifications) is perhaps the most commonly used type because of its simplicity. Modern labour costs dictate a minimum of members and connections.

2.2 Highway Truss Bridges

The Warren configuration shown in Figure 2 is usually chosen. When the length of the gap to be crossed makes the use of a multiple span bridge unavoidable, it is cheaper and usually possible to raise the road line and build another type of bridge requiring a greater depth under the deck.

For this reason highway truss bridges usually have only one span (Figure 3). Their appearance is well adapted aesthetically to cross canals in flat landscapes.

The spans are usually between 60 and 120m which is the normal economic range. The longest span was the old Neuwied bridge over the Rhine (212m) which was replaced by a cable stayed bridge.

The span-to-depth ratio is normally about 15.

2.3 Choice of Truss Configuration For Railway Bridges

Three basic truss bridge configurations are shown in Figure 1.

The most economic truss bridge configuration, especially for railway bridges, is the underslung truss where the live load runs at the level of the top chord. The

top chord then serves the dual function of support for the live load (as the sleepers sit directly on the chord) and the main compression member. There is, however, the disadvantage that clearance under the bridge is reduced. It is thus common for the approach spans over a flood plain, or over unnavigable parts of the river, to be underslung while the navigation channels are crossed with through trusses.

Where the spans are short, and underslung trusses are not possible, it may be economic to have the top chord below the loading gauge level by using semi-through trusses. Bracing between the top chords is not possible and restraint to the compression members has to be provided by U-frames. However for spans where semi-through trusses have been used in the past, plate girder bridges are now very competitive, and now semi-through trusses are seldom used for railway bridges.

Where the spans of railway bridges are long the economic depth is usually great enough to allow bracing to be provided above the loading gauge level. Such trusses are termed 'through trusses'. The use of material in bracing rather than U-frames is considerably more efficient.

For shorter spans the choice is between the Warren and the Pratt configuration. In the simple Warren truss, the diagonals work alternatively in compression and tension, whereas in the Pratt truss, all the diagonals are in tension and the shorter posts take compression.

To cater for the heavy loading on railway bridges, the cross girders should be fairly close together. This requirement leads to the hangers of the Modified Warren truss which sub-divide the bottom chord. Economic design of the top compression chord leads to sub-division with a post.

The majority of truss bridges are simple spans, but there are many examples of continuous trusses. The immediate benefit on member forces when a continuous structure is employed is offset to some extent by increased fatigue effects. In a simple truss it is common for only some of the diagonals to be influenced by fatigue. These diagonals are usually those at mid-span where the smallest available section has to be used in any case. In contrast, most of the diagonals in a continuous truss, and some of the chord members may well suffer fatigue, particularly when welded construction is used.

Even where continuous trusses show savings in the use of steel, they may not be economic. On a 1700m long bridge in India, the alternative continuous truss design was about 5% lighter that the simple spans which were considered more economic on account of standardization of fabrication detail and erection procedure.

It should be noted here that the design loading has a considerable effect on the truss configuration. For example, with combined highway and rail loading, trusses with two decks can be very economic.

2.4 Particular Applications

- As dead load is a dominating factor for movable bridges, bascule spans are often built using steel truss girders. Figure 4a shows an example of a rear part made for a movable truss bridge. Most connections are butt-welded and governed by fatigue considerations. This kind of bridge will not be further discussed here. For more information, see Lecture 1B.6.2.

- Temporary bridges for emergency purposes are almost always truss bridges because of their adaptability to various spans and support conditions, e.g. Eiffel, Bailey, Arromanches, Callender-Hamilton, see Figure 4b.

3. GENERAL DESIGN PRINCIPLES

3.1 Span Range

For spans from 60m to 120m for highways and from 30m to 150m for railways, simple spans can prove economic when favourable conditions exist.

Large spans using cantilever trusses have reached a main span of 550m. Trusses have to compete against plate girders for shorter spans, against box girders for medium spans and cable-stayed bridges for longer spans.

3.2 Ratio of Span to Depth

The optimum value for this ratio depends on the magnitude of the live load that has to be carried. It should be in the region of 10, being greater for road traffic than for rail traffic. For twin track rail loading the ratio may fall to about 7,5. A check should always be made on the economic depth for a given bridge.

3.3 Geometry

For short and medium spans, it will generally be found economic to use parallel chords to keep fabrication and erection costs down. However, for long continuous spans, a greater depth is often required at the piers, Figure 2e.

Skew truss bridges should be avoided as far as possible.

An even number of bays should be chosen to suit the configuration of diagonals in a Pratt truss. If an odd number is chosen there will be a central bay with crossed diagonals. This arrangement is not usually desirable except perhaps at the centre of a swing bridge. The diagonals should be at an angle between 50° and 60° to the horizontal.

Secondary stresses should be avoided as far as possible by ensuring that the neutral axes of all intersecting members meet at a single point, in both vertical and horizontal planes. This will not always be possible, e.g. cross girders will be deeper than the bottom chord and bracing members may be attached to only one flange of the chords.

3.4 Grade of Steel

Grade Fe 510 steel should be used for the main members with Grade Fe 430 or 360 used only for members carrying insignificant load, unless the truss has to be fabricated in a country where there is no ready supply of higher grade steel. For a truss designed using Grade Fe 510 steel, the amount of Grade Fe 430 or 360 steel used would normally be about 7%. For very long spans higher grades will be economical, e.g. quenched and tempered steel or thermo-mechanically processed steel with yield strength 500 - 600 MPa, provided that fatigue is not governing.

3.5 Compression Chord Members

These members should be kept as short as possible and consideration given to additional bracing if economical.

The effective length for buckling in the plane of the truss is normally not the same as that for buckling out of the plane of the truss. This effect can be further complicated in through trusses where horizontal bracing may be provided at mid panel points as well as at the main nodes. When making up the section for the compression chord, the ideal disposition of material will be one that produces a section with radii of gyration such that the ratio of effective length to radius of gyration is the same in both planes. In other words, the member is just as likely to buckle horizontally as vertically.

Eurocode 3: Part 1.1 [1] permits the effective length factors for truss members to be determined by analysis. Otherwise very conservative values are given of 1,0 and 0,9. However, as Eurocode 3: Part 1 applies to buildings, which have relatively small span trusses where absolute economy in steel weight is not vital, it is assumed that the clause is not appropriate to bridges. It is anticipated that the effective length of bridge truss members will be covered in Part 2 of Eurocode 3 [2]. As an example of current practice see Table II of BS5400 Part 3 [3].

In the case of semi-through bridges, the top chord is supported laterally by the diagonals and behaves as a strut supported on springs. The method of determination of its effective length is given in the appropriate bridge codes.

The depth of the member needs to be chosen so that plate dimensions are sensible. If they are too thick, the radius of gyration will be smaller than it would be if the same area of steel was used to form a larger member using thinner plates. The plates should be as thin as possible without losing too much area when the effective section is derived.

Trusses with spans up to about 100m often have open section chords, usually of "top-hat" section, see Figure 5. Here it is often desirable to arrange for the vertical posts and struts to enter inside the top chord member, thereby providing a natural diaphragm and also, usually, avoiding the need for gussets at alternate nodes, although packs will be needed.

For trusses with spans greater than about 100m, the chords will usually be box shaped so allowing the ideal disposition of material to be made from both economic and maintenance viewpoints.

For shorter spans rolled sections or rolled hollow sections may occasionally be used.

Advantages and disadvantages and comments on fabrication of the five alternative configurations shown in Figure 5 are:

a. Top Hat (i)	Welding distortion may be a problem although the situation at the bottom flange may be improved by adding a sealing fillet. This provision is recommended to avoid corrosion.
	Welds need to be ground flush at gusset positions.
	Battens or lacing are required as local bracings.
b. Top Hat (ii)	More suitable for automatic welding than Top Hat (i). Requires machining at nodes to allow entry of verticals and gussets.
	For fatigue reasons keep toes of fillets at least 10mm from edges of plates.
	Battens or lacing required.
	Keep outstand of bottom flange large enough to permit direct attachment of top lateral system.
c. Box	Provides optimum buckling strength.
	Provides clean profile and easy maintenance.
	No lacing required.
	Access for installation of internal diaphragms is difficult.
	Additional gussets required for attachment of top laterals.
d. Rolled Sections	Bad for trapping dirt and debris.
	Packing required at joints as nominal section depths vary slightly.
e. Rolled Hollow Sections	Crevices are formed at gussets unless special precautions are taken.

3.6 Tension Chord Members

Tension members should be as compact as possible, but depths have to be large enough to provide adequate space for bolts at the gusset positions. The width out of the plane of the truss should be the same as that of the verticals and diagonals so that simple lapping gussets can be provided without the need for packing.

It should be possible to achieve a net section about 85% of the gross section by careful arrangement of the bolts in the splices. This means that fracture at the net section will not govern for common steel grades.

As with compression members, box sections would be preferable for ease of maintenance but open sections may well prove cheaper.

Four alternative configurations are shown in Figure 6. Their advantages and disadvantages are:

a. Open Box	Welding distortion may be a problem but could be improved by adding sealing fillets at the corners. Welds need to be ground flush at gusset positions. Battens or lacing required.
b. Box	Provides clean profile and easy maintenance. No bottom plates required. Access for installation of internal diaphragms is difficult.
c. Rolled Section	Bad for trapping dirt and debris. Packing required at joints.
d. Rolled Hollow Section	Crevices formed at gussets unless special precautions are taken.

3.7 Vertical and Diagonal Members

These members should be all the same width normal to the plane of the truss to permit them to fit flush with or to be slotted inside the top chord (where the top-hat section is used) and to fit flush with the bottom chord. However, the width of the diagonals in the plane of the truss should be reduced away from the supports by about 75mm per panel. This reduction may mean that some members are understressed. It is often possible to use rolled sections, particularly for the lightly loaded members, but packs will probably be required to take up the rolling margins. This fact can make welded members more economic, particularly on the longer trusses where the packing operation might add a significant amount to the erection cost.

Aesthetically, it is desirable to keep all diagonals at the same angle, even if the chords are not parallel. This arrangement prevents the truss looking over-complex when viewed from an angle. In practice, however, this is usually overruled by the economies of the deck structure where a constant panel length is to be preferred.

Five alternative configurations are shown in Figure 7. Their advantages and disadvantages are:

a. Open Box (i)	The partial penetration welds are expensive and alternative (ii) might be cheaper. Battens or lacing required.
b. Open Box (ii)	Continuous or intermittent fillet welds can be laid simultaneously. Intermittent welds should only be used in bridges when corrosion is not a significant problem. Battens or lacing required.
c. Made-up I	Not recommended for end diagonals as it acts as a water chute down to the bearings.
d. Rolled Sections	Packing required at joints as nominal section depths vary slightly.
e. Rolled Hollow Sections	Crevices form at gussets unless special precautions are taken.

3.8 Maintenance

As with any structural design, the problems that may confront the maintenance team should be fully appreciated. The problems can be numerous, but a good design will avoid most of the common difficulties. For example:

Water	Try to keep water out but always assume it is going to get in and provide a way for it to escape when it does. "Sealed" sections should be provided with a drainage hole at the lowest point.
Dirt and Debris	Try to keep dirt and debris out, remembering that wind and rain will bring them in.
Painting	Remember that, if access is difficult, the bridge will not be painted or at least only badly, and probably not inspected. Box sections make painting easier, but rolled hollow sections leave nasty crevices at gusset positions, unless the joints are welded.
Birds	Birds will nest and roost in the most unlikely places!

4. LATERAL BRACING

Unless an orthotropic or concrete deck is provided, stringer bracing, braking girders and chord lateral bracing are needed to transmit the longitudinal live loads and the wind and/or earthquake loads to the bearings and also to prevent the compression chords from buckling. When a solid deck is used, the interaction between deck and trusses has to be considered.

For the lateral bracing of the chords, where a "Saint Andrew's Cross" type system as shown in Figure 8a is adopted, the nodes of the lateral system will coincide with the nodes of the main trusses. Interaction will take place which must be taken into account. As a result of the interaction, the lateral system may carry as much as 6% of the total axial load in the chords.

Figure 8b shows the lateral system in its original form and in its distorted form after axial compressive loads are applied in the chords. Owing to the shortening of the chord members ac and bd, the rectangular panel deforms as indicated by the dotted lines, causing compressive stresses in the diagonals and tensile stresses in the transverse members. The transverse batten members are indispensable for the correct performance of a St Andrew's cross bracing system.

The interaction can be significantly reduced by using a "Diamond" system of lateral bracing where the nodes of the lateral system occur midway between the nodes of the main trusses, Figure 8c. With this arrangement, "scissors-action" occurs when the chords are stressed, and the chords deflect slightly laterally at the nodes of the lateral system.

In the principal buckling mode of a "Diamond" lateral bracing system, one half of the diamonds have all their members in tension (see Figure 9).

For railway bridges, Figure 10 illustrates an economic lateral system at deck level which consists of a simple single member which also functions as part of the braking girder. The additional girders help to resist the braking forces arising from trains.

Wind loading on diagonals and verticals can be split equally between top and bottom lateral systems. The end portals (either diagonals or verticals) then have to carry the load applied to the top chord down to the bottom chord.

Clearly, where only one lateral system exists (as in semi-through or underslung trusses), then this single system must carry all the wind load.

In addition to resisting externally applied transverse loads due to wind, etc. lateral bracing stabilizes the compression chord. The lateral bracing ensures that reasonably small effective lengths are obtained for the truss members. Local lateral bracing is also required at all 'kinks' in the chords where compressive loads are induced into the web members irrespective of whether the chord is in tension or compression because of the angular direction change of the chord.

5. ANALYSIS

5.1 Global Load Effects

Generally trusses have stiff joints. The secondary stresses due to joint stiffness and truss deformation can be ignored in the ultimate limit state check. They have to be considered where the serviceability limit state check is required, and for fatigue. However, these secondary effects are generally insignificant.

The serviceability check is not required for tension members or for some slender compression members. Where it is not required, the traditional manual method of truss analysis assuming pin joints is adequate for global analysis.

Computer analysis can take joint stiffness into account and secondary moments are determined automatically. The effects of the primary axial loads and the secondary moments are combined by the use of suitable interaction formulae.

In a statically indeterminate truss, temperature effects have to be considered. They are usually not significant.

5.2 Local Load Effects

i. **Loads not applied at truss joints**

Two types of local load effects have to be considered:

a. Those due to loads applied in the plane of the truss away from a joint. A typical example of this type of loading is on the upper chord of an underslung railway bridge where the sleepers rest directly on the top flange of the chord.

b. Eccentric loads not in the plane of the truss, such as loads from cross girders.

ii. **Eccentricities at joints**

Flexural stresses due to any eccentricity at joints have to be taken into account by sharing the moments due to eccentricity between the members meeting at joints in proportion to their rotational stiffness. For the main trusses the centroidal axes of all members should meet at a point wherever possible. The only case where a small degree of eccentricity is unavoidable is when asymmetrical "top-hat" sections are used and it is not possible for the centroidal axes of adjacent members of different sizes to be co-linear.

Where possible the axes of the lateral systems should be in the same planes as those of the truss chords. However sometimes the upper laterals of a through truss have to be connected to the top flange of the upper chord and eccentricity is unavoidable. Since the loads in upper lateral systems are generally small, the additional resulting stresses are insignificant. Similarly on some through bridges, the bottom laterals have to be connected to the bottom flange of the lower chord to avoid the cross girders and stringers.

6. CONNECTIONS

6.1 General

The major connections in bridge trusses occur at the truss nodes where the web members are connected to the chord members. This connection usually incorporates a splice in the chord member and sometimes also in one or both of the minor truss connections joining the cross girder and the lateral system to the truss.

Site connections can be made by high strength friction grip bolts for reasons of economy and speed of erection. Good site welds are difficult to achieve where access is difficult and fatigue life of welded joints is lower than that of bolted joints.

However, in several countries, the connections are now usually butt-welded on site. Figure 11 shows different gusset geometries which are used to obtain durability in view of the fatigue-governing effects.

When a concrete slab is cast in place to support the highway or the railway, the horizontal forces caused by the shrinkage of the concrete should be taken into account in the design of the lower chord connection joints.

6.2 Truss Joints

At the nodes of a truss where the web members are connected to the chords, there is a change in load in the chord which necessitates a change in its cross-section area. The node is, therefore, the point at which there is a joint in the chord as well as being the connection point of the web members.

The web members are connected to the chords by vertical gusset plates. They are usually bolted to the chord webs and the web members fit between them (Figure 12a).

The chord joint is effected by providing cover plates. They should be so disposed, with respect to the cross-section of the member, as to transfer the load in proportion to the respective parts of the section (Figure 12b). The gusset plates form the external web cover plates. Since they work in the dual capacity of cover plate and web connector, their thickness takes this into account. The joint is designed to carry the coexistent load in the lesser loaded chord plus the horizontal component of the load in the adjacent diagonal. The load from the other diagonal is transferred to the more heavily loaded chord through the gussets alone. In compression chords which have fitting abutting ends in contact, the compressive load to be carried through the abutting ends of the splice is designed for a minor amount of the compression.

Sometimes the gusset is formed by shop-welding a thicker shaped plate to the chord in place of the chord web. The web members are then all narrower than the chords and the chord splice is offset from the node. An advantage occurs in erection as the web connections can be made before the next chord is erected.

At the connections of all tension members and elements, care has to be taken in the arrangement of bolt holes to ensure that the critical net section area of the section is not so small that fracture will govern. If necessary staggering the lines of bolts helps to increase the effective nett area. Remember that the critical net section is usually at the ends of the section or the centre of the cover plates, and that elsewhere some of the load has been transferred to the other parts of the joint and more bolt holes can be tolerated.

Connections of web members to gussets are quite straightforward and special treatment such as the use of lug angles is rarely required. In connecting rectangular hollow sections the method shown in Figure 12d is preferable to that of Figure 12c.

Unsupported edges of gussets should be such that the distance between connections does not exceed about 50 times the gusset plate thickness (Figure 12a). If this is unavoidable, the edge should be stiffened.

6.3 Cross Girder Connections

They are quite straightforward. The 2 or 4 rows of bolts in the cross girder end plate are made to correspond with the equivalent central rows of bolts in the gusset. Packing plates are required to accommodate the difference in height of gussets and cross girders (Figure 12e).

6.4 Lateral Bracing Connections

As recommended in 5.2(ii) the axes of the lateral systems should be in the same planes as those of the truss chords. This requirement is met in 2 of the 3 types of lateral members and connections described below:

i. For long and medium spans, the lateral members are frequently made from two rolled channel sections connected by lacing to give an overall depth the same as the chords. They are connected to the chords by gussets bolted to the chord flanges exactly as the main web members are connected to the main joint gussets.

ii. For medium spans, laterals consisting of two rolled angles arranged toe to toe in "star" formation and with intermediate battens are often ideal. They are connected to the chords by gussets positioned at the chord axis (Figure 12f). Note, angles "back-to-back", but separated by a small gap should never be used because of maintenance problems.

iii. On short spans single laterals often suffice. They can be connected by a gusset to the upper or lower chord flange, as the moments due to eccentricity are small.

7. CONCLUDING SUMMARY

- Trusses can be assembled from small pieces and are particularly advantageous where site access is difficult.
- Keep the configuration simple, using a minimum of members and connections.
- For long bridges continuous trusses may be the economic solution, but remember that least weight of steel does not necessarily mean least-cost.
- Fatigue effects have to be considered, particularly in continuous trusses.
- Trusses can be economic for spans of 30m to 200m.
- Avoid eccentricity of loading and connections to reduce secondary stresses.
- Configuration of members and careful arrangement of bolts at splices are particularly important.
- In case of welded connections, only use butt-welded connections in order to avoid fatigue effects. The weld penetration should be complete.
- Avoid potential corrosion areas. Remember birds will nest and roost in the most unlikely places!
- Select a non-participating lateral system.

8. REFERENCES

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ESDEP WG 15B
STRUCTURAL SYSTEMS: BRIDGES

Lecture 15B.6: Box Girder Bridges

OBJECTIVE/SCOPE

This lecture gives information on details and features particular to box girder bridges. The lecture is intended for engineers with some training.

PREREQUISITES

Lecture 8.5.1: Design of Box Girders
Lecture 8.5.2: Advanced Methods for Box Girder Bridges

RELATED LECTURES

Lecture 15B.1: Conceptual Choice
Lecture 15B.2: Actions on Bridges
Lecture 15B.3: Bridge Decks
Lecture 15B.10: Bridge Equipment
Lecture 15B.12: Introduction to Bridge Construction

SUMMARY

The history of box girder bridges is briefly reviewed. Their general design is discussed, examining economic span range, span-to-depth ratio, design of cross-section and selection of steel grade. Critical details are examined. The methods of analysis are summarised, with reference to the more detailed treatments in Lectures 8. Methods of erection are presented and the lecture concludes with a summary of the lessons that need to be learned from the box failures of the 1970's.

1. HISTORY

The nomenclature of the structural elements in a steel box girder is given in Figure 1 which shows, as an example, a single cell box girder with a composite concrete deck.

Until 1940 the structural possibilities for box girders were limited; structures had to be assembled from rolled sections, plates, and riveted connections.

Notwithstanding these limitations, the first box girder, the Britannia Bridge (1850) with main spans 152 m, Figure 2, served as a model of what could be achieved with innovative design.

The basic concept of using hollow sections was only occasionally repeated with rivetted construction.

- The Britannia Bridge was duplicated only once; in America. A box girder uses more material than a truss, and material was much more expensive than labour in those days.
- The tubular members of the Firth of Forth Bridge (1890) were a second exception.
- The railway bridge across the Oude Maas, Dordrecht, The Netherlands, has tubular riveted diagonals. Here a corrosion problem arose. Riveted boxes are not completely water tight. Humid air, sucked in, condensed and water collected at the bottom.

  Note: A similar mechanism could occur by porosity in single run welds, e.g. in troughs in orthotropic decks. However, it is not common practice to use a double run. The porosity is accepted as it is.

With the development of electric welding and precision flame cutting, the structural possibilities increased enormously. It is now possible to design large welded units in a more economical way, e.g. box girders, using the techniques similar to those of shipbuilding.

A box girder consists of:

- a concrete deck or an orthotropic steel deck as the top flange, and sometimes a combination of the two,
- a stiffened plate or a bracing as a bottom flange,
- webs, vertical or inclined,
- stiff diaphragms or bracings at the supports and lighter cross bracings between the supports at distances of about 2,5 times the construction depth, Figure 1.

This basic cross-section can now be found in many bridges:

The great torsional rigidity makes a box girder a particularly appropriate solution where the bridge is curved in the horizontal plane, Figures 3a and 3b. Many bridges on European highways may serve as examples. Launching as an erection method is then still possible as long as the curvature is constant.

In wide cross-sections the box is sometimes subdivided into cells, Figure 4a. In such structures the bottom flange is not very efficient.

Alternatives are:

- The three cell box is replaced by:
  two cells on the outside,
  a central "cell", consisting of cross braces connecting two outer cells and grids as deck plate, Figure 4b.

- Several separate smaller boxes, Figure 4c. The advantage is the smaller bottom flange; a disadvantage is the greater number of less effective webs and the loss of a great deal of torsional rigidity.

- The last step is self evident: the replacement of these separate boxes by welded I-sections, Figure 4d.

Strengthening and widening of existing bridges is an ever recurring problem. By its nature, a box girder offers excellent opportunities for reinforcement by prestressing or by additional plates welded to the bottom flanges.

So far only "closed" box girders have been discussed. However, a form of structure with great torsional rigidity has been known for a long time: the three dimensional truss. The stiffening girders of many early suspension bridges were sometimes made of a "box girder", with two, three or all walls consisting of plane trusses.

2. GENERAL DESIGN PRINCIPLES

2.1 Span

Box girders are suitable for longer spans than I-girders and allow larger span to depth ratios. The limits for competitiveness may vary due to local market conditions.

Steel or steel-concrete composite box girders are usually more expensive than plate girders because they require more fabrication time. They have, however several advantages over plate girders which make their use attractive:

- very high torsional rigidity: In closed box girders, torque is resisted mainly by Saint Venant shear stresses because the Saint Venant torsional stiffness is normally much greater than the torsional warping stiffness. For highly curved spans, this stiffness of the box girders is virtually essential during their construction, as well as under service loads. All steel box girders provide torsional stiffness during their erection. Composite box girders only achieve their torsional rigidity after concreting. During erection and concreting they may require expensive temporary bracing, which can also interfere with the execution of the concrete slab.

- very wide flanges allow large span-to-depth ratios.

- a neater appearance since the stiffening can remain invisible in the box.

- very good aerodynamic shape, which is equally important for large suspension or cable-stayed bridges as is the torsional stiffness.

- a very good adaptability to the most difficult conditions. Box girders are able to cross greater torsional spans than flexural spans using piers with a single bearing as shown in Figure 3.

Table 1 Span range for box girder bridges

	Composite concrete deck (m)	Orthotropic deck (m)
Simple span	20 - 100	70 - 120
Interior span of continuous girder	30 - 140	100 - 250

Table 1 indicates economic span limits for road bridges.

The longest span so far is 300 m achieved in 1974 by the Costa e Silva bridge in Rio de Janeiro. It is always probable that the longest span existing has passed the limit of best economy.

2.2 Span-to-Depth Ratio

The span-to-depth ratio will normally be around 20 to 25 for simple girders and around 25 to 35 for continuous girders. It is possible to reduce the depth, if necessary, without violating deflection limitations, at the expense of additional steel. The above ratios are valid for road bridges. For rail bridges the ratios should be smaller, say 15 and 20. It is advisable to check the most favourable span-to-depth ratio by trial designs.

2.3 Cross-section

A box girder may have vertical or inclined webs. It is cheaper to manufacture a girder with vertical webs. This section shape may be the best solution for a narrow road or a single track railroad.

A single narrow closed box girder can be positioned on the bridge centre line and completed with cantilever brackets (Figure 3b).

A combination of a wide deck on a short or medium span bridge favours inclined webs, Figure 1. For instance, a 13 m wide concrete deck without transverse prestressing requires a width of the box of 6 m at the top. If it were made with vertical webs the bottom flange would be much too wide to be efficient. Inclined webs reduces the width in a favourable way. Normally the webs are inclined 20 - 35 degrees from the vertical. In many cases inclined webs are chosen for aesthetic reasons.

There are several effects that make wide flanges inefficient. One is shear lag and another is local buckling of areas in compression. Further the minimum thickness specified in codes may often make the flange area excessive.

Plate widths of 3,3 m are readily available and some French or German mills produce wider plates, up to 5 m. (Greater widths are available with thinner plates.). If even wider plates are needed a longitudinal weld adds costs. In this case the longitudinal weld does not need to be a full penetration butt weld. It is generally preferable to adopt the maximum available width and avoid the longitudinal weld even if a slightly thicker plate would lead to less stiffening. This advice is valid for the bottom plate as well as the webs.

The best economy is achieved if sections can be fabricated in the full width at the shop. If the sections can be delivered by boat the only limitation is the handling equipment. Composite box girders will frequently be small enough to be shipped in one piece, also by road. Local restrictions for road transport should be checked. The normal width limit, 2,5 m, may be exceeded if special permits are requested, e.g. maximum of 4,5 m is allowed in Sweden. The costs of the escort should be checked.

2.4 Grade of Steel

The common steel grade for box girders is Fe 510 with a yield strength f_y = 360 MPa in the main structure and Fe 360 or Fe 430 for bracings. For long spans it is cost effective to use higher grades, e.g. f_y = 460 MPa.

Since the higher grade steels are now thermomechanically processed, their use may be economically attractive provided that fatigue is not governing.

3. STRUCTURAL DETAILS

This section deals only with details typical for box girders excluding the deck. For decks see Lecture 15B.3 and for plates in general see Lecture 8.5.2.

3.1 Longitudinal Stiffeners

Stiffeners are needed on the bottom flange at least at the piers where it is in compression and sometimes also on the webs. In designing an economical girder, the cost of handling and welding the stiffeners has to be taken into account. With increasing labour costs the tendency is to have fewer stiffeners and thicker plates. For instance it is common in Sweden not to use stiffeners on webs until the depth exceeds 2,5 - 2,8 m (1 man-hour equals 60 kg steel). National practice varies in this respect. Contractors also have their own preferences.

The bottom flange will in most cases have a very small effective area if it is not stiffened at the support. An efficient profile is the cold-formed trapezoidal stiffener. One to two will be sufficient if they are made big enough.

If the bridge is to be erected by launching or cantilevering, it is often necessary to stiffen the bottom flange along the whole girder in order to resist the hogging moments during erection.

3.2 Pier Diaphragms and Intermediate Cross Frames

At the supports considerable forces from torque and shear have to be transmitted to the bearings. The recommended solution at piers is a diaphragm, i.e. a steel plate transverse to the girder. The plate is designed to carry the shear from the torque and is strengthened locally in order to carry the support reactions. The diaphragm at the pier sections prevents deformation of the section (distortion of the box cross-section). If the bottom flange is narrow it may be necessary to put the bearings outside the flange and to provide the webs with external stiffeners.

In order to prevent cross-section distortion, the girder is provided with intermediate cross frames (see Figure 1). The webs and bottom flange have transverse stiffeners at these sections. The intermediate transverse stiffeners are made of flats or strips of plate when cross bracings are used. For the intermediate frames, the bracings can be omitted if the rigidity of the cross frames constituted by web and flange stiffeners is enough. The intermediate transverse stiffeners are made of a T section when bracings are not used.

3.3 Intermediate Transverse Elements Between Boxes

The design of the transverse elements between two longitudinal box girders are generally subject to important stress variations under eccentric live loads. Their design is generally governed by fatigue considerations. Large, widely spaced diaphrågms may be adapted, Figure 5a. Alternatively cross girders at 3 to 4m

spacings may be adopted that support a concrete or orthotropic steel deck as shown in Figure 5b.

3.4 Bearings

As a box girder is torsionally rigid it is possible to use a single bearing at one or more supports and to transmit the torque to where the foundations are suitable to resist it. This is particularly common if the bridge is highly curved. The single bearings may be supported by slender columns.

At each end of the bridge, there are generally two bearings if the bridge consists of a single box girder. Special attention has to be paid in this case to ensure sufficient distance between the two bearings.

Another consequence of the torsional rigidity is that extra care has to be taken to get the correct support reactions when there are two bearings at each support. One way of doing this is to let the box rest on jacks with predetermined loads and to fix the permanent bearing when the jack loads are correct.

If the bearings on the pier are under the diaphragm, care must be taken to ensure that thermal moments do not lead to longitudinal eccentricity occurring at the bearings. Additional stiffeners may need to be provided.

3.5 Corrosion Protection

The interior of a box girder is exposed to far less risk of corrosion than the outside. Hence, the interior corrosion protection can be made simpler or even omitted completely. There is always a possibility of water leaking into the box, especially if the deck is made of concrete. For this reason the box should be equipped with a dehumidifier to keep the air dry. This is an inexpensive precaution.

White painting or very light colours should be used for the interior to facilite future inspections.

4. ANALYSIS

4.1 General

A box girder may be analysed as a beam subjected to bending, shear and torsion. Simple beam theory is however not an adequate tool and additional considerations are required, e.g. shear lag, warping and distortion of the cross-section [1]. For details, see Lecture 8.5.1.

The additional stresses caused by cross-section distortion depend largely on the distance between the cross braces. With a sufficiently small distance these stresses may be neglected. National practice varies on this point.

4.2 Torsion

A torque is primarily carried by shear stresses corresponding to the theory of pure torsion. These stresses are readily calculated from the assumption of a constant shear flow in a single cell box. In addition restrained warping changes the distribution of shear stresses slightly and, more important, gives rise to longitudinal stresses which add to the bending stress. The stresses due to restrained warping are not very large and an approximate estimate is sufficient, see Lecture 8.5.2.

4.3 Braced or Unbraced Intermediate Cross Frames

The cross bracings act to restrain cross-section deformation. The loads on them arise from eccentric loading and the loads can easily be calculated if the cross-section is assumed to have zero stiffness for deformation in its own plane or, an equivalent assumption, hinges are assumed in the corners. The loads are assumed to be carried by flanges and webs acting as beams with rigid supports at the cross braces. The support reaction from those fictitious beams are the forces that are carried by the cross braces. For details see Lecture 8.5.2.

When intermediate bracings are omitted, special attention should be given to the design of the corners of the unbraced cross frames which should resist the bending moment in the plane of the cross frame. (The steel box girder works in the same way as pre-stressed concrete box girders). In this case web and flange T stiffeners are designed for this purpose. The design of the corners is generally governed by fatigue considerations.

When the intermediate cross frames do not support traffic load directly, they are in general lightly stressed.

5. ERECTION METHODS

Box girders may be erected with normal methods such as launching or cantilevering. If the bridge is curved in a circle, launching works without complication. If the box has an orthotropic deck it is rigid enough even for highly curved bridges. However, boxes with composite concrete slabs are normally erected as an open trough. This open shape is torsionally very soft. The shear centre is unusually far under the centre of gravity, so that the section will deflect substantially, vertically as well as horizontally, under selfweight complicating the launching. Further, the casting of the concrete slab creates additional eccentric load and further deformations and stresses if the box is curved and open.

One solution is to provide the box with horizontal bracings between the top flanges. The bracings must be designed to avoid interference with the casting of the concrete slab. These diagonals may be temporary if it is deemed worthwhile to remove them after casting the slab. Another possibility is to use lost shuttering.

An ingenious system was used for the erection of the Pont de Martigue, Figure 6. The girder was fabricated in three parts. The two end parts were placed below the abutments and the third part mounted in between. Next the stiffening girder was lifted by two portal cranes, the legs mounted and the leg to girder connections made. The gaps between the sloping legs and the girder were closed, using ballast and the portal cranes.

6. LEARNING FROM FAILURES

Structural failures occur as a result of human failures. Moreover human failures have the inclination to repeat themselves.

During the period 1969-1971, several accidents happened with box girder bridges, all during the erection stage:

- 1969: bridge across the Danube, Vienna
- 2 June, 1970: bridge across the Cleddau, Milford Haven, Wales
- 15 Oct, 1970: bridge across the Lower Yarra, Melbourne, Australia
- 10 Nov, '971: bridge across the Rhine, Koblenz, Germany.

These four cases are briefly discussed below:

Vienna

The erection of this bridge proceeded without problems by cantilevering from both sides. The final gap was closed on a hot summer day. The deformations of the bridge due to temperature expansion are shown in Figure 7. During the night an evenly distributed temperature was restored. The bridge straightened, leading to plate buckling. The buckling was corrected and no collapse occurred.

Milford Haven

The midspan of this bridge was erected by cantilevering. With this method of erection the cross frame above a pier suffers extra loading due to the cantilevering part. This load causes no problem provided the diaphragm is designed to carry it. This was not the case. The bridge collapsed, Figure 8.

Melbourne

The stiffening girder of this cable stayed bridge consisted of three cells. For erection, the box was divided into two parts longitudinally, Figure 9. On each side of a pier one part of the box was assembled, hoisted to the correct level and shifted into the correct position to be connected.

Some complications:

- Both parts were asymmetric. Vertical and horizontal deflection were to be expected due to dead load.

- It is practically impossible to assemble both parts independently in such a way that the sag is exactly the same. An actual difference of 120 mm was measured.

- The laterally cantilevering top flange is strongly inclined to buckle.

The last two problems were solved by putting ballast on top of the bridge. The difference in overall deflection disappeared but buckling of the cantilevering top flange increased. To solve the problem of the final buckle, some high strength friction grip bolts were taken out to remove the incompatibility in flange length,

with the disastrous result of passing the ultimate load carrying resistance. The bridge yielded and collapsed 50 minutes later [2].

Koblenz

Cantilevering was used as the erection method and again a collapse occurred. The failure was due to the coincidence of three unfavourable aspects, each of which separately would, most probably, not have caused the collapse.

- due to welding of the cross weld, a deformation was introduced, increasing the eccentricity of the compressive stress, Figure 10a.

- a gap of about 460 mm was kept free between the longitudinal stiffeners, so that automatic welding equipment could pass the stiffeners without stopping, Figure 10b. The buckling length was taken as 460 mm. The effective buckling length was larger, Figure 10c.

- The effect of effective width on the locally unstiffened plate was not taken into account.

The accidents in the United Kingdom particularly led to a rigorous investigation programme [3]. In 1974, the 'Merrison Rules' were issued, a code giving recommendations on calculation and erection of box girders [4].

7. CONCLUDING SUMMARY

- Box girder bridges are suitable for longer spans than I-girder bridges and they are particularly efficient for curved bridges.
- Box girder bridges are able to cross greater torsional spans than flexural spans using piers with a single intermediate bearing.
- The deck may be a composite concrete slab or an orthotropic steel deck. The latter is suitable for longer spans than the former.
- The economical depth of a box girder is smaller than that of a plate girder.
- When designing the cross-section, available plate widths, transport and erection should be considered.
- Boxes are torsionally very rigid when completed. If erected as an open trough they are very flexible which may cause problems.
- Strong diaphragms should be used on piers.
- Longitudinal stiffeners should not be interrupted.

8. REFERENCES

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ESDEP WG 15B
STRUCTURAL SYSTEMS: BRIDGES

Lecture 15B.8: Cable-Stayed Bridges

OBJECTIVE/SCOPE

To describe the main features of contemporary cable-stayed bridges in steel and to present guidelines for their design and analysis.

PREREQUISITES

None

RELATED LECTURES

Lecture 15B.1:	Conceptual Choice
Lecture 15B.2:	Actions on Bridges
Lecture 15B.3:	Bridge Decks
Lecture 15B.4:	Plate Girder and Beam Bridges
Lecture 15B.6:	Box Girder Bridges
Lecture 15B.9:	Suspension Bridges

SUMMARY

The continuous development of the cable stayed bridge since the 1950's is described and the arrangement of the stay cables, the supporting conditions for the girder, the cable planes and type of girder are introduced. The choice of elements is discussed and special aspects of behaviour and analysis are described. The special connections required are also described and special features of construction are given.

1. INTRODUCTION

Cable-stayed bridges have been built for centuries but up to the 1950's they had not been developed to the same extent as other bridge types, such as truss bridges, arch bridges and suspension bridges.

However, since the completion of the Strömsund Bridge in 1955, the cable-stayed bridge has been continuously developed. It has appeared in a larger number of variants than any other bridge type during this period.

The cable-stayed bridge is mainly used for road bridges, where it is applicable for both narrow 2-lane roads and for wide 6 or 8 lane motorways.

Another application is within the field of pedestrian bridges where cable-stayed bridges can prove advantageous also for smaller spans.

Finally, cable-stayed bridges have been designed to carry railway lines, in a few cases.

The cable-stayed bridge has been used for a span range from approximately 150m to 400m, where it has proved to be very competitive against truss bridges, arch bridges and box girder bridges. Recently, the cable-stayed bridge has started to increase its span range up to almost 900m, i.e. moving into a span range that previously has been entirely in the domain of suspension bridges.

The impressive development of cable-stayed bridges is reflected in Table 1 which shows the largest cable-stayed bridges built from 1955 to 1993.

2. TYPES

The cable-stayed bridge consists of the bridge girder, the stay cables and the pylons (Figure 1).

2.1 Arrangement of Stay Cables

For the system of stay cables, two main configurations are generally found: the fan system, Figure 1a, and the harp system Figure 1c.

The fan configuration leads to the most efficient structural system as it is entirely composed of triangles. In contrast, the harp system contains mainly quadrangles, and, therefore, an additional bending stiffness of the girder or the pylon is required to carry a non-uniform load.

In the pure fan system, all stay cables radiate from the pylon top as indicated for System (a). It will, however, in many cases be complicated to anchor all the cables at one point at the pylon top. To avoid this difficulty, the fan system is often modified so that the cable anchors at the pylon top are spread over a certain height, as shown for System (b).

Provided that the cable anchors are concentrated in a relatively narrow zone at the pylon top, there is no significant difference between the behaviour of bridges with the pure fan or with the modified fan.

The efficiency of the harp system can be significantly improved by adding intermediate supports in the side spans, as indicated by dotted lines in System (c) of Figure 1.

In modern cable-stayed bridges, the cable system is generally of the multi-cable configuration where each stay consists of a mono-strand prefabricated in full length and full cross-section. To achieve this arrangement, it is necessary to have the stay cables closely spaced. The distance between the cable attachments at the girder is therefore often chosen between 10 and 20 m.

2.2 Supporting Conditions for the Girder

Cable-stayed bridges are generally built as self-anchored systems where the supporting conditions are chosen so that vertical load from the self-weight and the traffic introduces vertical reactions only.

This loading can be achieved by supporting the stiffening girder on one fixed bearing and three longitudinally movable bearings, as indicated on Figure 2a for a system with the pylons fixed to the stiffening girder.

There are, however, many variants to this basic system and in some systems, horizontal reactions of moderate size might occur due to compatibility phenomena.

For example, Figure 2b shows a system with a fixed end bearing and pylons that are rigidly connected to the substructure.

For this system, the elongation of the left anchor cable will force the top of the left pylon to deflect in the longitudinal direction introducing bending and horizontal shear. In addition, the contraction of the girder, e.g. under traffic load, will result in a longitudinal displacement of the movable end bearing to the right and this displacement will partly be transmitted to the top of the right pylon by the right anchor cable.

It should also be noted that elongations or contractions of the girder due to temperature effects will introduce displacements of the pylon tops, and thereby horizontal shear in the pylon legs.

In some cable-stayed bridges, the girder will have direct vertical supports only at the end piers whereas it will be free to move vertically at the pylons. With this system, a total symmetry under temperature variations can be achieved if both end bearings are longitudinally movable, as indicated on Figure 2c. With this system longitudinal forces, e.g. from braking, will have to be transferred to the soil by bending of the pylons. This system is, therefore, only applicable if the longitudinal forces are of moderate intensity.

2.3 Position of Cable Planes and Type of Girder

In cross-section the cable system is generally arranged in one vertical plane above the centre line, in two vertical planes at the edges of the girder or in two inclined cable planes - as shown in Figure 3.

A central cable plane with the stay cables attached along the girder axis provide (elastic) vertical support to the girder, but no torsional support. It is, therefore, essential that the girder has a sufficient torsional stiffness to transmit any twisting moment from a load with an eccentric resultant, e.g. traffic load in only one carriageway.

To achieve the required torsional stiffness, the girder will have to be of the box type, Figure 3a.

With two vertical cable planes attached along the edges of the girder, both vertical and torsional support is provided by the cable system and it is therefore not required that the girder in itself possesses torsional stiffness.

The girder can simply consist of two I-shaped plate girders directly under the cable planes Figure 3b.

With two inclined cable planes intersecting at the top of the pylon, the girder in principle gets the same cable support as with two vertical cable planes. In this case a girder with torsional stiffness is also not required.

In cable-stayed bridges with very long spans, where the torsional stiffness becomes essential to achieve aerodynamic stability, it is often advantageous to have a box girder combined with two cable planes, and also to give the girder a favourable streamlined shape, as illustrated in Figure 3c. It should, however, be emphasized that a lay-out such as that of Figure 3c is only required for very long spans (above 500m) or for small width-to-span ratios (below 1/25).

3. CHOICE OF ELEMENTS

3.1 Stay Cable

The main advantage of applying a cable support to bridges is linked to the fact that cable steel can be manufactured with a much higher strength than structural steel.

For cold drawn steel wires with a diameter of 5-7mm, an ultimate strength of 1600 MPa is easily achieved, whereas ordinary structural steel has ultimate strength of 350-500 MPa. In other words, cable steel is 3-4 times stronger than ordinary structural steel. This difference implies that an element under pure tension, if made of cable steel, will have a cross-section (and a weight) that is only 25-33% of that required with structural steel.

Each stay cable is composed of a large number of wires, either with a circular shape and diameters between 5 and 7mm or with a special shape to give a higher degree of compaction and a more dense surface.

In the so-called locked coil cable, Figure 4, the outer layers are composed of Z-shaped wires that fit tightly together, whereas the inner wire layers are cylindrical. All layers are helical with the direction of helix changing from one layer to the next.

Due to the twisting of the wires, the locked coil cable becomes self-compacting, so that a wrapping is unnecessary. At the same time the interlocking Z-wires of the outer layers ensure a tight surface of the cable under tension, and the required corrosion resistance can therefore often be achieved just by galvanizing the wires.

In helical cables the axial stiffness is influenced by the twisting of the wires and the modulus of elasticity is therefore reduced by 15-25% to a typical value of 170 $\times 10^3$ MPa.

The twisting of the wires also slightly influences the fatigue strength, so that the stress range endured by the cable is smaller than for the wires themselves.

In the other type of stay cable, the parallel wire strand (PWS), the drawbacks of the helical strand are eliminated by having all wires parallel and straight (or twisted with a very long lay corresponding to a twist angle of less than 3°).

With parallel and straight wires the cable is without a self compacting effect. A special wrapping is, therefore, required to keep the wire bundle together and establish the necessary corrosion protection.

In the early cable-stayed bridges with parallel wire strands (PWS), the wires were generally blank (ungalvanized) and the corrosion protection established by placing the wires inside a polyethylene tube that was injected with cement grout after installation of the stay cable.

In the more recent developments the PWS is composed of galvanized wires and the cement grout is substituted by a corrosion inhibiting compound or the tube is extruded directly onto the wire bundle, Figure 5.

3.2 Girder

In steel bridges, the girder is composed of stiffened steel panels, as illustrated for a box girder in Figure 6.

The deck plate is typically 12-14mm thick and stiffened by longitudinal ribs giving support along lines a distance of 300mm apart, i.e. with trapezoidal ribs each attached along two lines the distance between the centres of the ribs is 600mm.

The longitudinal ribs are supported by cross beams or diaphragms spaced 2,5-4m apart, and these transverse elements are finally attached to the main girder. Thus the transfer of concentrated wheel loads acting on the bridge deck to the main girder induces plate bending in the deck plate, and bending plus shear in both the longitudinal ribs and the cross beams or diaphragms. This results in a rather complicated biaxial stress distribution in the deck plate. To determine this distribution, the local structural system should be modelled as an orthogonal grid of beam elements.

The webs and the bottom plate of the box girder also have to be stiffened by longitudinal and transverse ribs. In this case the main purpose of the stiffeners is to prevent buckling - a phenomenon that is especially important to consider as the girder forms an important part of the primary structural system by transmitting in compression the horizontal components of the stay cable forces.

Full diaphragms, either plated or braced, are generally positioned at all cable anchor points and at the pylons, whereas the intermediate transverse elements can be composed of relatively shallow plate girders.

For a girder with an open cross-section and a concrete deck, an efficient structural system can be achieved by applying plate girders directly under the cable planes and interconnecting these main girders by transverse girders at intervals of 3-5m (Figure 7).

With this system, the transverse girders are subjected to positive moments over the entire length so that they can fully benefit from acting compositely with the concrete slab. Similarly the composite action is also favourable for the longitudinal girders, being subjected to compression by the horizontal components of the stay cable force.

Both the main girders and the transverse girders therefore, have shear studs on their top flanges.

3.3 Pylon

The configuration of the pylon is closely related to the lay-out of the cable system, as the main function of the pylon is to support the stay cables.

In bridges with a central cable plane the pylon can be designed as a free standing column or as a lambda-shaped frame, as shown in Figure 8a.

The free standing vertical pylon at the centre of the bridge deck is well suited to support both a harp-shaped and a fan-shaped cable system, whereas the lambda pylon requires a modified fan system.

The vertical pylon must have a rigid moment connection to either the box shaped main girder, or to the bridge pier, to be stable in the lateral direction.

The lambda pylon has in most cases its inclined legs passing outside the girder without a direct connection.

In bridges with two vertical cable planes, the pylon can either consist of two vertical columns or form a portal frame, as shown on Figure 8b. Regarding supporting conditions at the bottom and lay-out of the cable system, the double pylon under (b) closely corresponds to the matching single pylon under (a).

With two inclined cable planes, the pylon is A-shaped in most cases, Figure 8c, in combination with a modified fan system. Other combinations are theoretically possible.

The cross-section of the pylon generally forms a rectangular box with a single cell. Due to the dominating compression it is necessary to stiffen the side plates primarily with longitudinal stiffeners, as shown in Figure 9.

Transverse diaphragms are required to support the longitudinal stiffeners at certain intervals. Due to the fact that very little torsion is applied to the pylon, the diaphragms do not need to be very rigid. They can therefore be made with relatively large openings (man holes) to ease inspection and maintenance.

At the cable anchor zones, it may be necessary to add more robust horizontal diaphragms and/or vertical bulkheads to ensure the transmission of the stay cable forces to the pylon cross-section and to the stay cables in the opposite side.

4. SPECIAL ASPECTS OF BEHAVIOUR AND ANALYSIS

For the design of the structural elements in a cable-stayed bridge it is sufficient in most cases to use a standard two- or three-dimensional frame analysis program.

For bridges with only one, central cable plane, a two-dimensional structural system is adequate to analyse the structure under vertical loading due to dead load and traffic load. This analysis gives the forces in the stay cables, and the axial forces, shear forces and bending moments in the girder and the pylons.

For the torsion induced in the girder under one-sided traffic load, the girder can be analyzed subsequently without taking the cable system and the pylon into account. The same type of analysis applies to the girder under lateral load, e.g. wind load.

In bridges with two cable planes the mathematical model has to be three-dimensional.

For a bridge with two main girders, see Figure 3b, the mathematical model should be as indicated in Figure 10a, i.e. with two longitudinal beam elements to model the main girders and a large number of transverse beam elements to model the transverse girders. The stay cables can in most cases be modelled as straight bars carrying pure tension.

For bridges with a single box shaped girder and two cable planes, as illustrated in Figure 3a, the mathematical model should comprise a single longitudinal girder with the flexural and torsional stiffness of the box shaped main girder. At the points of cable attachment the central beam should be joined to transverse beam elements, as illustrated in Figure 10(b).

In the analysis, the stay cable is in most cases regarded as a straight member subjected to pure tension. In reality an inclined cable is always slightly curved due to the action of the cable's own weight.

For long stay cables, the sag, or rather the sag variations, tends to reduce the axial stiffness as the elongation is due not only to the elastic strains in the cable wires but also to a reduction of the sag, as illustrated in Figure 11.

For stay cables with horizontal projections up to 150m and moderate stress variations, the sag effect can generally be disregarded, but for longer cables the stiffness of the cable support is overestimated if the axial strain is only considered.

The sag effect can be taken into account in the analysis by substituting the real modulus of elasticity E of the cable material by an equivalent modulus of elasticity E_{eq} determined by:

$$E_{eq} = \frac{E}{1 + \frac{\gamma^2 a^2}{24}\left[\frac{\sigma_1 + \sigma_2}{\sigma_1^2\,\sigma_2^2}\right]E} \tag{1}$$

where

γ is the density of the cable material
a the horizontal projection of the cable
σ_1 is the initial dead load, stress
σ_2 the final stress (from dead + live load)

Equation (1) which introduces a secant modulus, requires an iteration in the structural analysis, as the final stress σ_2 is unknown from the beginning.

For stay cables with moderate stress variations and horizontal projections up to 250-300m, it is allowable to substitute the secant modulus of Equation (1) by a tangent modulus E_{tan} derived simply by inserting $\sigma_2 = \sigma_1$ in Equation (1):

$$E_{tan} = \frac{E}{1 + \frac{\gamma^2 a^2 E}{12\sigma_1^3}} \tag{2}$$

In this case an iteration can be avoided as the tangent modulus depends only on the initial stress σ_1.

When applying the more exact secant modulus approach, the first step in the iteration is often based on axial cable stiffnesses found from Equation (2), whereas the following steps are based on stiffnesses determined from Equation (1).

The pylon is subjected primarily to compression from the vertical components of the cable forces. For this reason it is very important to consider column buckling when designing the pylon.

It is, therefore, essential to determine thoroughly the effective length of the column to be applied in the actual case. To illustrate this feature, three examples are shown in Figure 12.

Figure 12a shows the lateral buckling of a free standing pylon supporting a pure fan system at the top. The column length l_c is equal to the pylon height h due to the fact that the cable plane rotates with the anchor point at the pylon top so that the resultant of the stay cable forces still points towards the bridge axis.

If this effect had been neglected, then the column length of the free standing pylon would have been stipulated at twice the pylon height, i.e. a much more severe condition.

Figure 12b shows the buckling of a pylon in the longitudinal direction in a system where the girder is longitudinally supported at the end pier. In this case the anchor cable leading from the fixed bearing to the pylon top restrains the pylon top in the longitudinal direction and the column effective length in the plane is therefore close to 0,7 times the pylon height.

Finally, Figure 12c shows the buckling of a pylon in a system where the girder has no longitudinal restraint through fixed bearings. In this case, the buckling is accompanied by a longitudinal sway of the girder so the cable reaction remains vertical. The column length is, consequently, twice the pylon height.

5. CONNECTIONS

In cable-stayed bridges, special connections are required to allow the transmission of the cable forces to the girder and the pylon.

Due to the fact that the high strength of the wires is achieved by a carbon content approximately five times larger than in normal structural steel, the wires cannot be welded.

Instead sockets are fixed to the ends of the wire bundle constituting the stay cable.

The sockets are made of cast steel in the form of a short cylinder with a conical cavity, Figure 13. Inside this cavity the strand is broomed and subsequently the space is filled with a metallic zinc alloy or a mixture of epoxy resin, zinc dust and steel balls.

The transmission of the cable force from the socket to the adjacent structure is established as contact pressure on the end face or through an external thread to a nut. The socket might also have a shape that allows a pin connection.

During erection the length of the stay cables has to be adjusted either by inserting shims between the socket and the supporting structure or by turning the nut.

To vary the length of the stay cable, it is only necessary to make adjustments at one end (at the active anchorage). Thus the other end can be made without provision for adjustments (the passive anchorage).

The active anchorage can be positioned either at the girder or at the pylon. The choice between these two possibilities depends on the accessibility in the actual locations.

In modern cable-stayed bridges with stay cables made of mono-strands it is generally a requirement that the stay cables can be replaced in the event of corrosion or fatigue leading to wire breaks. The anchorage detail should, therefore, also allow the stay cable to be released and removed in the service state.

When designing the cable anchor point at the girder, it is necessary to consider thoroughly the transmission of both the vertical and the horizontal component of the cable force.

Throughout the history of cable-stayed bridges a very large number of anchorage details have been developed for the special conditions at the girder level.

As an example, Figure 14 shows a simple solution for anchoring a central cable to a box girder. Here the cable force is transferred from the socket through a bearing plate to two gusset plates welded to the deck plate and to a plated diaphragm. Thus, the horizontal component of the cable force is transferred to the deck plate by shear in the longitudinal weld and the vertical component to the diaphragm by shear in the vertical weld.

It is emphasized that the deck plate and the diaphragm must intersect exactly at the cable axis in order to exclude moments due to eccentricity.

At the pylon, the stay cables might be anchored to inclined, secondary diaphragms extending between the two longitudinal side plates of the pylon, as illustrated in Figure 15 for a pylon supporting a modified fan.

6. SPECIAL FEATURES OF CONSTRUCTION

The success of the cable-stayed bridge is to a large extent linked to the efficient erection procedure which characterizes this type of bridge. Thus, a cable-stayed bridge can be erected by free cantilevering from the pylon, either symmetrically in both directions (Figure 16a) or only into the main span (Figure 16b). In the latter case, the side span is erected initially as a normal girder bridge.

With the double cantilevering, Figure 16a, it must be remembered that the entire stability in the temporary stage depends on the flexural stiffness and the fixity of the pylon. In some cases this stiffness governs the design of the pylon.

With cantilevering only into the main span (Figure 16b) the stay cables are generally installed in pairs so that the fan (or harp) of the side span is established simultaneously with that of the main span.

Typically an erection sequence comprises the following steps:

1. Cantilevering the girder from one cable anchor point to the next - in most cases achieved by lifting girder units by a derrick crane positioned on the bridge deck.

2. Installation of the stay cable, often performed by unreeling a prefabricated strand from a reel positioned on the bridge deck.

3. Controlled tensioning of the stay cable by jacking at the active anchorage.

4. Moving the crane to the tip of the girder.

In many cases the stay cable is subjected to its maximum tension after cantilevering the girder to the next cable anchor point. Subsequently, the tension is relieved when the following stay cables are being tensioned.

It is of the utmost importance to realize that the distribution of dead load moments in the girder is entirely governed by the tensioning of the stay cables during erection. An optimum distribution of dead load moments can therefore be achieved by choosing the initial cable tension accordingly.

The required analysis of the erection stages may conveniently be carried out "backwards", i.e. by initially choosing a desired distribution of dead load moments and then "moving backwards" by "demolishing" the structure in the same stages as assumed for the erection.

To determine the dead load moments by subjecting the final structure to the dead load of the structural elements is not only erroneous, but is also very uneconomical in most cases.

7. CONCLUDING SUMMARY

- Modern cable-stayed bridges cover a span range from approximately 150m to 900m (for road bridges).
- The cable system comprises straight cables in a fan or a harp configuration.
- The girder cross-section is chosen taking into account the support offered by the cable system. With only one central cable plane the girder must possess considerable torsional stiffness.
- The spacing of the stay cables should be chosen so that each stay can consist of a single strand (mono-strand cables).
- For stay cables with a horizontal projection of more than 150m, the non-linear sag effect has to be taken into account in the analysis.
- The distribution of dead load moments in the girder should be determined by considering the initial tensioning of the stay cables during erection.

8. ADDITIONAL READING

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4. Roik, K.; Albrecht, G.; Weyer, U.: Schrägseibrücken, W. Ernst & Sohn, Berlin, 1986.

5. Troitsky, M.S.; Cable-Stayed Bridges: Theory and Design, BSP Professional Books, Oxford, 1988.

YEAR	SPAN M	NAME	COUNTRY	GIRDER MATERIAL AT MID-SPAN
1955	183	Strömsund Bridge	Sweden	Steel
1957	260	Theodor Heuss Bridge	Germany	Steel
1961	302	Severins Bridge	Germany	Steel
1969	319	Knie Bridge	Germany	Steel
1970	350	Duisburg-Neuenkamp Bridge	Germany	Steel
1975	404	St. Nazaire Bridge	France	Steel
1983	440	Barrios de Luna Bridge	Spain	Concrete
1986	465	Alex Fraser Bridge	Canada	Composite
1991	490	Iguchi Bridge	Japan	Steel
1992	530	Kvarnsund Bridge	Norway	Concrete
1995	856	Normandy Bridge	France	Steel
1999	890	Tatara Bridge	Japan	Steel

Table 1 Evolution of span length for cable-stayed bridges

ESDEP WG 15B
STRUCTURAL SYSTEMS: BRIDGES

Lecture 15B.9: Suspension Bridges

OBJECTIVE/SCOPE

To introduce the main concepts and definitions concerning suspension bridges.

PREREQUISITES

None

RELATED LECTURES

Lecture 4A.4:	Corrosion Protection of Bridges
Lecture 15B.1:	Conceptual Choice
Lecture 15B.3:	Bridge Decks
Lecture 15B.4:	Plate Girder and Beam Bridges
Lecture 15B.5:	Truss Bridges
Lecture 15B.6:	Box Girder Bridges
Lecture 15B.8:	Cable-Stayed Bridges

SUMMARY

The lecture begins with a historical introduction to suspension bridges with emphasis on the specific 'jumps' in the development of their spans. The various types of suspension bridge are discussed and their main parts are separately presented, i.e. main cables, hangers, pylons and stiffening girders.

The influence of temperature and aerodynamic excitation are introduced. Finally, some notes on structural analysis and erection are given.

1. INTRODUCTION

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Generally, cable supported bridges can be divided mainly in two groups, cable-stayed bridges (see Lecture ~~15B.7~~) and suspension bridges. Their use leads to a competitive solution for spans between 200m and 1500m (and beyond). They cover therefore the major part of the present span range of bridges. These two groups, although similar in philosophy, have many differences in practice. One of the main reasons for their superiority in relation to other forms of bridge is the most efficient way in which they use materials, i.e. direct stress under which all the fibres have the same stress resulting in full utilization of the material.

In this lecture the conventional suspension bridge is discussed. Relevant nomenclature is given in Figure 1.

Since the beginning of the 19th century, suspension bridges have gradually grown in size, and since 1930 the suspension bridge has completely dominated the upper span range.

The following table indicates the development of spans:

Bridge	Place	Year	Mid span (m)
Menia Straits Bridge	Wales	1826	168
Clifton Suspension Bridge	England	1860	214
Cincinnatti and Covington Suspension Bridge	USA	1865	322
Brooklyn Bridge	USA	1883	480
Philadelphia-Camden Bridge	USA	1926	535
George Washington Bridge	USA	1931	1066
Golden Gate Bridge	USA	1937	1280
Tacoma Narrows Bridge	USA	1940	853
Mackinac	USA	1958	1158
Firth of Forth	Scotland	1964	1002
Severn Bridge	England	1966	988
Humber Bridge	England	1981	1410
Great Belt East Bridge	Denmark	1997*	1624
Akaski Kaikyo Bridge (Figure 2)	Japan	1998*	1990

* under construction

From this table two specific 'jumps' in the development of spans can be distinguished:

- 1883 The Brooklyn bridge is 50% longer than any suspension bridge made previously.
- 1931 The George Washington Bridge which was virtually double the free span of existing bridges.

As the design of suspension bridges continued to develop, the girder became lighter and lighter, the main girder torsionally weaker and weaker and designers more and more audacious.

A rude awakening came in November 1940 when the Tacoma Narrows Bridge collapsed due to wind effects.

Since then, the progress that has been made has been due to:

a. New concepts, e.g. orthotropic decks.

b. New methods of analysis, e.g. electronic computers.

c. New materials, e.g. high-strength steel and concrete.

d. New fabrication and erection methods (bolting, welding, mass production, modern equipment, etc).

e. Better quality control.

f. Better understanding of wind and seismic forces, e.g. derived from wind tunnel tests, etc.

The majority of all suspension bridges carry road traffic only, but a limited number carry rail traffic as well.

In more special applications, the suspension system is also used for pedestrian bridges and for pipeline bridges.

2. TYPES

Suspension bridges may be subdivided according to several criteria:

a. **Suspension of the girder:**

- Both the main span and the side spans are suspended: the S-type, Figure 3(a)
- Only the main span is suspended and, the end spans are substituted by independent approach spans: the F-type, Figure 3(b).

  For the F-type, the back stays have a slope corresponding to $\alpha = 30 - 45°$. This slope determines the position of the anchorage and with straight back stays the tower top is well supported against horizontal displacements. As compared with the S-type, with 2 hinges (code: S2), the F2-type is stiffer.

b. **Anchorage of the main cable:**

- In present practice, all major suspension bridges are earth anchored with separate anchorages allowing the main cable forces to be transmitted to the soil.

c. **Position of expansion joints (Figure 3(c) and (d))**

The stiffening girder can have its expansion joints positioned at the pylons or at the anchor blocks.

Corresponding to these two conditions the stiffening girder will either consist of three individual girders or one continuous girder.

3. CHOICE OF ELEMENTS

3.1 The Main Cables

The main cables run from anchorage to anchorage.

A cable is composed of a number of strands. These strands can be made in situ by cable spinning or they are prefabricated.

The number of strands builds up in a simple arithmetric progression. The cable thus consists of a central strand surrounded by 6 strands, 6 + 12 strands, 6 + 12 + 18 strands, etc.

The cable in Figure 4 contains 37 strands and this is a common number in bridges with spun cables. (The numbers indicate the sequence of spanning the individual strands.) Thus, in the Humber Bridge the main cables are composed of 37 strands: each having 404 wires.

In bridges with main cables made of parallel wire strands, the number of strands is generally higher as each strand has to contain fewer wires for reasons of handling.

Before starting the main cable erection a catwalk is made, running from anchorage to anchorage via the pylon tops. A tramway is mounted on this catwalk if the cables are to be spun. The tramway is not necessarily required for the transport of prefabricated strands. To protect the catwalk to some extend against the wind, a pretensioning system of counter parabolas is used.

The essentials of the cable spinning process are shown in Figure 5. The catwalk and tramway are indicated. Two spinning wheels, connected to a loop, carry separate wires from one side to the other. At one of the anchor blocks the dead wire is fixed. On the other side the wire is taken off the spinning wheel and looped around a strand shoe (Figure 6). Next the live wire is fixed. When a strand, consisting of 300 to 400 wires, is finished, this strand is located carefully in its correct place. This operation is carried out at night when the temperature is nearest to uniform.

The optimum wire diameter is 5,0-5,5mm. A larger diameter makes the wire too stiff, while a smaller diameter requires more wires and more labour. The wire material has an ultimate strength up to 1600 - 1800N/mm^2.

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With ~~spen~~ spun cables, each working shift can mount about 160 wires a day, and in most cases two shifts per day per cable is used. Prefabricated strands can be mounted in a shorter period.

After having erected all wires the bundle of strands are compacted into a circular shape. As a protective treatment, the cable is finally wrapped with a mild steel wire. During this wrapping a protective paint is added.

Where the cable passes over the pylon tops and the splay saddles, they lose their circular shape. Special provision is necessary at these locations to keep corrosion under control.

3.2 Pylons

The striking developments in pylon design are best presented by considering some milestones:

- The Golden Gate Bridge (Figure 7)
    - high strength steel is used in the outer cells to carry the bending moments.
    - a substantial part of the material is concentrated at the centre.
- The Firth of Forth Bridge (Figure 8)
    - three separate box sections, connected during erection.
    - the material distribution is much better.
- The Severn Bridge (Figure 9)
    - all material is located in the right place, at the edges.
    - the connections are designed to be completed from the inside.
    - the dimensions are considerably reduced.
- Pont de Tancarville.

    Concrete is the ideal material to carry compressive loads. Consequently it is natural to use concrete pylons. The Pont de Tancarville Bridge was the first suspension bridge with concrete pylons. Concrete pylons with heights of 250 m are used in the Great Belt East Bridge. However, in earthquake areas steel pylons are still preferred because of their capability for energy dissipation.

It should be considered that there are many factors influencing the choice of the material for pylons, e.g. soil conditions, speed of erection, stability during their construction, etc. Therefore, the choice should not be based entirely on a quantity-based cost estimate.

3.3 Stiffening Girder

The choice of the cross-section for the stiffening girder is a very important step, since this influences the behaviour of the total structural system.

The developments in girder design have been closely connected to the development of calculation methods, in particular the deflection theory which is especially relevant for lighter girders.

A clear difference between flexural and torsional frequencies is required. Since 1960 this difference has in many cases been obtained by making the girder a box girder:

- a closed box girder, e.g. Severn Bridge.
- an open box girder, e.g. Pont de Tancarville.

A serious draw-back of suspension bridges is their relatively large flexibility. For a long time it was accepted that the deformations of suspension bridges were too large for railway traffic. This view has more recently been completely abandoned. In Japan suspension bridges have been built with spans of more than 1000m, with two railway tracks (that can later be increased to four) at bottom chord level and four roadway lanes on top.

3.4 Anchorages

The main cable force is composed of two components, the vertical trying to lift the anchor block, and the horizontal trying to pull the anchor block towards the centre of the bridge.

The method of anchoring the main cable depends largely on the local soil conditions.

Two basic solutions are:

- In rock, see Figure 10 (Firth of Forth Bridge).

  The anchorage is firmly connected to the surrounding rock.

- In a clay environment, see Figure 11, (Lille Belt Bridge), and (Pont de Tancarville).

  The anchorage is almost entirely based on gravity.

In gravity anchorages equilibrium between the cable force and soil pressure has to be carefully considered during erection.

During erection the vertical component of the cable force increases, exerting a vertical lifting force on the anchorage.

In order to limit the variations in soil pressure it might prove advantageous to increase the mass of the anchorage in pace with the erection. This procedure reduces subsidence problems and could make the construction, in particular the foundation, cheaper.

4. SPECIAL EFFECTS OF BEHAVIOUR AND ANALYSIS

4.1 Temperature

Due to temperature expansion a bridge with a continuous stiffening girder is subject to an elongation over the full length. This expansion has detrimental effects on the short hangers near the anchorages.

An upper value for the change in axial stress in a short hanger can be determined by the following calculation:

Length of the hanger	$H = 3000$mm
Youngs modulus of the hanger	$E = 1{,}7 \times 10^5$N/mm^2
Girder length	$L = 1200$m
Difference in temperature	$\Delta t = 15°C$
Expansion coefficient	$\alpha = 11 \times 10^6$

With these values the displacement, δ, on one side becomes:

$$\delta = 0{,}5 \times \alpha \times \Delta t \times L = 0{,}5 \times 15 \times 1200000 \times 11.10^6 = 144\text{mm}$$

Assuming (on the safe side) that the vertical distance between the hanger ends remains constant, the following results are obtained:

$$\text{Elongation} \quad (\Delta H) \quad = \quad \sqrt{(3000^2 + 144^2)} - 3000 = 3{,}45\text{mm, and}$$

$$\text{Stress} \quad (\sigma) \quad = \quad \frac{3{,}45}{3000} \times 1{,}7 \times 10^5 = 207\text{N/mm}^2$$

In modern bridges with slender stiffening girders the assumption of a constant vertical distance is unrealistic, and the stress increase is therefore considerably smaller than determined above.

4.2 Aerodynamic Excitation

Aerodynamic excitation of the superstructure of any type of long-span bridge, but particularly of suspension bridges, may cause unacceptable oscillations. Five distinct forms of excitation may occur:

a. Vortex excitation.
b. Galloping.
c. Classical flutter.
d. Stall flutter.
e. Gust response.

In addition, a quasi-static aerodynamic instability known as divergence may occur.

Design against these effects requires specialised expertise and cannot be considered fully in this lecture. However some of them will be described briefly to give some knowledge of what has to be considered.

a. **Vortex Excitation**

When wind flows past a bridge deck, vortices are shed alternately from the upper and lower surface thus creating an alternating differential pressure and hence force on the bridge. The frequency of vortex shedding (Figure 13) is proportional to the wind speed, and the strength and regularity depend on the cross-section shape. If the frequency of shedding coincides with a natural frequency of the bridge, there is a risk of a resonant oscillation occurring.

Irregular sections, such as trusses, are rarely prone to vortex excitation. The excitation even of regular sections is very seldom strong enough to cause large amplitudes. The amplitude is inversely proportional to the structural damping, and thus adding damping can always be a cure.

b. **Classical Flutter**

Classical flutter is another serious aerodynamic phenomenon, in which vertical and torsional oscillations are coupled and the lift moment on a moving cross-section reinforces the movement. It is a well known phenomenon of flight control surfaces in aircraft. It is amenable to mathematical analysis for plate-like structures (the streamlined boxes of the Severn Bridge type are nearly plate-like).

It is probably true to say that any section whose torsional frequency is higher than its bending frequency will eventually flutter. The important objective is to ensure that this effect occurs at a wind speed substantially higher than expected to occur in the actual location. The further apart the bending and torsional frequencies, the higher the wind speed causing flutter. The box sections of suspension bridges are very good in this respect, since the frequencies are separated by a factor of about 3.

The situation becomes worse, however, with longer spans. The Great Belt East Bridge is close to the maximum span for which the simple streamlined box cross-section will remain safe from classical flutter. Furthermore, in areas of tropical storms with very high wind speeds, special measures are required. Like galloping, classical flutter is destructive and not particularly sensitive to added damping.

Normally it is the fundamental modes which are coupled. However in the first Tacoma Narrows Bridge, there was a coupling of higher modes in a flutter-like oscillation. Sometimes asymmetric modes can be suppressed by using a central tie between the cable and deck at midspan. This tie stops the longitudinal cable movement associated with such modes. However very large forces occur in the tie, so special care has to be taken when designing the connections.

Truss-type suspension bridges are not necessarily safe from classical flutter, since the roadway deck is like a plate. They can be improved by leaving open slots between the carriageways to allow air to pass through, or by having permeable grillages within the carriageways themselves.

c. **Structural Precautions**

- For the closed box section a streamlined shape is simulated as close as possible (Slide 11), giving the additional advantage of reducing the drag coefficient. The problem is now removed as far as the bridge is concerned. However, the traffic on the bridge can be subjected to stronger side winds.

- For an open lattice box girder, some precautions are aimed at a disturbance of the vortices:

  - the deck is constructed to make the wind pass through openings (Slide 12).

  - a part of the deck is built with grids (Figure 14).

A very effective method is a tight connection between the cable and main girder at midspan (Figure 15).

Due to the torsional vibration mode, there is a considerable horizontal displacement of the main cable. This displacement, and consequently the torsional mode, is hampered effectively by clamping (Figure 16).

4.3 Analysis

The suspension bridge has little rigidity of its own. In fact deflections have an influence which must be considered in the analysis. The problem is to determine what bending moments occur, taking deformations into account.

This problem was solved by Ritter in 1877, and confirmed by Melan in 1888, with the deflection theory. This theory was first used by Moisseiff in 1908 in the design of the Manhattan Suspension Bridge. Since that time it has become the classical method of suspension bridge analysis, after some developments by Steinman, Timoshenko, etc.

The main assumptions of this theory were:

1. The cable, the shear centre of the stiffening girder, and all loads lie in a single vertical plane.

2. (a) The hangers are vertical, inextensible, and long enough for any departures from verticality to be ignored.

   (b) Their spacing is infinitely small, i.e. they form a continuous sheet between the cable and the girder.

3. Shear strains of the stiffening girder are negligible.

4. Live-load deflections are sufficiently small such that all forces applied by the hangers to the cable may be taken to act along lines fixed by the dead-load geometry.

5. The dead load is uniformly distributed along the girder.

6. The moment of inertia of the stiffening girder is constant.

7. The dead load is taken by the cable alone and produces zero bending moment in the stiffening girder.

Using these assumptions, the differential equation for live load-deflections in the deformed states could be obtained and the whole problem solved. In the analysis, dynamic terms also had to be inserted to compute frequencies.

Today the calculations are based on finite element programs that can include large deformations and the associated non-linear effects. With these programs most of the classical assumptions listed above can be omitted.

5. CONNECTIONS

5.1 Hangers and Cable Bands

Socketed hangers connect the main cable and girder.

The hanger is connected to the main cable via a cable band consisting of two semi-cylindrical halves, connected together by high tensile steel bolts to develop the necessary friction. The hanger is connected to this cable band via a pin connection or it may be looped over the cable band.

The cable bands are firmly tightened onto the main cable and get their load carrying resistance mainly from friction and compression of the cable (Figure 17). The cable bands are carefully machined, taking into account an air void of approximately 20% in the cable.

The main cable is subject to an axial loading that increases during erection of the bridge. The elongation of the cable from the anchor block to the pylon should be taken into account, e.g. by giving the pylons a pull-back (Figure 18).

For the cable bands, the transverse contraction of the cable section is of the utmost importance. It causes the friction between cable band and cable to decrease and, as a result, the load carrying resistance goes down. Precautions should be taken to measure the relaxation and to tighten the bolts during erection, e.g. by making a backlash (Figure 19). For reasons of maintenance, the remaining gap is filled with rubber. In view of the contraction, the cable wrapping should be carried out after the bridge carries almost all of its full dead load.

Vertical hangers are usual. For a period of about 15 years inclined hangers were popular (Figure 20). The use of inclined hangers started with the Severn Bridge (1966) and was concluded with the Humber Bridge (1981).

The idea was to make the bridge more rigid ($\approx$ 25%), due to the truss behaviour, and to reduce the tendency to oscillate (flutter). The aim was to increase damping by utilizing the hysteresis of the spiral wires forming the hangers. However, the constantly changing forces in the hangers can create fatigue problems, and this is one of the reasons why designers returned to apply vertical hangers only.

6. SPECIAL FEATURES OF CONSTRUCTION

The dead load of the girder and the cable systems can be carried entirely by the main cable provided that this will have a configuration coinciding with the funicular curve of the applied load.

This favourable transmission of dead load is achieved during erection of the bridge. The anchorage, the towers, the cables, the cable bands and the hangers are erected, in sequence.

The erection of the girder follows and is usually performed by:

- lifting sections from a barge by a crane above the main cables beginning at midspan.
- connecting the sections to the hangers and to each other by temporary joints.

As the sections of the girder behave like concentrated loads, the deflection of the main cable is large and the sections show openings at the bottom (Figure 21).

As the erection continues these openings close and finally openings on the top side of the girder appear. This is due to the fact that the girder is lighter during erection than in its serviceability condition, e.g. the wearing surface is missing.

After having erected a little over half of all sections, the final connections are usually made.

In the erection stage aerodynamic oscillations may also occur. Therefore temporary erection stages also have to be tested in a wind tunnel. Special devices to meet these problems have in some cases been required, e.g. in the girder of the Humber Bridge.

7. CONCLUDING SUMMARY

- The types of suspension bridges may be described according to the suspension of the girder, anchorage of the main cable and the number of hinges in the girder. The main parts of suspension bridges are the anchorages, main cables, hangers and cable bands, pylons and stiffening girders.
- Suspension bridges are influenced by temperature and may be subject to aerodynamic excitation. Their structural analysis is normally carried out on the basis of the deflection theory.
- The erection procedure is chosen to exclude the dead load from introducing major bending moments in the girder.

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ESDEP WG 15B
STRUCTURAL SYSTEMS: BRIDGES

Lecture 15B.10: Bridge Equipment (Bearings, Parapets, etc.)

OBJECTIVE/SCOPE

- To describe the various items of equipment of a bridge, and to explain their function in the bridge structure.

- To draw the attention of the design engineer to the importance of choosing the equipment as a function of the life and the maintenance of the bridge.

RELATED LECTURES

Lecture 4A.4:	Corrosion Protection of Bridges
Lecture 15B.1:	Conceptual Choice

SUMMARY

In addition to the load carrying structures that make up the bridge deck, bridges also include a number of items of equipment which are essential to their service, function and life duration.

- bearing systems
- finishes
- expansion joints
- parapets
- anti-corrosion protection
- drainage
- fascia
- inspection facilities.

The choice of these items and installations depends not only on their initial cost, which may reach 10% of the total construction cost, but also on the operating cost related to their routine maintenance and possible replacement.

The items contribute to the length of life of the structure and as a consequence, they should not be the origin of problems that may affect the resistance of the bridge.

1. BEARING SYSTEMS

Bearing systems provide the mechanical fastening between the main load carrying members (main girders, arches) and the bridge supports (piers, abutments, foundation blocks, etc.). They contribute therefore to the functioning of the bridge as a whole.

1.1 Function

The function of a bearing system is to transmit to the supports:

- the vertical and horizontal actions;

In doing so, the bearings must accommodate:

- the rotational and translational displacements of the structure caused by permanent and service loads, earthquake actions, thermal effects, wind and settlement of supports.

The designer must choose the types of bearing in each bearing system to suit the action effects and degrees of freedom required at each connection between the main members and the supports.

In general, the forces and displacements on a bearing are as shown in Figure 1. It is conventional to define the X axis as 'longitudinal' and the Y axis as 'transverse'.

1.2 Layout

As a general rule, a bearing system includes three kinds of bearing:

- fixed - resisting the horizontal forces in both X and Y directions.
- unidirectional - providing restraint in either X or Y direction.
- multidirectional - providing no restraint in either X or Y direction.

The number and layout of the three kinds of bearing is a key feature of the bearing system for a bridge.

In most cases the bearing system will be indeterminate. For a simple span the arrangement might be as shown in Figure 2a. Note that only one fixed and one unidirectional bearing are required. <u>All</u> others should be multidirectional (or 'free') for the system to be determinate.

For a longer bridge, for example a curved viaduct, it may be necessary to provide lateral restraint at each intermediate support, as well as at the ends. Then two alternative arrangements are possible. In the first, Figure 2b, unidirectional bearings are arranged radially from the fixed bearing; in the second, Figure 2c, they are arranged tangentially. Both of these two systems are indeterminate and this must be taken into account in the global analysis of the structure.

1.3 Types of Bearing

There are four distinctive groups of bearing whose differences result from the structural materials used and from their structural behaviour.

1.3.1 Steel bearings

These bearings function through direct contact between steel elements. Originally executed in cast steel, steel bearings are now fabricated from structural steel plate and bars.

- **Linear bearings**

Linear bearings provide support through contact between a flat surface and a cylindrical one. The line contact allows rotation about one axis, usually the transverse axis, W_y.

Restraint against modest horizontal forces is provided by a shear key, often in the form of a dowel.

Rocker bearings consist of a lower rocker bearing (usually set in the concrete) and an upper rocker plate fastened to the main girder. Horizontal forces are transmitted by a shear key (Figures 3 and 4).

Roller bearings comprise one or several solid steel cylinders (or rollers) put between two parallel rocker plates, so that relative displacement in the X direction is made possible by rolling action. Small linear shear keys are used to resist the transverse forces H_y (Figure 5). To ensure that the roller axis does not skew in service, toothed guides are usually provided on the ends of the rollers.

The minimum radius R of the cylindrical surface is determined by the contact pressure between the cylinder and the flat surface.

The elastic stress between a cylindrical and a flat surface is given by the Hertz formula:

$$\sigma_c = 0{,}418 \sqrt{VE/bR}$$

where:

V is the reaction
b is the length of contact line
E is the modulus of elasticity
R is the radius

Although this formula is applicable to an elastic condition, it has been found that it is satisfactory to allow the Hertz stress to be limited to values in excess of unaxial yield. Typically, the limitation for cylindrical rollers is:

$$\sigma_c \leq 1{,}7\ \sigma_u$$

where:

σ_u is the ultimate tensile strength of the steel.

Where a larger roller is required but the longitudinal movement is small, a flat-sided roller can be used (Figure 6).

- **Point bearings**

Sometimes bearings are required which offer multi-directional freedom of rotation. In such cases, recourse is necessary to a spherical hinge produced by a point bearing

plane/spherical contact point bearing (Figure 7)

The contact pressure between a sphere and a plane is given by the Hertz formula:

$$\sigma_c = 0{,}388 \sqrt[3]{VE^2/R^2}$$

Typically the limitation on stress is:

$$\sigma_c \leq 2{,}1\ \sigma_u$$

double spherical point bearing (Figure 8)

For a double spherical contact, the pressure is given by:

$$\sigma_c = 0{,}388 \sqrt[3]{VE^2[1/R_1 - 1/R_2]^2} \qquad (R_2 > R_1)$$

- **Sliding point bearings**

In addition to a spherical bearing contact, these bearings include a (uni- or multi-directional) sliding plane. Sliding occurs between a stainless steel (inox) insert and a PTFE plate (Figure 9).

1.3.2 Elastomeric bearings

These bearings are essentially non-isotropic rectangular blocks which can resist:

- vertical deformations;
- horizontal distortions;
- rotations.

Each bearing consists of several elastomer layers of various thicknesses, 8 to 20mm, glued together with 2 to 5mm thick steel plates (Figure 10).

Half-fixed simply supported bearings

This type of bearing offers resistance to horizontal forces and displacements as well as carrying vertical loads. To do this the bearing must be securely fastened to the bridge structure and to the bearing supports.

In the bearing system, the horizontal forces are distributed in proportion to the combined stiffnesses of the bearings and of the piers and foundations (Figure 11).

This type of bearing is suitable provided the height of the block in relation to horizontal displacement is rather low. Beyond a certain height, the block becomes unstable.

Elastomeric sliding bearings

When horizontal displacements are large, elastomeric bearings can be used with a sliding surface.

This is achieved by providing a stainless steel (inox) plate sliding on the upper face of a PTFE plate glued to the block of reinforced elastomer (Figures 12 and 13).

1.3.3 Pot bearings

Pot bearings consist of a plain elastomer enclosed in a metal cylindrical pot. The upper face of the pot is a piston cover freely installed in the pot and bearing on the elastomer (Figure 14).

The elastomer enclosed in the pot distorts under constant volume and behaves as a fluid. It withstands both high pressures (25 MPa) and rotations (1/100 radians) because of zero shear stresses. This type of bearing effectively offers multi-directional freedom of rotation.

Pot bearings are less bulky than ordinary elastomeric bearings and give a higher performance. They are widely used.

Horizontal freedom is obtained by addition of a sliding plane (stainless steel-PTFE) on the piston cover (Figure 15).

A sliding uni-directional bearing may be obtained by the addition of an exterior or interior guide.

1.3.4 Spherical bearings

This type of bearing is an all-steel construction somewhat similar to the pot bearing but with the elastomer replaced by a convex spherical cap sliding on a concave spherical element (Figure 16).

As before, introduction of sliding plane gives minor multi-directional freedom of displacement.

1.4 Setting Conditions for the Bearing Systems

The stability over time of a bearing system depends largely on careful installation:

- accurate levelling on each bearing line;
- suitable bedding of the bearings on the supports;
- taking into account the condition of residual camber of the main girders in the setting stage;
- alignment of directional supports;
- adjustment of the mean position of sliding bearings according to the temperature at the setting time;
- protection of sliding surfaces.

2. FINISHES

In association with the deck slab, finishes consist of:

- the waterproofing course;
- the wearing course.

2.1 Waterproofing Course

2.1.1 On a concrete slab

The waterproofing course should protect the slab against any penetration of water which may contain more or less aggressive agents mainly, from the salt used for de-icing the carriageway (Figure 17). These agents of various origins may be harmful to the slab concrete, but even more detrimental to the steel reinforcement bars causing corrosion.

A good waterproofing course may thus contribute to the life of the structure.

Several techniques may be applied:

- A thick course consisting of 4 - 8mm thick asphalt mastic linked to the slab by a bonding layer and 22 - 26mm thick gritty asphalt.

- A thin layer, 2 - 3mm thick, consisting of a two-component system of coal pitch and epoxy resin.

- A prefabricated foil made up of a bitumen modified by a polymer and reinforcement. The foil is glued by partial fusion of the binder on a cold impregnated plaster.

2.1.2 On an orthotropic slab

After descaling of the steel plate and immediate application of a bonding varnish, the waterproofing is added as a course of elastomer bitumen of about 3 - 5mm thickness (Figure 18).

The waterproofing layer is continued under the footway and covers all the upward (safety fence support, kerbstones) and downward (gutter channels) parts.

2.2 Wearing Course

The thickness of the wearing course varies from 6 to 10cm. To obtain satisfactory performance of the wearing course requires a good preparation of and bonding to previous course with strict observance of the prescribed hygrometric conditions. Where the slab is flexible it is necessary to ensure the wearing course has sufficient fatigue strength and is laid to the specified thickness.

The evenness of the surface as well as the continuity of the profile provide a smooth surface for traffic, reduce mechanical vibrations of the traffic, and avoid the formation of ruts and surface deterioration due to the winter frost.

2.2.1 On a concrete slab

The wearing course consists generally of bituminous concrete.

2.2.2 On an orthotropic slab

The common systems make use of specific compositions and spreading procedures which take into account the slab flexibility and fatigue behaviour. The systems used are a special material, which is made up of a bituminous concrete with a proportion of binder to confer a satisfactory plasticity to the course.

3. EXPANSION JOINTS

Expansion joints provide continuity of the road surface at the interface between the bridge deck and the abutments.

3.1 Characteristics of Expansion Joints:

3.1.1 Range of movement

An expansion joint must be able to accommodate a range of movement, opening and closing, from its 'neutral' position, or critical setting.

The range of movement, i.e. the maximum displacements at open and closed positions of the joint, depends on several factors:

(i) linear thermal expansion and contraction of the bridge deck:

$$\Delta \ell_1 = L.\lambda.\Delta T$$

where:

L is the distance from a fixed bearing

$\lambda = 1{,}1 \,.\, 10^{-5}$ per °C for steel

ΔT is the difference between extreme and neutral or setting temperature of the bridge.

(ii) horizontal displacements resulting from rotations about a transverse axis under service loads. At any supports:

$$\Delta \ell_1 = \theta \,.\, h$$

where:

θ is the rotation
h is the distance from the neutral axis.

Note that the displacements due to rotation both at the fixed bearing and at the expansion joint should be considered.

(iii) long term deformation of the concrete slab (shrinkage and creep)

(iv) horizontal displacements due to the braking forces and the flexibility of the "fixed" bearing.

The total range required will determine what types of joints are suitable. Joints must be set carefully, taking into account the temperature of the bridge at the time of setting and the opening and closing movements (which are not usually equal) from the neutral positions.

3.1.2 Design characteristics

Designers need to consider the following points:

- The resistance of the joint and of the anchoring points in the structure to the fatigue loading due to the traffic.
- Waterproofing of the attachment between the joint and the waterproofing facing.
- Ease of maintenance and replacement.
- Silence and comfort (The best joint is the one which is not apparent).

3.2 Types of Expansion Joints

There are various technical solutions for expansion joints. The differences between the types relate to the amount of movement or displacement between the open and closed positions at the break.

3.2.1 Joints with continuous surfacing (Asphaltic plug joint)

This type gives a very comfortable riding surface, but the capacity of movement is restricted to 30mm. Only light or semidense traffic can be carried (Figure 19).

3.2.2 Toothed joints

Two thick and firmly anchored sheets slide one into the other. They are in the form of straight or biased teeth which allow movements of 25 to 350mm (Figure 20). Larger joints require intermediate support of the teeth.

3.2.3 Elastomeric joints

An elastomeric profile with steel plate inserts is fastened on two steel plates and anchored in the slab. Movements of up to 300mm are possible (Figure 21).

3.2.4 Roller shutter joints

This joint consists of a series of hinged shutters, side by side. Each shutter consists of a train of linked plates sliding on a guide. Large deformations of 1m or more are possible (Figure 22).

3.2.5 Multiple steel joints or bellow joints

These joints consist of a series of transverse steel beams linked with flexible strip seals (Figure 23). Each steel beam is supported on girders or joints below the beams. Movement is accommodated in 'accordion' fashion as each of the seals flexes. The number of beams can be increased in a modular way so that movements of up to 800mm can be accommodated.

4. PARAPETS

Parapets are necessary to protect both the users and the carriageways and railways. There are several types:

- pedestrian parapets;
- crash barriers for light vehicles;
- vehicular barriers for heavy lorries.

This equipment which is needed to meet general safety requirements, has to conform to detailed specifications. Acceptance by official inspection bodies is usually based on full scale testing.

4.1 Pedestrian Parapets

Pedestrian parapets are dedicated to the safety of people. Their shapes vary depending on their use and requirements of appearance.

Whether made of steel or aluminium alloy, all pedestrian parapets should conform to the same strength and safety requirements (Figure 24).

4.2 Crash Barriers

To be efficient, both sliding rails and barriers should:

- absorb the shock of a crash;
- permit vehicles to slide on them;
- retain and re-direct the vehicle.

Crash barriers are usually bolted to the structure through an anchorage incorporated in the deck slab. The fastening is designed to ensure the structure is not damaged in case of an accident so that a quick and easy repair is possible (Figure 25).

4.3 Safety Fences

Depending on their purpose and the structural materials of which they are constructed, safety fences may be of various types:

- a rigid fence in reinforced concrete (Figure 26);
- a very flexible fence consisting of a chain of prestressed concrete blocks;
- a flexible steel fence (Figure 27).

5. ANTI-CORROSION PROTECTION

Depending on the ambient atmospheric conditions, steel corrodes naturally and continuously and the resulting corrosion affects its service life.

As a result the protection of steel from possible electro-chemical corrosion is absolutely necessary.

Because of the importance of this problem, Lectures Group 4A are dedicated to corrosion protection.

6. DRAINAGE OF RAINWATER

The durability of the bridge as well as the users' safety also depend on good drainage of the deck (Figure 28).

Drainage is carried out by means of:

- a transverse profile of both the carriageway and the footway with a slope of 2 - 2,5%, which leads the rainwater into gutter and along the kerb of the footway;
- a longitudinal profile which eases the drainage downstream;
- water gulleys and water traps under the gutter channels whose location and dimensions are determined as a function of slope and of water volume to be drained;
- water downpipes which may be connected to collectors and to outfall sewers in towns or pollution-protected areas.

7. FASCIA

The fascia are built on the deck edge and perform several functions (Figure 29):

- **functional role** - the fascia includes a drip stone which prevents water falling on them from flowing onto the underface of the slab and then onto the girders;
- **aesthetical role** - the fascia mark the crown line of the bridge. By associating them with the parapets the architect may design the shapes, material qualities and aspects of the facings in order to enhance the impression of the structure in the environment.

The current tendency is for the fascia to have essentially a decorative role. Fascia are therefore dealt with as both a light and aesthetical cladding element.

8. INSPECTION FACILITIES

Inspection facilities giving access to all the parts of bridge structures are required because of the need for periodic visits to the structures for inspection and maintenance purposes.

There are three types of inspection facility:

8.1 Fixed Installations

Fixed installations are service platforms located in the beam grillage of the bridge structure (Figure 30).

8.2 Movable Installations

A motor-driven platform gantry moves along tracks fixed the full length of the bridge. Retractable and flexible elements allow access to both exterior and underside of the overhangs (Figure 31).

Clearances are provided with design so that the bridge deck can be reached and the piers can be passed.

8.3 Special Equipment

Special pieces of equipment are available, such as telescopic arms mounted on lorries or trucks which are parked on the carriageway. Such mobile equipment is economic if the equipment may be used on a large number of structures (Figure 32).

9. INTEGRATION OF THE EQUIPMENT INTO THE GENERAL DESIGN

The global design of a bridge structure is first of all determined by the geometrical and geotechnical conditions of the site, the nature of the route, the clearances and the construction conditions.

The overall design should integrate the various pieces of equipment. They may generate stresses and constraints as follows:

- **Geometrical considerations**

  They take up space and require incorporating into the structure with adequate reinforcements and anchoring points. Consideration needs to be given to their safe erection.

- **Mechanical stresses**

  The weight of the equipment is one of the non-negligible parts of the dead load of the structure.

  The forces to which some of the pieces of equipment are subjected, e.g. passage of axles over expansion joints, impacts on parapets, load transfer on bearing systems, generate significant stresses in the secondary structure.

- **Aesthetical constraints**

  The design of the fascia and parapets should be in harmony with the bridge structure.

- **Maintenance constraints**

  In view of the needs for inspection, maintenance or replacement of structural members of the bridge, sufficient space for the access facilities should be provided.

A satisfactory project results only if all these factors are taken into account.

10. CONCLUDING SUMMARY

- Bridges include a number of types of equipment which are essential to their overall performance and life.
- Bridge equipment includes: bearing systems, finishes, expansion joints, parapets, anticorrosion protection, drainage, fascia and inspection facilities.
- Selection of the equipment should be based on both the initial cost and the cost related to maintenance or replacement.

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ESDEP WG 8
PLATES AND SHELLS

Lecture 8.5.1: Design of Box Girders

OBJECTIVE/SCOPE

To describe the main features and advantages of box girders; to introduce the methods of global analysis used and to typical reinforcing details using stiffeners and diaphragms.

PREREQUISITES

Lecture 6.1: Concepts of Stable and Unstable Elastic Equilibrium
Lecture 8.1: Basic Introduction to Plate Behaviour
Lecture 8.4: Introduction to Plate Girders

RELATED LECTURES

Lecture 8.6.2: Box Girders - Special Topics

SUMMARY

The advantages of box girders are compared to those of plate girders. Their structural behaviour is discussed in global and detailed terms, covering matters such as diaphragm and stiffener design, web buckling and torsion. Recommendations regarding fabrication details are also given.

1. INTRODUCTION

Box girders are used in building structures (Figure 1) as well as in bridges (Figure 2). In general, they are more expensive than plate girders because they require more time to make. They have, however, several advantages over plate girders which make their use attractive:

- very good torsional rigidity; for highly curved spans, box girders are almost essential;
- very wide flanges allow large span to depth ratios;
- a neater appearance (since the stiffening need not be visible) especially when the webs are inclined. In certain cases, for aesthetic reasons, it is the only solution which is preferable to concrete (Figure 2a).
- a very good aerodynamic shape, which is important for large suspension or cable-stayed bridges where the resistance to lateral wind is the main problem (Figure 2b).

For bridges the two main types of cross-section are:

- two box girders, on discrete columns, connected by cross beams and a composite concrete slab. Each box girder has longitudinal stiffening on the compression flange (Figure 3).
- a wide box girder, to facilitate fabrication, with the top flange formed by the concrete deck slab (Figure 1).

The box girder with an orthotropic deck is closed during all stages of erection (Figure 4). On the other hand, temporary horizontal bracing is generally provided between the webs of box girders with a top flange formed by a concrete deck slab until the concrete has hardened.

2. MAIN FEATURES OF BOX GIRDERS COMPARED TO PLATE GIRDERS

- The main problem in the design of a box girder is the stability of the longitudinally and transversely stiffened flange in compression; this is a more complex problem than the web of a plate girder in bending, even when it is stiffened longitudinally.

- The tension field theory, used for the shear verification of plate girders, needs to be corrected to take very wide flanges into account.

- At supports, special diaphragms generally located between the webs, are used to transmit the reactions (Figure 5).

In the case of narrow box girders, the load bearing stiffeners can be located outside the box to increase stability (Figure 6).

Special attention must be given to shear lag, distortion and warping stresses.

3. GLOBAL ANALYSIS

Depending on the features of the structure (short or long span, presence of horizontal curvature, etc.) different methods of analysis can be used, generally in the elastic range:

- classical theories based upon the strength of materials - grillage methods for global analysis and beam on elastic foundations analogy for torsional, distortional and warping stresses.

- folded plate analysis.

- finite element analysis in complex cases.

In very wide flanges, the shear lag effects, which are neglected in the global analysis, have to be taken into account for the verification of stresses, especially for short spans. The calculation is performed by means of an effective breadth which depends on the ratio of width to span.

Torsional moments on the box girder tend to distort the cross-section. Intermediate diaphragms, spaced at suitable intervals, are necessary to prevent this distortion.

The warping stresses are in general very low but should be taken into account.

Most steel bridges have been designed using the linear theory of buckling (for example, Kloppel's charts). More advanced methods are now available which will be used in the future; two of these, the strut approach and the orthotropic approach, are covered in subsequent lectures.

4. DESIGN OF STIFFENERS

The design of the longitudinal stiffeners can be by linear buckling theory or by the non-linear method. Torsional or local buckling is avoided by limiting the b/t ratios of the cross-section to class 3 or less when using the non-linear theory.

The transverse stiffeners should satisfy two criteria:

- stiffness

- strength; in addition to the direct transverse forces, the transverse stiffeners must resist a lateral load of 0.5% to 1% of the compressive forces in the stiffened panel.

5. WEB BUCKLING

- In bending the problem is the same as for plate girders.

- Usually, under shear forces the slenderness of the flange plate is such that it is not possible to anchor part of the post-critical tension band at the flange. In such cases, it is advisable to limit the design shear resistance to the "web only" shear buckling resistance, i.e. $V_{bw.Rd}$ (see Lecture 8.5.2) which is the value given by the tension field method assuming:

$$s_c = s_t = 0$$

6. TORSION

Torsion produces a shear flow in the box section. Each panel in the webs or flanges, is designed to resist this shear flow, generally using linear buckling theory.

Post-critical behaviour under torsion

The post-critical resistance is provided by diagonal bands in tension, in the flanges and webs. Figure 7 illustrates the tension field 1 and 1'; due to the large flexibility of the horizontal panels, the width of this band is limited in comparison to that for the web of a plate girder. Moreover, equilibrium for a box girder length, between two diaphragms, requires compressive forces (see force 2, Figure 7) at the edges of the box girders. It is therefore advisable not use the post-critical reserve.

7. DIAPHRAGMS

7.1 General Function and Description

The main functions are:

- to preserve the shape of the box against distortion.
- to resist an externally applied torque through shear flow.
- to limit the length of longitudinal stiffeners under compression, or of unstiffened panels.
- in certain instances to support traffic loads directly (orthotropic deck, for example).
- to transmit vertical forces from webs, through shear and compression, to the supports.

The main types of diaphragms are shown in Figure 2 (diaphragm with a man hole), Figure 3 (unbraced cross frames) and Figure 4 (braced cross frame).

7.2 Intermediate diaphragms

For large deep box girders, intermediate diaphragms are generally unbraced or braced cross frames. An effective width of flange plate and web is taken into account when calculating stresses in the ring. Special attention is given to the design of the corners of the unbraced cross frame (which should resist the bending moments in the plane of the cross frame), and to possible eccentricities in the case of braced cross frames. When the intermediate diaphragm does not support traffic loads directly, it is, in general, lightly stressed.

7.3 Support Diaphragms

In addition to performing the different functions of intermediate diaphragms, the main purpose of support diaphragms is to transform the large support forces into shear flow along the web of the box section (Figure 5).

If a bridge consists of a single box girder, there are generally two supports at each end. Special attention has to be paid to any differential settlement between these two supports because of the inherently high torsional rigidity of the box girder. Single intermediate supports can sometimes be provided which also facilitate traffic flow under the bridge.

For large box sections, with large support forces, the use of a finite element program is recommended for the design of support diaphragms.

8. DETAILING

The detailing recommendations are essentially the same, whether linear or non-linear buckling theory is used.

General recommendations

- The longitudinal and transverse stiffeners must be welded to the plate. However, for bottom flanges that are not too wide and not loaded transversely, the arrangement with transverse stiffeners above longitudinal ones is acceptable (see Figure 11).

- For reasons concerned with fatigue and shrinkage, it is recommended that the longitudinal stiffeners are continuous and therefore pass through openings in the transverse ones. In this case the openings should satisfy the recommendation given in Figure 8.

- All changes in plate thicknesses and stiffener depths must be progressive (a slope of 1/5 or 1/6 is recommended), see Figure 9. A longitudinal stiffener should be tapered at its extremity.

- Cut-outs in the stiffeners, intended to facilitate welding or the control of butt welds in the plate, should comply with the requirements of Figure 10.

- When two plates of different thickness are welded in a compression zone by aligning plate faces, a transverse stiffener should be welded to the thinner plate as indicated in Figure 11 (b is the spacing of the longitudinal stiffeners.

9. CONCLUDING SUMMARY

1. Box girders are used because of their good resistance to torsion, good aerodynamic shape and aesthetic appearance.

2. The design of diaphragms, and the analysis of the stability of the compression flange, both require careful consideration.

3. Different levels of refinement can be used in the global analysis; however, a classical grillage method is sufficiently accurate for most purposes.

4. The transverse stiffeners contribute to the load transmission as well as preventing distortion of the cross-section.

5. It is not advisable to use the post-critical reserve under torsion.

10. ADDITIONAL READING

1. Eurocode 3: Design of steel structures": European Prestandard, ENV 1993-1-1: Part 1.1: General rules and rules for buildings, CEN, 1992.

2. Dubas, P. and Gehri, E., Behaviour and Design of Steel Plated Structures, Technical Committee 8 Group 8.3, ECCS-CECM-EKS No 44, 1986.

3. Johnson, R. P. and Buckby, R. J., Composite Structures of Steel and Concrete, Volume 2: Bridges, Collins, London, 1986.

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5. Kollbrunner, C. F. and Basler, K.: Torsion, Spes/Bordas Lausanne/Paris, 1955.

6. Stahlbau Handbuch: Stahlbau Handbuck für Studium und Praxis, Band I, Stahbau Verlag, Kōln, 1982.

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ESDEP WG 8
PLATES AND SHELLS

Lecture 8.5.2: Advanced Methods for Box Girder Bridges

OBJECTIVE/SCOPE

To introduce methods of global analysis, methods of determining cross-section distortion, and shear lag in box girder bridges.

PREREQUISITES

None.

RELATED LECTURES

Lecture 8.5.1: Design of Box Girders

SUMMARY

Global analysis may be made by the grillage, orthotropic plate, folded plate and finite element methods.

Distortion of the box may have to be controlled by diaphragms or cross frames. Simple or refined methods are available for the calculation of the forces in the diaphragms or cross frames.

In very wide flanges, shear lag effects have to be taken into account.

1. INTRODUCTION

Steel or steel-concrete composite box girders are usually more expensive than plate girders because they require more fabrication time. They have, however several advantages over plate girders which make their use attractive:

- very high torsional rigidity: In closed box girders, torque is resisted mainly by Saint Venant shear stresses because the Saint Venant torsional stiffness is normally much greater than the torsional warping stiffness. For highly curved spans, this stiffness of the box girders is almost essential during their construction, as well as under service loads. Metallic closed-top boxes even allow torsional stiffness during their erection, without needing expensive temporary bracing, which would interfere with the execution of the concrete slab.
- very wide flanges allow large span to depth ratio.
- a neater appearance since the stiffening can remain invisible in the box.
- a very good aerodynamic shape, which is equally important for large suspension or cable-stayed bridges as is the torsional stiffness.
- a very good adaptability to the most difficult conditions. Box girders are able to cross greater torsional spans than flexural spans using piers with a single bearing as shown in Figures 1 and 2.

2. GLOBAL ANALYSIS METHODS

The principal methods which are in use for the global analysis of box girder bridges, and their applicability to the various types of steel or steel-concrete composite bridges, are discussed below.

Most composite bridge decks fall into one of two groups.

Beam theory can be used for bridges in the first group which can be idealised as a grillage or an orthotropic plate. Bridges in this first group include beam-and-slab and multi-separate-box types, and are the simplest to analyse. The box girders are assumed here to be stiffened enough to stay undeformable. For uniform loading, elementary beam theory gives useful results, but distribution analysis by a grillage or an orthotropic plate method is needed for more complex loadings.

In the other group are bridges in which the longitudinal members consist of a closed box with a few internal webs, which effectively resist vertical shear only. In general, the effects of loading on such structures are best evaluated by dividing the load into bending, uniform torsional, warping torsional, distortional and local components. The Bredt and Leduc theory for thin-walled hollow sections gives the primary shear stresses due to torsion. Timoshenko and Vlassov have shown the validity of this hypothesis with the theory of undeformable girder sections : the Saint Venant torsional stiffness is much greater than the torsional warping stiffness.

However the shear stresses are accompanied by longitudinal torsional and distortional warping stresses. The distortional warping stresses are the consequence of the bending deformation of the section walls in the transverse plane. They usually are several times larger than the torsional warping stresses. They reach their maximum values at bearings, at cross-sections subjected to eccentric point loads, and at the location of the diaphragms which prevent the deformation of the girder section.

3. GRILLAGE

In the grillage analysis, the structure is represented by a plane grillage of discrete but interconnected beams. Almost any arrangement in plan is possible. Thus skew, curved, tapering or irregular decks can be analysed. The usual layout consists of sets of parallel beams in two directions. This type of layout is discussed below, assuming the plane of the grillage to be horizontal.

The method does not enable study of warping stresses and shear lag. Local effects in decks and slabs can only be studied with a grillage by the use of a dense network of beams.

In a simple form of grillage analysis, each beam is allotted a torsional stiffness and a flexural stiffness in the vertical plane. Vertical loads are applied only at the intersections of the beams. The matrix stiffness method of analysis is used by existing computer software to find the rotations about two horizontal axes and the vertical displacement at these nodes. Hence the bending and torsional moments and vertical shear forces in the beams at each intersection are found.

3.1 Choice of Grillage

Care must be taken to select an appropriate idealisation for a continuous structure, and also to deduce the stress resultant in it from the results of the grillage analysis. The principal conclusions relevant to the design of composite structures are as follows:

- Grillage analysis is not the most suitable method for one simple beam-and-slab deck.
- Each longitudinal web should be represented by a grillage beam (so that a simple box becomes two beams). The alternative would be to use one single beam in the axis of the box to represent it, and to dispose very rigid extensions on each side of the longitudinal beam at the location of every transverse beam with a total length equal to the box's breadth : the global effects in the structure are easier to model with this second method.
- Each transverse diaphragm or cross-frame should be represented by a transverse beam. If there are none within the span, or if their spacing exceeds the mean spacing of the longitudinal beams by more than 50%, additional transverse beams representing the slab alone should be added, and there should be at least nine in all, within every span.
- The transverse grillage members should extend to the edge of the real slab and their ends should be attached to longitudinal grillage beams, even if the real slab has no significant edge stiffening.

3.2 Skew Bridges

Skew bridges should be avoided, if possible in the design scenario of the road, especially when high torsional stiffness is caused by the choice of a box girder section.

For skew decks, the grillage cross beams should be parallel to the real cross members, if any. The stiffness and position of the real supports should be accurately modelled in the grillage. Real cross members can, with advantage, be skewed for skews up to about 20°, but for high skews they should be orthogonal. For decks without cross members, a skew grillage is preferred for skews up to about 30° because the preparation of input data for an orthogonal grillage is more complex. For highly skewed decks, the cross members, if any, and the grillage beams should be orthogonal, to avoid the high torsional moments inherent in skew layouts. Transverse reinforcement in the slab should be parallel to the transverse grillage beams.

3.3 Local Effects in Decks

If the purpose is to model the local bending and shear effects in deck slabs, the concrete slab of a composite bridge should be represented by a dense network of members. The distance between two parallel members of the grillage should be 0,50 m. For a plane grid analysis, the torsional stiffness of such a concrete strip should not, for instance, be b.t3/3 but should be reduced to only b.t3/6, where b is the breadth of 0,50 m and t the thickness of the deck. This reduction is due to the fact that no Saint Venant shear stress flux goes around the perimeter of the strip's cross-section. Vertical local loads should be only applied at the intersections nodes of the beams.

To model the equivalent loads, approximately one half of the local load can be distributed over the eight nodes of the vicinity to get correct results, even near the loaded point.

3.4 Torsional and Flexural Rigidities of Grillage Members

For steel-concrete composite box girders, the areas of structural concrete are transformed to steel on a modular basis. The Young's modular ratio is used for flexure, and the ratio G steel / G concrete = 5 is used for torsion due to temporary traffic loads.

For a composite box girder bridge, with a concrete upper deck (Figure 3), the torsional stiffness K is, for example:

$$K = \frac{4\,A^2}{\oint \frac{G_i}{G} \cdot \frac{ds}{t}} = \frac{h^2 \cdot (b_b + b_t)^2}{2 \cdot \frac{s}{t_w} + \frac{b_b}{t_b} + n \cdot \frac{b_t}{t_t}}$$

where n = G steel / G concrete = 5

Shear lag can usually be neglected in global analysis.

For regions where the concrete slab is in compression, the flexural stiffness is strictly that of the uncracked reinforced section. Even where the concrete has cracked under a traffic load, its stiffness remains. For these reasons, it is common to assume in global analysis that, for the computation of the beam rigidities, all concrete is uncracked and unreinforced.

3.5 Longitudinal Grillage Members

Flexural rigidities of longitudinal members depend on the breadth of concrete slab that is assumed to be associated with each steel web. For the deck shown in Figure 4, arbitrary division of the deck midway between webs would result in the neutral axis of the beam web EH being higher than that of the beam with web FG. More rigorous analysis of concrete decks shows the neutral axes of all the longitudinal members to be almost at the same level across the width of the deck. Such analysis also suggests that the breadth of deck allocated to the web EH and FG should be more nearly equal, as shown by the lines A, B and C. This alternately suggests that the transverse position of the steel boxes under the slab can in this case be improved.

3.6 Interpretation of the Output of a Grillage Analysis

A typical output gives the external reactions at each support these results should always be examined first. Computer software usually also gives values of the vertical shear, bending moments and torsional moment for each grillage member on both sides of each joint in the grillage. Care should be taken in deducing design values for the real members from these results. The values midway between two adjacent nodes are usually more representative of the real values.

The bending and torsional moments, in general, show a discontinuity at each joint, Figure 5. For an orthogonal grillage, each change in bending moment is equal to the change in torsional moment at that joint in the member at right angles to the one considered. Similarly, the change in torsional moment equals the change in bending moment in the perpendicular member.

If the values midway between two adjacent nodes are not directly available, the design bending and torsional moments at each joint can be taken as the mean of the values on each side of the joint, for the member considered.

4. ORTHOTROPIC PLATE ANALYSIS

In orthotropic plate analysis, the deck structure is 'smoothed' across its length and breadth and treated as a continuum.

The elastic properties of an orthotropic plate are defined by the two flexural rigidities D_x and D_y and a plate torsional rigidity H. The governing equation relating deflection w to load P acting normal to the plate of the plate is:

$$D_x \frac{\delta^4 w}{\delta x^4} + 2H \frac{\delta^4 w}{\delta x^2 \, \delta y^2} + D_y \frac{\delta^4 w}{\delta y^4} = p(x, y)$$

Design charts for decks that can be idealised as orthotropic plates have been derived from series solutions. They give deflections and longitudinal and transverse moments due to a point load, and so provide a rapid method for distribution analysis. Their applicability is limited to simply supported decks of skew not exceeding 20° whose elastic properties can be represented solely by length, breadth, and the three quantities D_x, D_y and H.

In composite structures, they can be used for beam-and-slab decks with not less than five equally spaced longitudinal members of uniform diaphragms over the supports.

In practice, the method has often been superseded by grillage analysis.

5. FOLDED PLATE ANALYSIS

The folded plate method is normally limited to assemblages of rectangular plates. It is not applicable to skew decks due to coupling between the harmonics. The orthotropic plates may extend over several spans but must be simply-supported at the extreme ends, with rigid diaphragms over the end supports. When folded plate diaphragms are used to represent the transverse frames, the advantages are that it can give a complete and accurate solution in much less computer time than is needed for the finite element method, and it can accept a wide variety of types of loading and both displacement and force boundary conditions.

5.1 Folded Plate Analysis: Beam on Elastic Foundation

If the deformations of the cross section of a single box girder bridge are assumed to be concentrated in the corners of the box girder, an analogy between this folded plate analysis and beam theory exists. The cross frames or the diaphragms transform the effect of eccentric loads into uniform of Saint Venant torsional shear stresses by resisting the resulting distortion. They become therefore the elastic supports for the box girder representing beam. This simplification offers an economic way of computing the internal forces in the cross frames of the box girder, as well as the distortion warping stresses.

To apply the same method to a double cellular box-girder bridge with one single internal web, the distortion must be divided into symmetric and asymmetric deformations. For boxes with more internal webs, it is possible to divide the deformations of the cross-section into eigenvalue functions of deformation.

6. FINITE ELEMENT ANALYSIS

The finite element method is used increasingly in civil engineering. It is the most versatile of the matrix stiffness methods of elastic analysis, and can, in principle, approach the solution of almost any problem of global analysis of a bridge deck. The finite element method is not only used for the study of statical problem, but for dynamic problems involving elasticity and/or plasticity. The finite element method allows the study of shear lag and the computation of effective flange breadths. It is also useful to study local effects in slabs.

Its disadvantage is its cost, especially because of the high level of expert time required for the idealisation of the structure. The expert's know-how is needed in selecting an appropriate pattern of elements, in determining the right limit conditions for boundary nodes along the supports or the symmetry axes, and in interpreting results. The choice of inappropriate elements can be misleading in regions of steep stress gradient, because the conditions of static equilibrium are not then necessarily satisfied. The selection of the discretisation density level, or of the material behaviour, may have serious repercussions on the accuracy of the results.

Other problems are that the method is generally not adapted to compute deformations and fabrication cambers.

Manual preparation of computer input and interpretation of output takes so much time that procedures for mesh generation and the plotting of trajectories of principal moment or stress are now incorporated into computer programs. Plotting procedures for the results may dangerously lead to underestimate of the maximum values of some effects, by computing non-representative average values.

Since a high level of expert judgement and experience are needed for use of the method, it should have little application in routine design work and is not therefore discussed further here.

7. CROSS-SECTION DISTORTION

The distortional loading causes the cross-section of the box to change shape by bending of its walls in the transverse plane. The associated transverse bending and shear stresses are usually acceptable in the relatively thick walls of concrete boxes. However, in steel and steel-concrete composite boxes, diaphragms or cross frames may be required to control the distortion. Longitudinal distortional warping stresses also occur. In steel and steel-concrete composite beams, their relative size depends on the spacing of the cross frames.

7.1 Force Computation in the Diaphragms

The cross frames or the diaphragms transform the effect of eccentric loads into uniform of Saint Venant torsional shear stresses resisting the resulting distortion. In this way, the rigidity of the cross frames or diaphragms limits the deformations of the girder section in the transverse plane.

The distortion effects are combined with the local bending and shear in deck slabs in the vicinity of point loads. The calculation of local bending stresses in plates is simplified by the use of influence surfaces. Influence surfaces for simply supported plates tend to give high bending stresses in box girders because they do not consider the true boundary conditions of the plate panel.

- **simple method**

  The method often used in practice consists of isolating a segment of the box girder bridge with a length is equal to the distance between two adjacent diaphragms or cross-frames. One diaphragm or cross frame is located in the centre of this segment. All the external loads applied to the segment are assumed concentrated on the cross frame. The segment is also assumed to exchange uniform Saint Venant torsional shear stresses, as well as bending shear forces with the rest of the bridge at both ends of the segment. All these shear stresses are also concentrated on the cross-frame.

  The diaphragm or cross-frame, is then calculated under the equilibrium set of actions. Their resulting dimensions are usually economic and acceptable.

- **refined method**

  The distortion forces due to the loads applied to the segment described above are resisted in fact by more than a single diaphragm or cross frame. Analysis as a beam on an elastic foundation is one way of computing the distribution of the distortion loads over the different diaphragms or cross-frames considered as elastic supports.

Figure 6 shows the distortion force which will be reduced in the refined method. For the simplicity of the example, the one box girder section is only loaded by two opposite forces applied to the top corners.

Figure 7 shows a parametric study of the influence of diaphragm separation for a simple box.

Figure 8 shows the distribution of the distortion forces in the different cross-frames of a four-span bridge in one of the two main spans of 80 m. The results were obtained from a folded plate analysis using diaphragms to represent the frames.

Figure 9 shows the warping stresses along the Cheviré Bridge. This isostatic bridge is a 162m length single span across the River Loire. The influence of two additional bracings has been studied here in order to demonstrate that additional transverse bracings were ineffective.

7.2 Diaphragms on Piers

The local effects of reactions at bearings cause complex states of stress in the support diaphragms and stress concentrations in the adjacent webs. The cross frame is usually a diaphragm on piers to obtain the greatest rigidity to resist distortion. If the bearings on the piers are under the diaphragm, care must be taken to avoid dilation problems due to longitudinal eccentricity occurring at the bearings. Dilation is a particular problem with steel diaphragms which suggests that concrete or composite members could be used more often in this location, where extra weight is easily carried. However this consideration is not the only one to take into account in developing the concept of these elements. In addition they must resist corrosion during their life, and allow the regular inspection of the welded connections.

8. SHEAR LAG

In very wide flanges, shear lag effects, which are neglected in the global analysis, have to be taken into account for the verification of stresses, especially for short spans.

Shear lag must be considered when verifying sections and calculating stresses, since it frequently causes the longitudinal stress at a flange/web intersection to exceed the mean stress in the flange by 20%.

In calculations based on the elementary theory of bending, an effective flange breadth less than the real breadth can be used. This effective flange breadth depends on the ratio of width to span.

For a simply supported beam, for example, the effective breadth Φ_e.b of the portion between the webs is given in Table 1:

where

b is the distance between webs.
L is the span of the beam
$\alpha = \dfrac{\text{Cross-section area of flange stiffeners in width b}}{\text{Cross-section area of flange plate in width b}}$
Φ_e is the elastic effective breadth ratio.

b/L	Mid-span		Quarter-span		Support	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0,00	1,00	1,00	1,00	1,00	1,00	1,00
0,05	0,98	0,97	0,98	0,96	0,84	0,77
0,10	0,95	0,89	0,93	0,86	0,70	0,60
0,20	0,81	0,67	0,77	0,62	0,52	0,38
0,30	0,66	0,47	0,61	0,44	0,40	0,28
0,40	0,50	0,35	0,46	0,32	0,32	0,22
0,50	0,38	0,28	0,36	0,25	0,27	0,18
0,75	0,22	0,17	0,20	0,16	0,17	0,12
1,00	0,16	0,12	0,15	0,11	0,12	0,09

Table 1: **Effective width factor Φ_e for simply supported beams**

9. CONCLUDING SUMMARY

Several methods of global analysis are available:

1. Grillage analysis is the method most often used. It allows a simple idealisation of the structure, and a sure interpretation of the output.

   Particular consideration is required for the choice of grillage, skew bridges, local effects in decks, torsional and flexural rigidities of grillage members, longitudinal grillage members, and the interpretation of the output.

2. Orthotropic plate analysis has a limited application.

3. Folded plate analysis is used to study the effect of the deformations of box-girder sections.

4. Finite element analysis is used increasingly. It is the most versatile of the matrix stiffness methods of elastic analysis.

Loading causes distortion of the box which, in steel and steel-concrete composite boxes may have to be controlled by the use of diaphragms or cross frames.

Forces in diaphragms or cross frames may be calculated by:

- a simple method.
- a refined method.

In very wide flanges shear lag effects have to be taken into account.

10. ADDITIONAL READING

1. Eurocode 3: "Design of Steel Structures", ENV 1993-1-1: Part 1.1, General rules and rules for buildings, CEN, 1992.

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