An old man going a lone highway
Came at the evening cold and gray
To a chasm vast and deep and wide.
The old man crossed in the twilight dim;
The sullen stream had no fears for him.
But he turned when safe on the other side
And built a bridge to span the tide.

“Old man,” said a fellow pilgrim near,
“You are wasting your time with building here.
You never again will pass this way—
Your journey will end with the closing day.
You have crossed the chasm deep and wide,
Why build you this bridge at eventide?”

The builder lifted his old gray head.
“Good friend, in the way that I’ve come,” he said,
“There followeth after me today
A youth whose feet must pass this way.
This stream which has been as naught to me
To the fair-haired youth might a pitfall be.
He, too, must cross in the twilight dim.
Good friend, I’m building the bridge for him.”

Will Allen Dromgoole
This book has been written with the intention of describing the fundamental structural behavior of the most commonly used prestressed concrete bridges. The authors believe the contents of this book will be especially useful to engineers having little or no previous experience in the design of prestressed concrete bridges as well as those whose practice includes an occasional bridge design.

The first chapter is devoted to basic information and serves as a foundation for subsequent chapters.

Chapter 2 is devoted to girder bridges. The authors elected to use this name over "stringer bridge" in view of the fact that the term "stringer" is not applied to beams of reinforced or prestressed concrete in the Standard Specifications for Highway Bridges which is published by the American Association of State Highway and Transportation Officials. This form of concrete bridge has been the type most commonly used in the United States. Its use has been widespread and is expected to continue. Methods of analysis for girder bridges which have been in use in Europe for a number of years are presented in this chapter. These methods have not been
commonly used in this country because they are not usually taught in our universities. In addition, they are not included in the bridge design criteria normally used in this country. The significant effect of well designed transverse beams or diaphragms on the distribution of live loads to the individual girders is emphasized.

Box-girder bridges are treated in Chapter 3. This important form of cast-in-place construction has been widely used in the western United States. Its use in other parts of the country is increasing and is expected to reach very significant levels in the next few years. The importance of the torsional stiffness of the box-girder cross section is explained as is its effect on the distribution of flexural stresses due to live loads.

A relatively new form of concrete bridges has been treated in Chapter 4. It has been referred to as a segmental box-girder or a segmental bridge in this book. Design considerations and construction techniques unique to this mode of bridge construction are treated in detail. This chapter contains information that should be of value to experienced bridge designers as well as to those without extensive experience.

The additional design considerations of Chapter 5 and the construction considerations of Chapter 6 have been included as a means of calling the reader’s attention to a number of factors requiring consideration in formulating a complete bridge design. Some of the subjects included may not be new or may be apparent to some of the readers. Others will find these chapters convenient sources of reference from time-to-time.

The authors wish to acknowledge the technical information and photographs that have been provided by the French engineering firm, Europe Etudes. In particular, the contributions of Jean Muller and Gerard Sauvageot are acknowledged with sincere thanks. The authors also wish to thank the publishing firm of Springer-Verlag for permission to publish the influence surface charts reproduced in this book as Figures 1.2 through 1.5 and Jacob Dekema of the California Department of Transportation for the excellent photographs of bridges designed and constructed under supervision of the Department.

San Diego, California
September, 1975

James R. Libby
Norman D. Perkins
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Index
1 Introduction

1.1 Scope of Book.

This book has been written with the principal purpose of describing the design methods that are applicable to the various major types of prestressed concrete highway bridge superstructures currently in use in the United States. Secondary purposes have been to describe the advantages and disadvantages of the various bridge types and to briefly discuss the construction methods used with the different types.

Reinforced concrete bridge superstructures are not considered. The basic principles of elastic design which are discussed in this book are, however, equally applicable to reinforced concrete and prestressed concrete.

Bridge substructure design is considered only as it affects the design of the bridge superstructure or the bridge as a whole.

The fundamental principles of reinforced concrete and prestressed concrete structural design are not presented in this book. It is presumed the reader is competent in the design of these forms of concrete construction (Ref. 1,2).* In addition, it is presumed the reader is familiar with the

---

*1, 2, etc. refer to references listed in the back of this book.
strength, elastic, creep and shrinkage properties of \textit{Portland} cement concrete as well as with the properties of ordinary reinforcing steel (Ref. 3) and the steels commonly used in the United States in prestressed concrete construction (Ref. 4,5). Finally, it is presumed the reader is familiar with the fundamental principles of structural analysis.

Not all types of prestressed concrete bridge superstructures used or proposed for use in the United States have been included in this book. Bridges which employ precast members used primarily in building construction, such as double-tee beams, single-tee beams, hollow-core slabs and solid precast slabs, are discussed briefly, but are not treated in detail. No attempt has been made to include a discussion of unique bridges utilizing specially fabricated precast sections peculiar to the specific bridge even though the bridge might be considered to be a major structure. Many of the fundamental principles discussed in this book are, however, equally applicable to structures of these types.

Specific cost data have not been included in the discussions of the various types of bridges. Construction costs vary with the constantly changing economy of the nation. The result is that specific cost data are normally only accurate for a short period of time. Relative construction costs vary throughout the country and hence escape anything but vague generalizations.

\section*{1.2 Design Criteria.}

The most widely used criteria for the design and construction of highway bridges in North America are contained in the “Standard Specifications for Highway Bridges” (Ref. 6) published by the American Association of State Highway and Transportation Officials.* These criteria, which are referred to subsequently in this book as the “AASHTO Specification”, or simply as “AASHTO”, are used as the basic criteria for design except where otherwise stated.

The design criteria pertaining to reinforced and prestressed concrete contained in the AASHTO Specification are based to some degree upon the American Concrete Institute publication “Building Code Requirements for Reinforced Concrete” (\textit{ACI 318}) (Ref. 7). This publication is referred to subsequently as \textit{ACI} 318. In some instances specific references to this publication are made in the AASHTO Specification. This \textit{ACI} publication, which is under constant review and frequent revision, reflects

*Previous to the year 1974, this organization was known as the American Association of State Highway Officials.
the best contemporary thinking relative to the design of concrete structures.

The designer of prestressed concrete bridges should be familiar with the provisions of the latest editions (with interim modifications) of both the AASHTO Specification and ACI 318. His design should incorporate the provisions of these publications which will result in a safe structure that behaves in a predictable manner.

ACI Committee 443, Concrete Bridge Design, has published two portions of what eventually will become a complete recommended practice for the design of concrete bridges (Ref. 8,9). These publications are highly recommended to all who are interested in the design of concrete bridges.

1.3 Design Loads.

Like other structures, bridges must be designed for the dead and live loads to which they are subjected.

The dead loads consist of the self-weight of the basic structural section itself as well as superimposed dead loads such as bridge railings, sidewalks, non-structural wearing surfaces, and utilities which the structure must support. Dead loads can generally be estimated with a high degree of accuracy during the design, accurately controlled during the construction and are normally considered to be permanent loads. Due to their more or less permanent nature, loads resulting from concrete volume changes are sometimes categorized as dead loads.

Live loads are those due to the effect of external causes and are generally transient in nature. Live loads include those resulting from vehicles and pedestrians which pass over the bridge as well as the forces resulting from wind, earthquake and temperature variation. Other live loads are secondary in nature and result from impact forces. Vertical impact forces are created by the vehicles using the structure. Horizontal impact forces result from braking and turning of these vehicles. The live loads that will be imposed upon a structure cannot generally be estimated with the same precision as can the dead loads. In addition, the designer often has little if any control over these loads once the structure is put into service.

The minimum live loads for which bridge structures must be designed are generally specified by design criteria such as the AASHTO Specification. Considerable differences exist in the live load design criteria used throughout the world. Much has been written on this as well as on the fact that the criteria used in the United States may be unrealistically low and may not be representative of the actual loads to which our bridges are exposed (Ref. 10,11). From these discussions the bridge designer should keep two facts in
mind. These are: (1) the live load requirements specified by the AASHTO Standard Specification are among the lightest loadings used in the world; and (2) these live load requirements may be lower than the maximum loads one might expect on a highway bridge in the United States.

It may very well be that other requirements of the AASHTO Specifications compensate to some degree for the relatively light design live loads specified therein. Some engineers feel the day has come for the AASHTO Standards to be materially revised with a view toward specifying’ more realistic truck loadings as well as encouraging more sophisticated methods of bridge design and analysis. If the design live loads of the AASHTO Standard Specification are too low, they should be increased so that elastic analyses will yield reasonable agreement with what is actually occurring in real bridges. One should not rely upon the conservatism of empirical coefficients to compensate for inadequate load criteria. This is especially true when strength rather than service load design methods are used.

The design loads that must be considered in the design of reinforced concrete and prestressed concrete are identical except for those caused by volume changes. The effect of concrete shrinkage is less in the case of reinforced concrete than in the case of prestressed concrete. This is due to the fact that non-prestressed reinforcing steel tends to resist concrete shrinkage strains and, in reinforced concrete members, promotes the formation of fine cracks. The fine cracks relieve the shrinkage stresses in the concrete as well as the need for the member to shorten. The important effect of concrete creep on reinforced concrete members is the time-dependent effect on deflection. In prestressed concrete the cracking mechanism related to shrinkage does not take place and provision must be made for the total shrinkage strain which may occur and cause undesirable effects. In prestressed concrete creep and shrinkage both affect deflection. This must be considered in the design. Shortening due to creep and shrinkage can be significant in prestressed concrete structures and must be taken into account if good results are to be obtained.

Although there are considerable data in the literature relative to creep and shrinkage of concrete, there is no accepted U.S. recommended practice for estimating the magnitude of the creep and shrinkage strains the designer should accommodate in his design. Methods have been proposed in the literature (Ref. 12, 13) but these have not achieved the status of a standard or recommended practice. For the benefit of the reader, the methods used for predicting concrete shrinkage and creep in the French Code (Ref. 14) are included as Appendix A of this book.

Due to the complexity of the live load criteria given in the AASHTO Specification, these provisions will not be repeated in this book. Most bridges are designed for the AASHTO HS20-44 live load. Live loads of
lower magnitude than HS20-44 are provided in the AASHTO Specification. The smaller live loads were originally intended for use on secondary roads but not on primary highways. Because there is virtually no practical way of insuring that the largest trucks will not be used on secondary roads, many jurisdictions use the HS20-44 loading in all bridge design.

The AASHTO Specification stipulates that a truck or lane loading shall be assumed to occupy a width of ten feet. Each 10-foot wide truck or lane load is to be positioned in a design traffic lane which is twelve feet wide. It is further stipulated that the number of design traffic lanes shall be two for bridges having roadway widths between curbs of from 20 to 24 feet. For roadway widths over 24 feet, each design traffic lane is assumed to occupy a width of 12 feet. The twelve-foot width traffic lanes are to be positioned in such a manner as to produce the maximum stress in the member under consideration. For bridges which are designed for three or more design traffic lanes, Section 1.2.9 of the AASHTO Specification provides load intensity reduction factors which are intended to account for the improbability of all lanes being frequently loaded simultaneously. The location of the specific live load-related requirements of the AASHTO Specification are summarized in Table 1.

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1.4 Design Methods

Empirical coefficients are included in the AASHTO Specification for determining the live load moments for which concrete deck slabs are to be designed as well as for determining the distribution of live loads to the girders which support the concrete slabs. The use of the empirical coeffi-
sients is not mandatory but they are given for use when more sophisticated methods of analysis are not used.

The live load distribution factors which are contained in Section 1.3.1 of the AASHTO Specification, are based upon the assumption a bridge can be divided into several longitudinal beams for the purpose of design and analysis. A bridge is, of course, a three dimensional structure and should be designed with this being taken into account. Approximate elastic methods of analysis, which consider the superstructure as a whole, are presented in Chapters 2, 3 and 4 for use in the design of bridge superstructures which are narrow with respect to their span as well as relatively deep with respect to their width. The approximate methods are applicable to most conditions encountered in practice.

Sophisticated methods of analyzing bridge structures including the
folded plate method, the finite segment method and the finite element method are described in the literature (Ref. 15). These methods result in higher precision in the determination of the stresses and deflections than can be obtained by use of the familiar flexural theory. These methods have a place in structural research and in the design of special structures but their use is not needed nor considered to be practical in normal design work. The slightly greater accuracy in determining stresses with these methods in bridges of normal proportions is not significant when one considers the differences between the loads used in design and the actual loads to which a structure can be subjected. The cost of employing the more sophisticated methods as a design procedure is prohibitive in most cases.

For many years the design of bridge decks has been done using empirical relationships contained in the AASHTO Specification. Relationships which are based upon the work of H. M. Westergaard (Ref. 16), are given for slabs which have their main span perpendicular to the direction of traffic as well as for slabs which have their main spans parallel to the direction of traffic.

A major revision of the empirical relationships for the design of bridge slabs occurred in the interim between the 1957 and 1961 AASHTO Specifications. A comparison of the design requirements for simply supported slabs having their main reinforcement perpendicular to the direction of traffic according to the 1957 and 1961 AASHTO Specifications for HS20-44 live loads without impact is given in Fig. 1.1.

Two basic design deficiencies exist with these empirical relationships. The first of these is the lack of provisions to account for the differences between the distributions of positive and negative moments in members having constant and variable depth. The second is the moment continuity coefficient of 0.80 which is specified for decks which are continuous over three or more supports regardless of the elastic restraints which are provided by the various members which are connected at the supports of the deck.

The empirical relationships of the AASHTO Specifications have proved to be satisfactory for the decks of bridges which are supported by torsionally flexible stringers that may or may not be connected together with flexurally stiff transverse diaphragms. Hence, these relationships can be considered to be conservative for all structural schemes. The relationships may, however, be overly conservative with respect to the moments in decks that are supported by torsionally flexible stringers connected with flexibly stiff intermediate diaphragms or which are supported by torsionally stiff systems. Additionally, the empirical relationships do not alert the designer to the importance of considering the live load deck moments.
Fig. 1.2 Influence surface for the midspan moment in the x-direction of a plate strip of constant depth with two restrained edges and Poisson's ratio = 0 (8 \( \pi \) times the actual values shown). (Courtesy Springer-Verlag).
induced in the webs of flexurally stiff supports. Hence, in this respect they are unconservative.

In view of the above, elastic design methods are recommended for decks of most concrete bridges.

Charts of influence surfaces, which can be thought of as being similar to two-dimensional influence lines, are available for the determination of moments, shears and deflections for slabs having a variety of dimensions and boundary conditions (Ref. 17, 18). Examples of the charts are given in Figs. 1.2 through 1.5. The charts are used by plotting the “footprints” of the applied wheel loads, adjusted to the proper scale, on the charts and computing the volumes defined by the area of the “footprints” and the ordinates of the chart. The sum of the products of the volumes and their respective loads is equal to the moment, the shear or the deflection coefficient for which the chart has been prepared. The charts are prepared with the assumption that Poisson’s ratio, for the material of which the slab is composed, is equal to zero. Hence, a correction factor must be applied to the computation to correct for this assumption. The instructions which are included with the charts explain how this correction should be made. The charts are based upon an elastic analysis. The moments or shears computed by use of the charts are expressed per unit of length (i.e. Kip-feet per foot or kips per foot for moment and shear respectively) at the location in the slab for which the chart was prepared.

The charts presented by Pucher are for slabs of constant depth only while those prepared by Homberg include charts for slabs of constant as well as variable depth.

The use of influence charts permits the designer to take the effects of variable slab thickness into account. In addition, because the charts are based upon rational elastic analysis, they permit the designer to analyze the effects of joint restraint. This is accomplished by determining the fixed-end moments for the critical conditions of loading and distributing them to the supporting members in accordance with normal elastic design procedures. As was stated above, the empirical relationships for the design of bridge decks which are contained in the AASHTO Specifications do not give the designer a basis for taking either of these factors into account.

The design span to be used in the design of prismatic solid slabs which are constructed monolithically with their supports is generally taken as the clear distance between supports. This assumption is limited to spans of 10 feet or less by the ACI Building Code Requirements for Reinforced Concrete (Ref. 19) but not by the AASHTO Specification (Ref. 20). For spans in excess of 10 feet, according to the ACI Requirements, one should base

*Courtesy of Springer-Verlag.*
Fig. 1.3  Influence surface for the moment at the support in the x direction for a cantilevered plate strip of constant depth and Poisson’s ratio = 0 (8 times the actual values shown). (Courtesy Springer-Verlag).
Fig. 1.4 Influence surface for the moment at the support in the x direction for a plate strip of variable depth (parabolic) with two restrained edges and Poisson’s ratio = 0. (Courtesy Springer-Verlag).
Fig. 1.5 Influence surface for the support moment in the x direction for a cantilevered plate strip of variable depth (constant variation from d to 2d) and Poisson's ratio = 0. (Courtesy Springer-Verlag).
the determination of moments upon center-to-center distances between joints but use the moments computed at the faces of supports in designing the slab for strength.

Little guidance is to be found in the usual structural design criteria employed in the United States relative to the design span to be used when haunched members are employed. The definition of the design span permitted by the French Code is shown in Fig. 1.6 for various conditions of haunches (Ref. 21). The French Code limits these definitions of design spans to spans of 6 meters (19.7 feet) or less and to slabs which are supported on their entire (or nearly so) perimeter and which are subjected to large transient concentrated loads. The complete provisions of the French Code relative to the design of slabs is recommended for reading but is not reproduced here.

1.5 Allowable Stresses

The allowable stress criteria that are followed in a structural design obviously have a major influence on the results. If, for example, no flexural tensile stresses are to be permitted in a prestressed concrete flexural member under full dead, live and impact loading, significantly more pre-stressing steel may be required in the member than would be required if the allowable stresses permitted by Section 1.6 of the AASHTO Standards were followed.

In spite of the fact that Section 1.6.6 of the AASHTO Specifications permits tensile stresses in prestressed concrete members, some engineers feel tensile stresses should not be permitted in certain instances. One of these is in the design of bridge decks. It has been argued that wheel loads in some instances, whether legally or illegally, exceed the design loads specified by the AASHTO Specification. If tensile stresses were permitted when designing for the AASHTO wheel loads, the tensile strength of the concrete could be exceeded if the slabs were subjected to wheel loads greater than the AASHTO wheel loads. Some engineers believe that segmentally constructed bridges, which are treated in detail in Chapter 4, should be designed as Class I structures.* This opinion is based upon the belief that the many joints in segmental bridges makes them more susceptible to deterioration than is the case for monolithic structures and hence more conservative stresses are indicated. In addition, the redistribution of

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*Current European practice is to categorize structures into three classes. The distinguishing factor is the allowable tensile stress and hence degree of cracking, which is permitted. Tensile stresses are not permitted in Class I structures. Tensile stresses as high as the tensile strength of the concrete are permitted in Class II structures. Tensile stresses which exceed tensile strength of the concrete are permitted in Class III structures (Ref. 22).
moments which occurs due to concrete creep in segmental bridges that are erected in cantilever is sometimes estimated by approximate calculation rather than being determined with precision and hence conservatism in areas of positive moment seems appropriate. Finally, the existence of a temperature differential between the deck and girders of a girder bridge or between the top slab and the remaining portions of a box-girder bridge results in the creation of moments and hence flexural stresses in a structure. The stresses due to differential temperature in a simple-span structure may be of relatively nominal magnitude and are directly dependent upon the magnitude of the temperature differential. In the case of continuous structures, it can be shown that nominal temperature differentials can result in flexural tensile stresses as great as 500 psi and can result in significant temperature induced variations in the reactions at the beam
supports (Ref. 23,24). Stresses as great as these should be considered in service load analyses of bridges in which flexural tensile stresses are permitted under live and impact loads. Methods of evaluating the effect of temperature variations within a section are given in Appendix D.

One should bear in mind that the provisions of the AASHTO Specifications, like most design criteria engineers are required to observe in their work, constitutes a minimum criteria. An engineer, using his own judgement and experience as a basis, may wish to follow more conservative criteria in his work.

1.6 Bridge Types Considered

Three types of bridges are considered in detail in this book. The design considerations unique to each of the three methods of construction are treated in Chapters 2, 3 and 4. Construction considerations for each of the three types of bridges are treated in Chapter 6. The bridge types considered in detail are referred to as girder bridges, box-girder bridges and segmental
bridges. Other types of prestressed concrete bridges are not discussed in detail either because they will behave structurally in a manner that is similar to one of the "basic" three bridge types or because their design is normally accomplished with sufficient accuracy using the empirical relationships included in the AASHTO Specification.

Girder bridges are bridges which incorporate two or more longitudinal beams together with a deck slab spanning transversely over or between the top flanges of the girders. The deck slab is normally connected to the top flanges of the girders in such a manner that it acts compositely with them in resisting a portion, if not all, of the longitudinal flexural stresses. It is not essential that the deck slab acts compositely with the girders. Girder bridges are frequently constructed as simple spans. They are also frequently constructed continuous over two or more spans. Bridges of this type have found wide use in North America.

Typical cross sections of two types of girder bridges are shown in Figs. 1.7 and 1.8. The first of these is typical of cast-in-place construction and is often referred to as a "T-beam" structure. (Rectangular precast beams can be used, in combination with a cast-in-place deck slab, to form a bridge of this type.) The longitudinal beams are normally considered to have effective cross sections which are T-shaped in the analysis for longitudinal flexure. The second cross section (Fig. 1.8) is typical of bridges formed of precast I-shaped or T-shaped beams together with a cast-in-place deck.

An important characteristic of the beams used in bridges of this type is their relatively small torsional stiffness. When sophisticated methods of
analysis are used to analyze girder bridges, the torsional stiffness of the beams is normally neglected. This is treated in greater detail in Chapter 2.

Transverse beams, which are called end diaphragms, are normally provided at each support of girder bridges. The end diaphragms connect the longitudinal girders to each other as well as to the deck and provide an efficient means of transferring lateral loads, acting upon the superstructure, to the substructure. The end diaphragms also prevent movement of the ends of the beams with respect to each other. Typical end diaphragm details are shown in Fig. 1.9.

Diaphragms are usually provided between the girders at one or more locations between supports. These diaphragms are termed intermediate
diaphragms. Intermediate diaphragms are frequently not used in bridges having spans of 40 feet or less. Commonly used details for intermediate diaphragms in bridges utilizing I-shaped beams are given in Fig. 1.10. (See Section 2.3)

A typical cross section for a box-girder bridge is shown in Fig. 1.11. A considerable number of bridges of this type has been constructed in the United States. This is especially true for the Western United States.

Box-girder bridges are cast-in-place on falsework. Superstructures of two or more spans are normally made continuous over the interior supports. They are frequently constructed with fixed connections between the superstructure and the abutments, piers or bents and hence form a frame. The use of frames has been considered important in regions where earthquakes might be expected.

Construction joints are normally provided in box-girder bridges near the junction between the webs and the upper slab as shown in Fig. 1.11. The bottom slab and web stems are usually constructed at one time. After the formwork for the interior webs has been removed, forms for the upper deck are installed and the upper deck is constructed. The forms used for the upper deck at interior cells are generally left in place.

End diaphragms are used with box-girder bridges as they are for girder bridges and for the same reasons. Although intermediate diaphragms have been used on many box-girder bridges, they serve little if any useful function (except in structures having significant horizontal curvature) due to the great torsional stiffness as well as the transverse flexural stiffness of the box-girder section. The fact that intermediate diaphragms are not needed in box-girder bridges is currently recognized by many bridge engineers and their use is expected to diminish rapidly in the future (Ref. 25,26).

The torsional stiffness of the box-girder bridge superstructure as well as the transverse flexural stiffness are important structural characteristics of this mode of bridge construction. This is considered in greater detail in Chapter 3.

Specific forms of box-girder bridges are referred to in this book as “segmental box-girder bridges” or as “segmental bridges”. Some may feel this distinction is not necessary or justified. The authors believe the distinction is not only justified but necessary at this time if the full potential of this form of bridge construction is to be realized. The use of a special name for this mode of construction will facilitate calling the attention of the designer to the fact that the load distribution factors of Section 1.3.1 (B) as well as the minimum slab thickness and diaphragm provisions of Section 1.6.24 (C) and (F) of AASHTO Specification either cannot or should not be applied to segmental bridges. Additionally, bridges which fall under the
classification of segmental bridges, as used in this book, have traditionally been erected using the cantilevering technique or other special erection techniques which require special engineering analysis on the part of the designer. The difference in design considerations between box-girder bridges which are constructed in place on falsework and segmental bridges which are erected with more sophisticated techniques will be brought to the attention of the designer by this distinction in terminology.

Typical cross sections for segmental bridges are shown in Fig. 1.12 through 1.19. An examination of these cross sections will reveal that the superstructures of the first three (Figs. 1.12, 1.13 and 1.14) are single-cell tubes of constant depth. All three of these bridges were constructed using precast segments. The superstructure of the Pine Valley Creek Bridge (Fig. 1.15) has two single-cell constant depth tubular girders which are structurally independent between the supports. The Pine Valley Creek Bridge was constructed with the balanced cantilever technique with segments that were cast-in-place on traveling forms supported by the previously constructed superstructures. The Napa River Bridge superstructure is shown in Fig. 1.16 from which it will be noted the depth of the two-celled superstructure is variable. The Napa River Bridge, which was under construction at the time this book was written, was being cast-in-place on falsework using the balanced cantilever erection method. The cross section of the B-3 Viaduct in Paris consists of two constant depth precast
Fig. 1.13 Typical cross section, Bear River Bridge, Nova Scotia.

Fig. 1.14 Typical cross section, Bridge on U. S. Route 50 over the Vernon Fork of the Muscatatuck River, Indiana.
single-cell tubular girders which are connected together transversely by a common slab span. The typical section for the B-3 Viaduct is shown in Fig. 1.17. The variable depth superstructure of the Saint André de Cubzac Bridge is shown in Fig. 1.18 from which it should be noted that the upper deck spans longitudinally between floor beams in the precast segments, rather than transversely between the webs as is more commonly the case.

By far the most commonly used cross section in the construction of segmental bridges is the single cell. The single cell sections may or may not be connected together with a common middle slab as shown in Fig. 1.17. The result is that the typical segmental bridge has considerably fewer webs than has been the case with the typical box-girder bridge. It is expected the number of webs used in box-girder bridges will decline in the future as a result of bridge designers comparing the structural efficiency of the cross sections of typical box-girder and segmental bridges. Cross sections with as many as four cells and without deck overhangs have been constructed using precast segments and the balanced cantilever erection method. The Saint Cloud Bridge over the Seine in Paris, which is shown in Fig. 1.19, is such a structure. The cross section of the Saint Cloud Bridge was not selected for its structural efficiency but for its effect on the appearance of the completed structure.
Fig. 1.16 Typical cross section, Napa River Bridge, California.

Fig. 1.17 Typical cross section, B-3 Viaduct, Paris, France
Segmental superstructures also have high torsional stiffness, but not generally as high as that of box-girder superstructures. The torsional stiffness of the tubular girders is an important design consideration as will be seen in Chapter 4.

Intermediate diaphragms are not used in superstructures incorporating tubular segmental girders. Provision of intermediate diaphragms in a single-cell superstructure may or may not enhance the structural performance of the structure (Ref. 27). Experience has shown that the torsional stiffness of the tubular girders renders the provision of intermediate diaphragms unnecessary in superstructures consisting of two or more tubular girders which are connected together by their top decks (Fig. 1.17). (See Section 4.4).

Diaphragms, of solid or open construction, are almost invariably provided in segmental bridges at both the abutments and intermediate points of support. These diaphragms are normally needed to transfer moments and shears from the superstructure to the substructure. In small grade-separation structures, diaphragms have been omitted at the intermediate bents and been provided at the abutments alone. Special construction details must be provided for transferring loads and moments from the superstruc-

![Fig. 1.18 Typical cross section, Saint André de Cubzac Bridge, France.](image)
ture to the substructure when diaphragms are not provided at the intermediate points of support.

Concrete slab bridges are normally used only for short spans because of reasons of economy. Slab bridges can be constructed with precast prestressed concrete components together with a cast-in-place topping or might be entirely cast-in-place. Slab bridges require the least construction depth of all the types of concrete bridges. Slab bridges are sometimes used in lieu of other bridge types because of the minimum construction depth associated with their use.

Typical cross sections of slab bridge superstructures are given in Figs. 1.20 and 1.21.

Slab bridges can be designed using the empirical relationships found in Section 1.3.2 of the AASHTO Specifications, using the influence charts of Pucher or Homberg (Ref. 17,18), or other methods of elastic analysis (Ref.
Due to the relative flexibility of slabs, the transverse bending moments which result from the application of concentrated loads must be considered in their design.

A bridge superstructure composed of longitudinal beams placed side-by-side as shown in Fig. 1.22 is referred to as a multi-beam bridge in the AASHTO Specification. Although bridges of this type could be formed of I-shaped beams, they most commonly are made with precast, pretensioned hollow-box beams as shown in Fig. 1.22.

The precast hollow-box beam possesses considerable torsional stiffness and hence rotates only slightly if loaded eccentrically. It is essential that the ends of the beams of multi-beam bridges be connected together with transverse ties extending across the structure. It is also essential that shear-transferring devices be provided between the individual beams. If adequately connected together transversely by the provision of reinforcing which extends completely across the structure together with shear connectors of sufficient size and shape, bridges of this type will behave structur-
ally in much the same manner as a monolithic box-girder bridge. If adequate intermediate transverse ties are not provided, the joints between the beams will tend to open under the application of concentrated live loads. This is due to the lack of tensile strength along these joints together with the flexural and torsional distortions of the beams. An excellent, although relatively costly, means of attaining an adequate transverse tie consists of providing post-tensioned tendons transversely at each end as well as at one or more intermediate points along the span.

Empirical relationships for the distribution of live loads to multi-beam bridges are given in Section 1.3.1 (D) of the 1974 AASHTO Interim Specifications (Ref. 6). The theoretically correct distribution of live loads to the beams of multi-beam bridges is dependent upon the adequacy and efficiency of the shear connectors and transverse ties between the individual beams. The empirical relationships found in the AASHTO Specifications are considered adequate for all commonly encountered conditions of span, width and loading. A sophisticated elastic analysis of multi-beam bridges would probably not result in sufficient savings (if any) to justify the extra engineering effort required to make the analysis.

Although the use of multi-beam bridges incorporating precast box beams has been extensive in several parts of the country, they have not proved to be economical in many other parts of the country.

The bridge superstructure formed of several precast box beams placed parallel to each other at a spacing which exceeds their widths and which are connected together with a cast-in-place deck, is referred to in the AASHTO Specification as a spread box-beam bridge. A typical section for a bridge of this type is shown in Fig. 1.23. Provisions for the distribution of live loads to spread box-beam bridges are given in Section 1.6.24 (A) of the
1974 AASHTO *Interim Specification*. The empirical live load distribution factors for spread box-beam bridges were derived by research conducted on actual bridge superstructures.

End diaphragms must be provided with bridges of this type in order to transfer lateral loads to the abutments and to prevent relative movement between the beams at their ends. The provision of adequate intermediate diaphragms in spread box-beam bridges reduces the live load induced torsional rotations of the beams as well as the differential deflections between beams and hence reduces the moments induced in the connecting slabs. If intermediate diaphragms are not provided, moments will be induced as a result of the torsional rotations and vertical deflections of the beams when subjected to live load. These moments can be evaluated using the methods described in Chapter 4 for segmental bridges having two
girders with a connecting slab. If one does not wish to evaluate these moments, the empirical slab design relationships of AASHTO Section 1.3.2(C) can be used.

Bridges are sometimes constructed using precast concrete members of types normally used in building construction. Typical sections for bridges of these types are shown in Figs. 1.24, 1.25 and 1.26. These structures are usually short-span structures and are provided with end diaphragms and cast-in-place decks which, among other functions, transfer shear forces between the girders. The structural behavior of these types of structures is similar to either girder bridges without intermediate diaphragms or slab bridges.

![Fig. 1.26 Typical cross section, bridge utilizing hollow precast slabs.](image)

1.7 Span Length vs. Bridge Type

The relative economy of the different types of prestressed concrete bridges varies between the various parts of the country as well as with changes in the general economy of the nation. Hence accurate specific conclusions pertaining to the relative economy of bridge type versus span cannot be made. One can, however, conclude from a review of practical applications that bridges of short span will probably be most economical using slab-type structures. Moderate spans are most economically done as girder-type bridges while the longer span structures are generally box-girder or segmental-girder bridges. The longest concrete bridge spans have utilized cast-in-place segmental-type superstructures.
2 Girder Bridges

2.1 Introduction

Girder bridges of two commonly used types were illustrated in Chapter 1. Typical cross sections for girder bridges of these types are shown in Figs. 1.7 and 1.8. It was explained that the negligible torsional stiffness of girders having I-shaped or T-shaped cross sections necessitates the provision of intermediate diaphragms in all girder bridges except for those of relatively short span. It will be shown in this Chapter that the torsional flexibility of the beams in girder bridges has great bearing on their response to load.

The designer of a girder bridge must determine the portion of the dead and live load that is distributed to each of the individual girders in the bridge. This can be done by elastic analysis or by the use of empirical methods. The most exact methods of elastic analysis involve the use of large electronic computers and sophisticated programs. The cost of sophisticated analyses of this type can not be justified except for structures of major importance. For the more commonly encountered conditions, approximate methods of elastic analysis yield adequate results. The empirical methods have proven to yield results which are generally satisfactory from a performance standpoint and are economical for structures of small or modest size.
Consider a multi-girder bridge superstructure, such as is shown in Fig. 2.1, which is intended for use on a span of 120 feet. If the girders are assumed to be without torsional stiffness and if intermediate diaphragms are not provided to connect the girders transversely, the lateral distribution of loads applied to the deck is accomplished by the deck slab. For the application of the dead load due to the safety walks and bridge railing, the structure would be expected to deflect as shown in Fig. 2.2 from which it will be seen that the exterior girders are required to carry more of the load than are the interior girders. For the application of concentrated loads on the deck, the structure would deform as shown in Fig. 2.3 from which it will be seen that the interior girder nearest to the point of application of the load, carries the largest portion of the load. From this it should be apparent that transverse flexibility is an important design consideration for girder bridges that are without intermediate diaphragms.

If an intermediate diaphragm is provided at midspan of the bridge of Fig. 2.1, it may be as shown in Fig. 2.4. It can be shown that the stiffness of the

---

**Fig. 2.1** Typical cross section of a multi-girder bridge superstructure.

**Fig. 2.2** Typical cross section of a multi-girder bridge which does not have intermediate diaphragms and which is subject to the loads of safety walks and railings.
diaphragm is great in comparison to that of the girders for these conditions (span 120 feet, distance between exterior girders, 28 feet, and depth of diaphragm of the order of that of the girders). For this example, the moment of inertia of the intermediate diaphragm is of the order of 533,000 in$^4$ while that of the deck slab alone is 244 in$^4$ per foot. If one were to assume the entire 120 feet of bridge deck effective in distributing loads transversely (it obviously is not) the stiffness of the deck slab would be of the order of one-eighth as much as that of the intermediate diaphragm. Hence the application of the safety walk and railing dead loads result in a
vertical deflection with virtually no transverse bending deflection as shown in Fig. 2.5. The application of an eccentrically applied concentrated load over the midspan diaphragm, will result in a deformation as shown in Fig. 2.6 from which it will be seen the transverse flexural deformation is virtually nonexistent and the exterior beam on the loaded side carries the
greatest portion of the load. The provision of a well designed intermediate diaphragm obviously has important effects on the distribution of loads in a girder bridge, such as this one.

For girder bridges which are wide in comparison to their span, even though they may have intermediate diaphragms, transverse elastic deflection resulting from the application of concentrated loads can be significant and should not be ignored. Such superstructures act in a manner that is not unlike the action of an elastic plate that is supported on two edges (their supports).

Consider the special case of a long span bridge which incorporates only two torsionally flexible girders as shown in Fig. 2.7. Concentrated loads applied to the deck of such a structure are distributed to the girders as if the deck were a simple span over two supports. The provision of an intermediate diaphragm at midspan of the structure will not change the distribution of loads to the girders. The provision of additional intermediate diaphragms will not change the distribution of loads to the girders nor change the shape of the deflected girders either transversely or longitudinally. From this simple example it is seen that the bridge superstructure of this type is a special case in which the distribution of loads to the girders can be determined with the basic rules of statics.

In employing the sophisticated methods of analysis, it is suggested that the procedure be to develop influence lines for the various critical moments, shears and deflections required to insure a safe and serviceable structure rather than attempting to analyze the structure as a whole for the many combinations of live loading that could be imposed upon it. With this

![Fig. 2.7 Cross section of a bridge containing two longitudinal girders.](image)
approach, influence lines for moments or shears at various locations in the structure, similar to those which are shown in Figs. 4.42 and 4.43 for the moments in the connecting slab of the two-tubular girder superstructure, can be developed with minimal computer cost.

An approximate method for the elastic analysis of girder bridges has been proposed by Courbon (Ref. 31). This method is applicable only to girder bridges which have intermediate diaphragms of adequate design (see Section 2.3). When the span length of the girders is equal to or greater than twice the distance between the fascia girders and the depth of the diaphragm is of the same order as the girder depth, the diaphragm may be considered to be infinitely stiff in comparison to the girders. It is claimed that under these conditions this assumption will result in an error of less than four percent in the live loads computed for any of the principal beams.

The method proposed by Courbon can be understood by considering a typical girder bridge superstructure composed of several parallel beams, each of which has a constant moment of inertia, and one intermediate diaphragm of infinite stiffness at midspan. The torsional stiffness of the beams is considered to be negligible. If a load is placed eccentrically over the diaphragm as shown in Fig. 2.8, the system will deflect downward with the exterior girder on the same side as the eccentric load (1) having the largest deflection whereas the other exterior girder (5) will have the smallest deflection. The interior girders will have deflections which vary

![Diagram](image)

**Fig. 2.8** Deflection at midspan of a multi-girder bridge which has an adequate intermediate diaphragm and which is subject to a single eccentrically applied concentrated load.
linearly between those of the exterior girders. It should also be apparent that the elastic curve of each of these members would be the same even if intermediate diaphragms existed at other locations in the span in addition to the one at midspan. If these other diaphragms did exist, under the condition of loading described above, they would not be transmitting loads.

The reaction of the diaphragm \((R_i)\) on each girder can be evaluated by first determining the elastic center of the system. The elastic center is defined as the point measured along the diaphragm where the summation of the products of the moments of inertia of the girders and their distances from the elastic center are equal to zero. If the moment of inertia of beam \(i\) is represented by \(I_i\) and it is at a distance of \(x_i\) from the origin, the elastic center is computed from:

\[
\sum I_i x_i = 0 \tag{2.1}
\]

If a load \(P\) is placed upon the diaphragm at eccentricity \(e\), the reaction of the diaphragm on beam \(i\) will be:

\[
R_i = \frac{P I_i}{\sum I_i} + \left[ \frac{P I_i}{\sum I_i} \cdot \frac{e x_i I_i}{\sum I_i x_i^2} \right] \tag{2.2}
\]

or

\[
R_i = \frac{P I_i}{\sum I_i} \left[ 1 + \frac{e x_i I_i}{\sum I_i x_i^2} \right] \tag{2.3}
\]

In the case of a system composed of \(n\) beams equally spaced at intervals of \(S\) feet each of which has the same moment of inertia, the reaction on beam \(i\) is:

\[
R_i = \frac{P}{n} \left[ 1 + 6 \left( \frac{n + 1 - 2i}{n^2 - 1} \right) \frac{e}{S} \right] \tag{2.4}
\]

In Eq. 2.4, the beam number \(i\) is to be taken with beam number 1 being on the same, side of the axis of symmetry as is the eccentricity. This is illustrated in Fig. 2.8

A comparison of the results obtained with the approximate method just explained to that found with a finite element analysis for the superstructure of Fig. 2.4 is shown in Fig. 2.9. It is apparent that the approximate analysis yields results of sufficient accuracy for normal bridge design work. It is important to note that each method of analysis reveals that the exterior girder is subjected to a considerably greater portion of the load than the interior girders.
Fig. 2.9  Deflection of the bridge having the cross section of Fig. 2.4 as determined by the approximate and finite element methods for (a) a single concentrated load at the center, and (b) a single concentrated load with an eccentricity of seven feet.
The distribution of dead and live loads to interior and exterior beams is covered in Section 1.3.1 of the AASHTO Specification. The wheel load distribution factors for interior beams that have been included in the AASHTO Specification since 1949 are shown in Table 2.1. Note that the distribution factors vary from $S/5.0$ to $S/6.0$ for torsionally flexible beams of different types. The reason for this variation is not clear and cannot be rational. The wheel load distribution factors for interior beams supporting concrete decks, as contained in the 1973 edition of the AASHTO Specification, are given in Table 2.2.

Using the empirical wheel load distribution factors from the AASHTO Specification, the portion of a wheel load assumed to be imposed upon an interior concrete T-Beam and a prestressed concrete girder are as shown in Fig. 2.10.
Table 2.1—Live Load Sizing Moment Distribution Factors From AASHTO Specification For Concrete Floors Supported by Various Types of Beams

<table>
<thead>
<tr>
<th>Year</th>
<th>Not Specified</th>
<th>Steel I-Beams</th>
<th>Concrete Stringers</th>
<th>Concrete T-Beams</th>
<th>Prestressed Concrete Girders</th>
<th>Timber Stringers</th>
<th>Concrete Box Girders</th>
<th>Multi-Beam Precast Concrete Beams</th>
<th>spread Box Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>1949</td>
<td>$/5.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1953</td>
<td>$/5.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1957</td>
<td>$/5.5</td>
<td>$/6.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1961</td>
<td>$/5.5</td>
<td>$/6.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1965</td>
<td>$/5.5</td>
<td>$/6.0</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1969</td>
<td>$/5.5</td>
<td>$/6.0</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1973</td>
<td>$/5.5</td>
<td>$/6.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1974</td>
<td>$/5.5</td>
<td>$/6.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Spread box beam superstructures are covered in Section 1.6.24(A) of the AASHTO Specification by special empirical formulas.

**Multi-beam bridges, formed of precast box girders placed side-by-side, were treated as slabs prior to the 1974 Interim Specification. They are currently covered in Section 1.3.1(D) of the 1974 Interim Specification.*

### TABLE 2.2

<table>
<thead>
<tr>
<th>Kind of Floor</th>
<th>Bridge designed for one traffic lane</th>
<th>Bridge designed for two or more traffic lanes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>On Steel I-Beam Stringers and Prestressed Concrete Girders</td>
<td>$/7.0</td>
<td>$10.5</td>
</tr>
<tr>
<td></td>
<td>If $S$ Exceeds 10' use footnote</td>
<td>If $S$ exceeds 14' use footnote</td>
</tr>
<tr>
<td>On Concrete T-Beams</td>
<td>$/6.5</td>
<td>$/6.0</td>
</tr>
<tr>
<td></td>
<td>If $S$ Exceeds 6' use footnote</td>
<td>If $S$ exceeds 10' use footnote</td>
</tr>
<tr>
<td>On Timber Stringers</td>
<td>$/6.0</td>
<td>$/5.0</td>
</tr>
<tr>
<td></td>
<td>If $S$ Exceeds 6' use footnote</td>
<td>If $S$ exceeds 10' use footnote</td>
</tr>
<tr>
<td>Concrete Box girders</td>
<td>$/8.0</td>
<td>$/7.0</td>
</tr>
<tr>
<td></td>
<td>If $S$ Exceeds 12' use footnote</td>
<td>If $S$ exceeds 16' use footnote</td>
</tr>
</tbody>
</table>

$S =$ average stringer spacing in feet.

Footnote: In this case the load on each stringer shall be the reaction of the wheel loads, assuming the flooring between the stringers to act as a simple beam.
Fig. 2.10. Note that even though the stiffnesses of the two beams may be identical, the prestressed girder is assumed to carry more load than the T-beam.

The 1969 edition of the AASHTO Specification provided that outside or exterior beams of girder bridges be designed for the dead load of the deck which they support. This is normally taken to mean the portion of the deck load that would be imposed upon the exterior beam if the deck slab were simply supported at the first interior beam and the exterior beam. In addition, the 1969 AASHTO Specification provided that the loads from curbs, railings and wearing surfaces could be considered to be equally distributed to all roadway beams if these loads were imposed after the deck slab had been cured. The live load to be carried by the exterior beam was specified as the reaction of wheel load or loads determined by assuming the deck slab to act as a simple span between the exterior and first interior girder.

Using the provisions of the 1969 edition of the AASHTO Specification, exterior beams were frequently designed for less live load than were the interior beams. A rational elastic analysis of girder bridges, as has been described above, reveals that the opposite situation might actually exist. The exterior beams are subject to greater stresses due to live load than are the interior beams if the intermediate diaphragms are of adequate design.

A more recent edition of the AASHTO Specifications states that the exterior beams of girder bridges shall in no case have less carrying capacity than an interior beam. The term “carrying capacity” is not defined and could be subject to different interpretations.

The number of traffic lanes for which a bridge is to be designed in accordance with the provisions of the AASHTO Specifications is covered by Article 1.2.6 as revised by the 1974 Interim Bridge Specifications. The provisions of this Article are as follows:

The lane loading or standard truck shall be assumed to occupy a width of ten feet.

These loads shall be placed in 12-foot wide design traffic lanes, spaced across the entire bridge roadway width, in numbers and positions required to produce the maximum stress in the member under consideration. Roadway width shall be the distance between curbs. Fractional parts of design lanes shall not be used. Roadway widths from 20 to 24 feet shall have two design lanes each equal to one-half the roadway width.

The lane loadings or standard trucks having a 10-foot width shall be assumed to occupy any position within their individual design traffic lane, which will produce the maximum stress.

It is interesting to compare the results one obtains for the portion of a
wheel load carried by a bridge girder using the AASHTO empirical coefficients of Table 2.1, to those obtained from the approximate elastic analysis of Courbon. Using the notation shown in Fig. 2.11 and the provisions of AASHTO Article 1.2.6, it can be shown that the maximum eccentricity of the live load on a bridge that is 24 feet wide or wider is as follows:

\[ e_{\text{max}} = \frac{W}{2} - 6N + 1 \]  

(2.5)

in which \( N \) is the number of design traffic lanes on the bridge and \( W \) is the width between curbs or railings. Using the Eqs. 2.4 and 2.5 for various combinations of bridge roadway width \( W \), number of traffic lanes \( N \), and number of beams \( n \), one obtains the results shown in Table 2.3. An examination of these results will show that the AASHTO coefficients are more conservative for bridges designed for two design traffic lanes than for bridges of greater width. The results summarized in Table 2.3 also illustrate that the AASHTO empirical coefficients do not reflect the effects of number of design traffic lanes on a superstructure nor the effect of load intensity reduction for multi-lane structures.

Some engineers feel that the design traffic lanes should be ten feet wide rather than twelve feet wide as provided by the AASHTO Specifications.
## TABLE 2.3
12' Traffic Lanes

<table>
<thead>
<tr>
<th>$W$ Ft.</th>
<th>$N$ each</th>
<th>$n$ of beams</th>
<th>$S$ Ft.</th>
<th>Portion of a Wheel Load</th>
<th>Concrete</th>
<th>Prestressed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Interior Stringer</td>
<td>Concrete</td>
<td>Concrete</td>
</tr>
<tr>
<td></td>
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<td>Girder</td>
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<td>7.83</td>
<td>1.25</td>
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</tbody>
</table>

* These factors could be reduced to 90% and 75% of the values obtained from Eq. 2.4 for three and four design traffic lanes, respectively, in compliance with Section 1.2.9 of the AASHTO Specifications.

## TABLE 2.4
10' Traffic Lanes

<table>
<thead>
<tr>
<th>$W$ Ft.</th>
<th>$N$ each</th>
<th>$n$ of beams</th>
<th>$S$ Ft.</th>
<th>Portion of a Wheel Load</th>
<th>Concrete</th>
<th>Prestressed</th>
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<td></td>
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<td>7</td>
<td>7.83</td>
<td>1.58</td>
<td>1.31</td>
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</tbody>
</table>

* These factors could be reduced to 90% and 75% of the values obtained from Eq. 2.4 for three and four design traffic lanes, respectively, in compliance with Section 1.2.9 of the AASHTO Specifications.
Using the design traffic lane width of ten feet rather than twelve feet, the relationship for maximum live load eccentricity becomes:

\[
e_{\text{max}} = \frac{W}{2} - 5N
\]  

(2.6)

With this value of eccentricity, one obtains wheel load distributions to the girders as shown in Table 2.4.

From the preceding discussion one must conclude that the provisions of AASHTO regarding the distribution of live load to interior girders of concrete bridges are conservative for bridges having two design traffic lanes and are progressively less conservative as the bridge roadway width increases and the width of the design traffic lane decreases. They do not vary in the same manner as one obtains from an approximate elastic analysis. It can also be shown they do not agree with the results one obtains from a rigorous elastic analysis.

2.2 Girder Design

Once the distribution of the loads to the girders, as was discussed in Section 2.1, has been determined, the design of the girders can be undertaken. Two procedures are equally acceptable. In the first, the girders are proportioned for design loads and checked for their adequacy under the appropriate serviceability requirements. The second procedure consists of designing for service loads by elastic analysis after which the strength of the member is checked to confirm compliance with the minimum strength requirements of the design criteria being used. The most complete single source of data relative to the service and strength design requirements for prestressed concrete bridges will be found in reference 32.

Girder bridges require a relatively great structural depth to achieve a specific span. The depth-to-span ratios normally employed in girder bridges ranges from 1 in 16 to 1 in 20. The data presented in Appendix B relative to the dimensions of the more commonly used precast prestressed concrete bridge girders includes the span ranges on which the various sections are commonly used. The girders are normally used at spacings of from 6 to 9 feet with the closer spacings being used with the lower depth-to-span ratios. The fact that girder bridges do require relatively great depth has resulted in their use not being economical or perhaps even feasible in certain applications where vertical clearance requirements have been critical.

Proportioning prestressed concrete bridge beams for optimum structural
efficiency involves two basic considerations which may not be obvious. The first of these is the amount of the total dead load moment that is acting upon the section under consideration at the time of prestressing. The second is the ratio of the dead load moment to the total moment for which the section must be designed. These factors are best illustrated by considering several examples. Before so doing, two fundamental axioms of structural engineering should be mentioned. These are: (1) Flexural members of relatively short span are generally more critical with regard to shear stresses whereas members of relatively long span are generally more critical with regard to flexural stresses, and (2) For flexural members of relatively short span, live load generally constitutes a larger portion of the total load for which the member must be designed than does the dead load, while the opposite is true for flexural members of relatively long span.

With the foregoing in mind, it is interesting to consider how the optimum shape of a simply supported prestressed concrete bridge girder will vary with span length. The girder for a short span requires a relatively thick web.

![Fig. 2.12 Cross sections of the AASHTO-PCI Standard Bridge Beams.](image)
because of the greater importance of shear stresses. In addition, because the dead load moment is relatively unimportant in comparison to the total moment, a large bottom flange is required to “store” the prestressing force until needed to resist the transient live loads. The size of the top flange of a short-span bridge beam is rarely if ever critical because the bridge deck, whether cast monolithically with the beam or separately, is almost invariably adequate to sustain the moments at service and design levels.

For simple prestressed concrete bridge beams of long span, flexure is the primary design consideration. Because the bulk of the moment is due to dead loads, most of which may be acting at the time of prestressing, a large bottom flange is not required. A substantial top flange is needed for both service and strength considerations. Shear stresses are relatively unimportant in the design of long span prestressed concrete bridge beams.

Further consideration of the above will reveal that precast prestressed girders of optimum design will have different shapes for different spans. This is illustrated in Fig. 2.12. Long, simple-span cast-in-place members of prestressed concrete which have T-beam cross sections similar to that shown in Fig. 1.7, can be shown to be efficient.

In the design of superstructures incorporating precast members which are designed to act compositely with a cast-in-place deck, the elastic analysis for the composite section should be based upon section properties computed with due regard to the difference in the elastic modulii of the two concretes. In the design of large post tensioned concrete beams of T- or modified I-shaped cross sections, the space occupied by the ducts for the post-tensioning may have significant influence on the initial concrete stresses in the member. Hence, it is recommended that large post-tensioned girders be designed taking into account the effects of the net, transformed and composite section properties (Ref. 33).

Dimensions for typical precast prestressed concrete bridge beams which have found wide use in the United States are given in Appendix B.

### 2.3 Intermediate Diaphragms

An intermediate diaphragm has an important function to perform in insuring the structural integrity of a prestressed concrete girder bridge which has more than two longitudinal girders. There are no specific requirements in the AASHTO Specifications relative to the design of intermediate diaphragms for concrete bridges. This is unfortunate because many details which are commonly used for intermediate diaphragms with precast prestressed concrete girders are poor and not adequate at loads approaching the flexural capacity of the girders (Ref. 34, 35, 36).
As previously stated, an intermediate diaphragm should be provided at midspan of each span of a girder bridge. Little if any benefit is to be gained by placing additional intermediate diaphragms between the supports and midspan because they have little if any additional effect in improving the deflected shape of the girders. The intermediate diaphragms should be designed for the reactions computed with Eq. 2.4 unless more exact methods of analysis are used. Care must be given in detailing the connection of the diaphragm to the exterior girders. The continuity of the diaphragm flexural reinforcement between the exterior girders must be assured if the diaphragm is to function properly. Adequate shear reinforcing must be provided. One of the best methods of constructing intermediate diaphragms not frequently used, although it has been used in the past, is to post-tension the diaphragm as shown in Fig. 2.13. Unfortunately, post-tensioned diaphragms are more costly than those of reinforced concrete and it is this that undoubtedly accounts for their no longer being used.
A novel approach to providing stiff intermediate diaphragms in girder bridges that has been successfully used on bridges constructed for the lumber industry in the State of Washington is shown in Fig. 2.14 (Ref. 37). These diaphragms consist of structural steel sections installed by connecting them to small pieces of steel cast into the precast beams. Diaphragms of this type can be installed much more rapidly than the more conventional cast-in-place concrete diaphragms. They do, however, require maintenance as do bridges incorporating steel beams and trusses.

2.4 Decks for Girder Bridges

In the design of the deck for a girder bridge, a distinction must be made between bridges which have flexurally-stiff intermediate diaphragms and those which do not. As previously pointed out, the individual bridge girders of normal configuration (i.e. I-shaped or T-shaped beams) have virtually no torsional stiffness and hence do not afford rotational restraint to the decks which they support. Because the decks themselves are relatively flexible in comparison to the girders, differential girder live load deflection must be considered for bridges which do not have adequate intermediate diaphragms.

In view of this, it is recommended that the decks of girder bridges containing more than two girders be designed using the empirical relationships of the AASHTO Specification in cases where the girders are torsionally flexible and well designed flexurally-stiff intermediate diaphragms are not provided between them.

In the case of multi-girder bridges incorporating girders which are torsionally flexible but connected by well designed flexurally-stiff intermediate diaphragms, it is recommended the decks be designed as being continuous over the supporting girders without differential settlement between supports. The analysis should be made on an elastic basis using the Pucher (Ref. 17) or Homberg (Ref. 18) charts or other sophisticated methods of analysis. The girders should not be considered as providing rotational restraint to the deck.

As a means of illustrating the effects of differential girder deflection on the flexural stresses in the deck of a bridge incorporating precast I-girders, the results of a finite element analysis for the cross section of Fig. 2.1 are summarized in Fig. 2.15. The analysis was made for the bridge having a simple span of 120 feet and having one intermediate diaphragm at midspan. Two concentrated loads of 100 kips were applied to the bridge. The loads were applied over the middle girder 30 feet from each end of the bridge as
Fig. 2.15 Study of secondary stresses in the deck of a girder bridge; (a) loading arrangement, (b) deflection at point of application of a 100 kip load, and (c) elastic curve of the deck between points 3 and 4.
shown in Fig. 2.15(a). Under this condition of loading, the maximum flexural stress in the slab occurs at the point of application of the load and is of the order of ±320 psi. When it is realized that the 100 kip load is of the order of five times the design wheel load (including impact) used with HS 20-44 loading it becomes apparent the live load stresses due to differential girder deflection in structures of this type are of secondary magnitude and can safely be ignored in practice providing the design criteria being employed reasonably reflects the actual wheel loads to be imposed on the structure.

As pointed out above, bridge superstructures which contain only two torsionally flexible longitudinal girders constitute a special case. The decks for bridges of this type (see Fig. 2.7) should be designed as simply supported spans because the torsionally flexible girders provide virtually no joint restraint at all. The designer has the option of designing decks of this type with the influence charts of Pucher (Ref. 17) or Homberg (Ref. 18) or, of course, with the empirical coefficients of the AASHTO Specification. The use of the influence charts will yield more rational results.

When torsionally stiff girders are used, it is recommended that special methods of analysis similar to those discussed in Section 4.4 be employed.

2.5 Continuity

Prestressed concrete girder bridges utilizing cast-in-place T-beams are not a commonly used form of construction. Cast-in-place reinforced concrete T-beam bridges are rather frequently used and it is not uncommon for them to be made continuous over three or more supports. Concrete T-beams have found greatest use in bridges of medium span length and this probably accounts for the rather infrequent use of this shape with prestressing. The design of a T-beam bridge using prestressed reinforcement is straightforward and only the computation of the secondary moment resulting from the prestressing is a factor that is markedly different from the design procedures that must be followed with the two different materials.

Continuity has often been established in bridges incorporating precast prestressed concrete beams. This is normally accomplished through the provision of reinforcing steel in the cast-in-place deck which is placed over the precast girders. In this mode of construction the major portion of the dead load is carried by the precast beams acting as simple beams. The dead load of railings, sidewalks, wearing surfaces and other superimposed loads are carried by the continuous structure, as are the live loads. In the design of bridges of this type, positive moments may occur at the interior supports due to the effects of temperature as well as creep and shrinkage of the
concrete. Special consideration should be given to these effects. In addition, due to the special nature of the stresses in the prestressed concrete girders and cast-in-place diaphragms for bridges of this type, certain deviations from the usual allowable concrete stresses at service and design loads seem to be appropriate. These special requirements appear in reference 38.

2.6 Overhanging Beams

Occasionally the bridge designer is faced with providing a high, long, channel span over a waterway in order to provide horizontal and vertical clearance for navigation. A traditional method of accomplishing this in steel bridges has been with the use of a span suspended from two overhanging beams. This mode of framing, which is illustrated in Fig. 2.16, presents certain difficulties when done with I-shaped prestressed concrete girders. Most of these difficulties are in the form of construction sequences and methods that must be followed. The effort required to design a bridge of this type is aggravated by the need to establish the construction sequence that must be followed. These difficulties can generally be eliminated or minimized by the use of a cast-in-place box girder section for the overhanging beams, in lieu of the use of I-shaped girders.

![Fig. 2.16 Elevation of a bridge with two overhanging beam spans and a suspended span.](image)

Problems which have been encountered in the use of I-shaped prestressed concrete girders in bridges with overhanging beams include the following:

1. Longitudinal prestressing of the girders frequently has to be done in several stages, as the construction progresses, in order to control initial concrete stresses in the structure.
2. Temporary prestressing tendons frequently must be used to control stresses in the girders at certain stages in the construction.
3. The sequence of placing concrete in the end diaphragms, inter-
mediate diaphragms and deck slabs must take into account the effects of elastic and non-elastic shortening of the concrete.

4. Elastic stability of the girders (buckling) must be given consideration.

5. Erection procedures that can be used with the suspended spans may be restricted due to the stresses that could be imposed on the overhanging beams.

6. Excessive principal tensile stresses sometimes occur in the overhanging portions of the beams prior to erection of the suspended spans.

Each of the above should be given careful consideration before this mode of framing is adopted for use.

2.7 Construction Details

Multi-span girder bridges which incorporate precast prestressed concrete members are frequently designed with all spans being of equal length. This is done for the purpose of having all beams as identical in detail as possible. The cost of the precast members will generally be lower when the members

Fig. 2.17 Recommended detail for deck-to-girder connection.
are of the same length and very similar in detail. With equal span lengths, the negative moments at the first interior supports will be greater than those at the other interior supports. This is not generally a serious problem because the negative moments result only from superimposed dead loads and live loads and, particularly for the longer span structures, are of less importance than the primary dead load which is carried by the precast members acting as simple beams.

Multi-span girder bridges of the cast-in-place T-beam type would normally, if possible, be proportioned with the end spans being about 80% as long as the interior spans. Where possible, this arrangement results in a well balanced distribution of moments.

Precast prestressed concrete beams frequently exhibit significant and variable camber. If not anticipated in the detailing of the bridge, this factor can cause considerable difficulty in the construction of the bridge. The commonly used detail shown in Fig. 2.17 is recommended as a means of accommodating camber and variations in camber. The additional slab depth specified at the centerline of the bearings (1 10 inches for Fig. 2.17)
can sometimes be made less in structures which are on “humped” vertical curves. For structures which are on “sagged” vertical curves, greater thickening of the deck may be required over the girders at the centerline of the bearings. The detail in Fig. 2.18 is occasionally seen and is not recommended.

Diaphragms between the girders, including those at the ends as well as those which are at midspan, should be placed along the skew on skewed bridges, as shown in Fig. 2.19, rather than perpendicular to the girders in a stepped layout as shown in Fig. 2.20. With the diaphragm at midspan along the skew, points of equal flexural stiffness in adjacent girders are connected to each other and continuity of the diaphragm flexural reinforcement can be achieved. With stepped intermediate diaphragms points of unequal stiffness are connected, continuity of the diaphragm flexural rein-
Fig. 2.21 Poway Road Overcrossing of State Route 163, San Diego, California. (Courtesy of California Department of Transportation.)

Fig. 2.22 Harbor Drive Overcrossing, San Diego, California. (Owner: City of San Diego.)
Fig. 2.23 Los Penasquitos Creek Bridge, San Diego, California. (Courtesy of California Department of Transportation.)
Fig. 2.24 West Mission Bay Drive Bridge, San Diego, California. (Owner: City of San Diego.)

Fig. 2.25 Pacora River Bridge, Republic of Panama. (Owner: Republic of Panama.)
forcing is not possible and torsional stresses are induced in the girders near the diaphragms. It must be recognized that the diaphragm connection details for bridges incorporating precast girders can become complicated in detail and difficult to construct along the skew as shown in Fig. 2.19, especially for a structure having a large skew.

Girder bridges incorporating precast prestressed concrete girders have been constructed with bent caps located below the superstructure as well as within the depth of the superstructure. The former type of cap construction is shown in Figs. 2.21 and 2.22. Although attractive appearance can be achieved with this mode of framing, many engineers prefer the appearance that is achieved when the pier or bent cap does not protrude below the superstructure as shown in Figs. 2.23 and 2.24.

A cast-in-place post-tensioned T-beam bridge having variable depth is shown in Fig. 2.25. This structure, has a main span of 200 feet. A suspended channel span of 120 feet is supported on overhanging beams cantilevering 40 feet from each pier.

The details of construction for prestressed concrete girder bridges varies throughout the country. Details commonly used in each locality should be determined from local contractors and producers of precast concrete. If the details which are commonly used are considered adequate, their use may be less costly than new and unfamiliar details. Other sources of standard details include the Prestressed Concrete Institute*, the various State Highway Agencies and the Federal Highway Administration.**

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*Prestressed Concrete Institute, 20 N. Wacker Drive, Chicago, Illinois 60606.

**Federal Highway Administration, 400 Seventh Street, S.W., Washington, D.C. 20590
3.1 Introduction

Typical cross sections for concrete box-girder bridges of the types which are commonly used in the United States are shown in Figs. 3.1, 3.2, and 3.3. Bridges of these types, in both reinforced and prestressed concrete, have been widely used in the western part of the country, although their use has not been limited to this region.

The relative economy of the box-girder bridge has contributed greatly to its popularity as has its relatively slender and unencumbered appearance. Some proponents of the box-girder bridge have claimed its smooth soffit to be very desirable in urban areas for reasons of esthetics. The structural simplicity of the box-girder bridge, particularly in continuous structures of medium to long spans, has been well demonstrated. The efficiency of the cross section for positive and negative longitudinal bending moments as well as for torsional moments is apparent even to the casual observer. Especially important advantages for this mode of concrete bridge construction include the low depth-to-span ratio that can be economically achieved together with the ease with which variable, conditions of bridge width, superelevation and curvature, both vertical
Fig. 3.1  Cross section of a box girder bridge with vertical exterior webs.

Fig. 3.2  Cross section of a box girder bridge with inclined exterior webs.

Fig. 3.3  Cross section of a box girder bridge with curved exterior webs.
and horizontal, can be accommodated. Variable superstructure depth can also be accommodated with relatively little difficulty.

### 3.2 Flexural Analysis

The torsional stiffness of the typical “closed” box-girder bridge cross section as contrasted with that of the “open” I- or T-shaped girder section is very significant. Also important is the transverse flexural stiffness of the box-girder cross section. Each of these should properly be taken into account in the flexural analysis of bridges of this type. The torsional stiffness virtually eliminates significant transverse rotation of the section due to eccentrically applied loads of the magnitudes used in contemporary highway bridge design. This characteristic significantly differentiates a box-girder bridge from a girder bridge of similar physical proportions. The relatively large transverse flexural stiffness, in combination with the torsional stiffness, eliminates the need for intermediate diaphragms which are so important in girder bridges.

To illustrate the action of a box-girder bridge under concentric and eccentric loading, the bridge shown in Fig. 3.4 was analyzed using the finite element method as well as the familiar flexural theory. In the latter case, the torsional stiffness of the gross cross section was assumed to be completely effective in resisting rotation due to eccentrically applied loads and the transverse cross section was assumed to be infinitely stiff flexurally. The analysis was made for the condition of no intermediate diaphragms and a simple longitudinal span of 120 feet. The results of the deflection analysis for the conditions of a single concentrated load of 100 kips applied concentrically and eccentrically are illustrated in Figs. 3.5 and 3.6, respectively. An examination of these figures will reveal the approximate flexural theory yields results that are quite close to that

![Fig. 3.4 Cross section of a simple span box girder bridge used in the comparison of the results obtained with the approximate and finite element methods of analyses.](image-url)
obtained by the more sophisticated (and costly) finite element method of analysis. The approximate method obviously yields results that are sufficiently accurate for normal design work.

When one considers the fact that the load of 100 kips represents more than a single HS 20-44 truck applied at a single point, rather than being distributed in six concentrated loads of varying magnitudes spread over a width of six feet and a length of 28 feet, it becomes apparent the approximate analysis yields conservative results for the analysis of longitudinal flexural stresses. It is easily seen this is true whether the structure is loaded with one or more trucks.

Furthermore, in the finite element analysis of the bridge of Fig. 3.4, the maximum stresses due to transverse flexure in the deck of the bridge resulting from the load of 100 kips, applied either concentrically or eccentrically, were of the order of ± 260 psi. Because these loads were applied directly over the beam webs, primary bending of the upper deck
was avoided. Hence, one can conclude that the secondary moments induced by truck design wheel loads in the decks of bridges of this type are of a minor nature under usual conditions and therefore can be ignored. The design of the deck can safely be made under usual conditions using the elastic methods of analysis described in Chapter 1 disregarding the relatively minor stresses resulting from transverse frame action.

An approximate longitudinal elastic flexural analysis for a box-girder bridge, which includes the effect of torsional rotation but which ignores the effect of transverse bending, consists of using the principle of superposition and combining the effects of the vertical deflection due to the load to that of the rotation due to the torsional moment. The procedure consists of computing the vertical deflection of the superstructure with due regard to the restraint of the supports, assuming the transverse stiffness to be infinite and using the section properties of the gross section. Secondly, the effect of the rotation due to the torsional moment is de-
termined based upon the torsional constant of the gross section, again neglecting the effects of transverse flexural deflection. Combining the results of these two steps results in the approximate deflected shape of the structure due to the loading condition under study. The flexural stresses are proportional to the deflection of the structure. Hence, the approximate (and conservative) values of the longitudinal flexural stresses are easily determined.

The torsional constant of a section must be determined in order to evaluate the effect of torsional stiffness. This can be done using the membrane analogy (Ref. 39) or by solving the equations of equilibrium (Ref. 40).

For the section shown in Fig. 3.7(a), shear flows exist as shown in Fig. 3.8 due to the torsional moment $\mathbf{M}_t$. The equations for the web shear flows at sections B-B’ through E-E’ are:

$$\phi_6 = \phi_2 - \phi_1$$  \hspace{1cm} (3.1)

$$\phi_7 = 43 - 42$$  \hspace{1cm} (3.2)

$$4' = \phi_3 - 41$$  \hspace{1cm} (3.3)

$$\phi_9 = 4 - 45$$  \hspace{1cm} (3.4)

Shear flow is equal to the product of the shear stress and the wall thickness of the element under consideration.

The relationship for the torsional couple is:

$$2\phi_1 A_1 + 2\phi_2 A_2 + 2\phi_3 A_3 + 2\phi_4 A_4 + 2\phi_5 A_5 = M_1$$  \hspace{1cm} (3.5)

in which the notation is as defined in Fig. 3.7(a), the areas are as defined in Fig. 3.7(b) and $\mathbf{M}_t$ is the torsional moment applied to the member.

The relationships between the shear flows and the rate of twist $\theta$, using the same notation from Fig. 3.7(a) are:

$$\phi_1 \left[ \frac{B_1}{t_{b1}} + \frac{H_{01}}{t_{01}} + \frac{T_1}{t_{11}} \right] - \phi_0 \frac{H_{12}}{t_{12}} = 2GA_1\theta$$  \hspace{1cm} (3.6)

$$\phi_6 \frac{H_{12}}{t_{12}} + \phi_2 \left[ \frac{B_2}{t_{b2}} + \frac{T_2}{t_{22}} \right] - \phi_7 \frac{H_{23}}{t_{23}} = 2GA_2\theta$$  \hspace{1cm} (3.7)

$$\phi_7 \frac{H_{23}}{t_{23}} + \phi_3 \left[ \frac{B_3}{t_{b3}} + \frac{J-3}{t_{33}} \right] - \phi_8 \frac{H_{34}}{t_{34}} = 2GA_3\theta$$  \hspace{1cm} (3.8)
It should be recognized that the terms \( \frac{B_1}{t_{b1}} \), \( \frac{H_{01}}{t_{o1}} \), etc. in Eqs. 3.6 through 3.10 are equal to:

\[
\int_{s=0}^{t} ds
\]

for these members of constant thickness. For sections which have components of variable thickness, such as haunched slabs, one must solve Eq. 3.11 for each component of each cell.
The relationship between the rate of twist and the applied moment is:

$$\theta = \frac{M_T}{JG} \quad (3.12)$$

in which $\theta$ is the rate of twist resulting from moment $M_T$, $J$ is the torsional constant and $G$ is the shear modulus. This relationship can be written:

$$G\theta = \frac{M_T}{J} \quad (3.13)$$

For a moment equal to unity,

$$G\theta = \frac{1}{J} \quad (3.14)$$

Hence, in equations 3.6 through 3.10 the terms on the right side can be taken equal to $\frac{2A_1}{J}$, $\frac{2A_2}{J}$, $\ldots$, $\frac{2A_5}{J}$ and in equation 3.5 the term on the right side can be taken equal to one. Solving these equations will give the values of $\phi_1$ through $\phi_6$ and $J$.

The unit shear stresses due to torsion can be found by dividing the shear flow for any element by its thickness. This is:

$$\nu_i = \frac{\phi_i}{l} \quad (3.15)$$

in which $\nu_i$ is the torsional shear stress.

Considering the box girder section shown in Fig. 3.4, and using the principles given above, the torsional constant will be found to be of the order of 732 ft$^4$. The moment of inertia of the gross section is 5,663,000 ft$^4$. 
From these the vertical, rotational and total deflections of the webs one through five for a vertical load of 100 kips applied with an eccentricity of seven feet can be determined. The vertical deflection is computed as:

\[ \delta = \frac{PL^3}{48EI} = \frac{100 \times 120^3 \times 1728}{48 \times 3000 \times 5,663,000} = 0.366 \text{ in.} \]  

(3.16)

The angle of twist at midspan is:

\[ \theta = \frac{M_d L}{2GJ} = \frac{50 \times 7 \times 60'}{732 \times 1200 \times 144} = 0.0001660 \]  

(3.17)

The resulting deflections for beams one through five are summarized in Table 3.1 together with the deflections obtained from finite element analysis. In each of the analyses the values for elastic modulus and shear modulus were taken as 3000 and 1200 kips per square inch respectively.

<table>
<thead>
<tr>
<th>BEAM NO.</th>
<th>Vertical Deflection (in.)</th>
<th>Rotational Deflection (in.)</th>
<th>Total Deflection (in.)</th>
<th>Deflection from a Finite Element Analysis (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.366</td>
<td>-0.028</td>
<td>-0.394</td>
<td>-0.362</td>
</tr>
<tr>
<td>2</td>
<td>-0.366</td>
<td>-0.014</td>
<td>-0.380</td>
<td>-0.378</td>
</tr>
<tr>
<td>3</td>
<td>-0.366</td>
<td>0.000</td>
<td>-0.366</td>
<td>-0.333</td>
</tr>
<tr>
<td>4</td>
<td>-0.366</td>
<td>+0.014</td>
<td>-0.352</td>
<td>-0.308</td>
</tr>
<tr>
<td>5</td>
<td>-0.366</td>
<td>+0.028</td>
<td>-0.338</td>
<td>-0.297</td>
</tr>
</tbody>
</table>

An examination of Table 3.1 will reveal the approximate method of analysis predicts a slightly larger deflection for all five beams. The ratios between the predicted deflections for the average beam (beam 3) and the maximum and minimum deflections with each method of analysis is in remarkably close agreement.

The AASHTO Specifications provide an empirical value of \( S/7.0 \) for the portion of a wheel load that is to be distributed to an interior girder of a box-girder bridge. In addition, these specifications provide that an exterior girder of a box-girder bridge is to be designed for a live load wheel distribution equal to \( W_e/7.0 \) in which \( W_e \) is defined as the top slab width measured from the midpoint between girders to the outside edge of the slab. It should be obvious that these empirical values may or may'
not reasonably reflect the actual loads which are imposed upon a bridge structure. A rational analysis, as described above, gives the designer a good understanding of the structural behavior that can be expected from a given set of conditions. The empirical distribution factors may be safe (perhaps unduly safe) but do not necessarily reflect the true behavior of the structure.

A comprehensive study of a model of a reinforced concrete box-girder bridge has been reported (Ref. 25, 26). The tests confirm the adequacy of usual flexural theory for the design of bridges of this type. This conclusion can reasonably be extended to include bridges of prestressed concrete.

### 3.3 Longitudinal Flexural Design

From the considerations of torsional stiffness explained above, together with the provisions of the AASHTO Specification relative to number and positioning of design traffic lanes, which were explained in Chapter 1, it should be apparent that the maximum longitudinal flexural stresses occur in the box-girder bridge of usual proportions when the maximum number of design traffic lanes is applied to the structure. This may not be true in the case of a girder bridge. Because the eccentricity of the live load is normally small when the maximum number of design traffic lanes is applied to the structure, the effect of the torsional moment on the maximum longitudinal stresses should not be expected to be significant and hence can normally be neglected without incurring serious consequences. Maximum torsional stresses are found when the bridge is subject to less than the maximum number of design traffic lanes.

It should also be recognized that the AASHTO Specifications, in Article 1.3.1, stipulates an empirical load distribution factor of S/7.0 for concrete box-girder bridges (see Table 2.1). In the case of the superstructure shown in Fig. 3.4, the use of the requirements of the AASHTO Specifications would result in the structure being designed for 2.5 design lanes (five wheel loads) in spite of the fact Article 1.2.6 of the AASHTO Specification stipulates fractional design lanes shall not be used. Both finite element and approximate elastic methods of analysis reveal that the distribution factor of S/7.0 is conservative in the case of this example. Designing for five wheel loads is not rational for a structure which behaves as this one does.

Hence, it is recommended that the longitudinal analysis of box-girder bridge superstructures of usual proportions be analyzed using the conventional flexural theory based upon the gross section properties of the...
section. The cross section should not be divided into fictitious “interior” and “exterior” girders. The effect of live load eccentricity should be included, using the approximate method explained herein, when it is of significant magnitude.

As stated previously, bridge superstructures which are relatively wide in comparison to their span, should be designed with consideration being given to transverse flexibility and its effect on the distribution of live load in the longitudinal direction.

The question of the effect of shear lag on the accuracy of using the gross section properties in the analysis of a box-girder bridge is sometimes raised. Analytical studies have shown this phenomenon does not significantly affect box-girder bridges, nor even segmental bridge superstructures of usual proportions (Ref. 41 and 42). The use of very thin deck slabs in box-girder and tubular-girder bridges could conceivably result in cases where shear lag should be taken into account but with current construction methods and materials, it is doubtful if designs of such proportions would be feasible.

3.4 Decks for Box-Girder Bridges

The great majority of box-girder bridges which have been constructed in the United States have bridge decks which were designed with the empirical relationships of the AASHTO Specifications. The live load moments induced in the webs of the box girders have generally been ignored. This practice has apparently yielded satisfactory results although it is conservative with respect to the live load moments in the slabs and unconservative with respect to the live load moments in the webs.

Because of the great torsional stiffness of the box-girder section and the large flexural stiffness of the transverse “frame” formed by the top and bottom slabs in combination with the several webs of the typical box section, moments induced in the deck due to transverse torsional or flexural deformation due to live loads are generally very small and can be ignored. Hence, the live load moments induced in the structure under normal conditions can safely be analyzed using an elastic analysis assuming no differential deflection exists between the webs due to the live load. That is to say it is generally safe to ignore the secondary moments due to transverse deformation. Account should, however, be made for any variations in deck depth as well as for the elastic restraints induced by the webs.

For superstructures which are unusually wide in comparison to their span or for those which are relatively shallow with respect to their
width, transverse flexibility becomes important and approaches the behavior observed in slab bridges. Special consideration due to this effect should be included in the transverse flexural design of bridges of these types.

Torsional stresses, due to the eccentricity of dead and live loads, are rarely a problem in box-girder bridges due to the high torsional capacity of the typical box-girder bridge cross section. In spite of this, torsional stresses should be computed and taken into consideration when proportioning shear reinforcing. Torsional stresses, due to bridges being constructed with built-in horizontal curvature, require special evaluation (See Section 5.3).

![Diagram](image_url)

**Fig. 3.9** Cross section of an “open” section used to illustrate the computations of flexural shear stresses.
3.5 Shear Distribution

The distribution of flexural shear stresses in “open” sections, such as rectangular, T-shaped or I-shaped beams, is computed from the well known relationship:

\[ v_a = \frac{VQ_a}{I_b} = \frac{V}{I_b} \int_{y_a}^{y_b} y_b dy_a ds \]  

(3.18)

Fig. 3.10 Cross section of a closed section which is symmetrical about a vertical axis.
in which the terms are shown in Fig. 3.9 and are defined:

- $v_a$ is the unit shear stress at fiber $a$.
- $V$ is the shear force acting at the section under consideration.
- $Q_a$ is equal to the moment of the area of the cross section between $y_t$ and $y_a$ with respect to the centroidal axis of the section.
- $l$ is the moment of inertia of the cross section with respect to the centroidal axis.
- $b_a$ is the width of the section at $y_a$ above the centroidal axis.
- $b_y$ is the width of the section at $y$ above the centroidal axis.

The locations of points of zero shear stress are known in open sections. This is also true for the case of closed sections which are symmetrical about a vertical axis as shown in Fig. 3.10.

In the case of a closed section that has a single cell and which is not symmetrical about a vertical axis, the location of the points of zero shear are not known and a special method of analysis is required to determine the distribution of shear stresses. The special method must also be used for sections composed of several cells, as shown in Fig. 3.7. This too is because the points of zero shear are not known (Ref. 43). This method of analysis is similar to that used in the analysis of a multi-celled section for torsional stresses.

The procedure consists of assuming slits or cuts made in each of the cells of the multi-celled section as shown in Fig. 3.11. The existence of the cuts results in the section being open and hence the distribution of shear stresses can be determined by using Eq. 3.18. If the locations of the assumed cuts do not coincide with the locations of zero shear stress, a shear deflection will exist in the section at the location of each cut. A shear flow, similar to that resulting from torsion, must exist to nullify these deflections. A different shear flow is required at each cell to restore the deflected shape of the section. It can be shown that for a sec-
tion having n cells, n equations of consistent deformation can be written. The solution of these equations results in the determination of the shear flows in each cell. The addition of the shear flows to the shears for the open section result in the actual shear stresses for the loading condition. The equations of consistent deformation for a cell j of a multi-cell section as shown in Fig. 3.11 takes the form:

\[ \delta_{ij} \bar{q}_i + \delta_{ij} \bar{q}_j + \delta_{jk} \bar{q}_k + \delta_{jo} = 0 \] (3.19)

in which \( \bar{q}_i, \bar{q}_j \) and \( \bar{q}_k \) are the shear flows for cells i, j and k respectively, that are required to nullify the deflections at the assumed cuts in cells i, j and k.

\[ \delta_{ij} = -\frac{1}{G} \int_{s}^{s} \frac{ds}{t} \] (3.20)

\[ \delta_{jk} = -\frac{1}{G} \int_{s}^{s} \frac{ds}{t} \] (3.21)

\[ \delta_{ji} = \frac{1}{G} \int_{s}^{s} q_{jo} \frac{ds}{t} \] (3.22)

and

\[ \delta_{jo} = \frac{1}{G} \int_{s}^{s} q_{jo} \frac{ds}{t} \] (3.23)

in which \( q_{jo} \) is the shear flow at any point in cell j with the section being open (assumed slits in each cell).

For cells composed of elements of constant thickness the terms \( \delta_{ij} \) and \( \delta_{jk} \) are equal to the length of the element ji and element jk divided by their thicknesses respectively. The term \( \delta_{ji} \) is equal to the length of each element forming cell j, divided by its thickness. The term \( \delta_{jo} \) is easily solved because \( ds/t \) is a constant for each element of the cell and \( q_{jo} \)

---

Fig. 3.12 Shear flow for the cross section of Fig. 3.11 if cuts exist in the bottom of each cell.
normally varies linearly or parabolically along the length of each of the elements.

The procedure can best be described with an example. Consider the typical box-girder bridge cross section shown in Fig. 3.4. The moment of inertia of the cross section is $5,663,000 \text{ in}^4$. The section is subject to a vertical shear force of 600 kips. Assuming cuts in the bottom slab at the center of each span, the shear flow for the open section is as shown in Fig. 3.12. Solution of the equations of consistent deformation reveals the shear flows to be as shown in Fig. 3.13. The actual distribution of shear stress is equal to the sum of the values given in Figs. 3.12 and 3.13 and are as shown in Fig. 3.14. Note that the shear flows in the first interior webs are 1.17 times that in the exterior webs.
3.6 Construction Details

Depth-to-span ratios for post-tensioned box-girder bridges have frequently been of the order of 1 to 25. They have, in some instances, been constructed with depth-to-span ratios as low as 1 to 16 and as high as 1 to 27.8. Although one would normally expect continuous box-girder bridge spans to be somewhat more slender than simple bridge spans, a review of existing box-girder bridges reveals the depth-to-span ratios to be very similar for continuous and simple span structures (Ref. 44).

The web spacings for box girder bridges in the recent past have been of the order of seven to nine feet. Although the smaller web spacings have resulted in lower quantities of concrete and perhaps in some cases less transverse reinforcing steel, the additional labor required for forming costs associated with the smaller web spacings have resulted in the larger web spacings being used.

Minimum slab thicknesses for the slabs of box-girder bridges are specified in the AASHTO Specifications, Section 1.6.24. The minimum thickness specified for a top slab is the greater of the clear span between webs divided by 16 or 6 inches. For the bottom slab, the minimum thickness is specified as 5.5 inches or the clear span between webs divided by 16, whichever is greater. It is frequently necessary to thicken the bottom slab in areas of negative moment to 10 or 15 inches in order to accommodate the flexural compression stresses which occur in the bottom slab in these areas. For exceptionally long spans, even thicker bottom slabs may be required in the areas of negative moment.

Fillets of the order of four inches by four inches have commonly been provided between the slabs and the webs of box-girder bridges (see Fig. 3.4). Current practice in California gives the contractor the option of not providing fillets between the webs and bottom slab if he so desires. The slabs, either top or bottom, have been traditionally made of constant depth in box girder construction in the United States. This is probably the result of the facts that no advantage exists in haunching the slabs when they are designed in accordance with the empirical coefficients of the AASHTO Specification and forming costs are greater for slabs of variable depth.

Box-girder bridges post tensioned with all the tendons being placed in the webs, as has been the practice in California, are normally detailed with webs that are twelve inches thick. Experience has shown that this width is necessary except where the prestressing force is relatively small in which case webs of 8 to 10 in. can be used. It is recommended the bridge designer make a rough layout of the tendons for a typical interior and exterior girder at the time the bridge design is being made as a
TABLE 3.2

<table>
<thead>
<tr>
<th>Effective @ 0.60f's</th>
<th>Jacking @ 0.75f's</th>
<th>Dimension' X (inches)</th>
<th>Dimension'' H (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>960</td>
<td>1200</td>
<td>4.5</td>
<td>29</td>
</tr>
<tr>
<td>1440</td>
<td>1800</td>
<td>6.0</td>
<td>44</td>
</tr>
<tr>
<td>2160</td>
<td>2700</td>
<td>8.5</td>
<td>58</td>
</tr>
<tr>
<td>2640</td>
<td>3300</td>
<td>11.0</td>
<td>72</td>
</tr>
</tbody>
</table>

*= D + A in Figs. 3.15 & 3.16

** = Height in Fig. 3.17

TABLE 3.3

<table>
<thead>
<tr>
<th>Max. Number of ⅛&quot; Grade 270 Strands Per Tendon</th>
<th>Min. Duct Dia. (in)</th>
<th>Maximum Prestressing Force Per Tendon (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>@ 0.60f's</td>
</tr>
<tr>
<td>12</td>
<td>-2%</td>
<td>297</td>
</tr>
<tr>
<td>18</td>
<td>3</td>
<td>446</td>
</tr>
<tr>
<td>24</td>
<td>3½</td>
<td>595</td>
</tr>
<tr>
<td>31</td>
<td>4½</td>
<td>768</td>
</tr>
</tbody>
</table>

*Maximum jacking stress permitted by Standard Specifications of California Department of Transportation.

**Maximum jacking stress permitted by AASHTO Specifications.

means of insuring the adequacy of the width of the webs for tendon and concrete placing as well as confirming the required eccentricity of the prestressing force can be achieved at the critical points.

An important item to be checked (See Fig. 3.15) is that adequate space is provided for both the pier or bent cap reinforcing and the longitudinal tendons at points of negative moment. The necessity of confirming the adequacy of space also applies in areas of positive moment where the longitudinal tendons must not conflict with the space occupied by the bottom deck (See Fig. 3.16) and intermediate diaphragm reinforcing (if any). As an aid in accomplishing this, the data in Tables 3.2 and 3.3 are presented. These data are to be used in conjunction with the details shown in Fig. 3.15 and 3.16.
CHECK FOR CLEARANCE OF CAP REINFORCEMENT AT PIERS AND BENTS.

Fig. 3.15 Typical joint detail of top deck and web in a box girder bridge.

CHECK CLEARANCE IF INTERMEDIATE DIAPHRAGMS ARE SPECIFIED.

Fig. 3.16 Typical joint detail of bottom deck and web in a box girder bridge.
End diaphragms from 18 to 36 inches in thickness are normally provided at the abutment ends of box-girder spans. The end diaphragms serve the usual purpose of an end diaphragm but also serve as an area in which the post-tensioning tendons flare to their anchorages (Fig. 3.17). The thicker end diaphragms are normally used for bridges with greater skew.

Although the use of 8-inch thick intermediate diaphragms has been common in the past with this mode of construction, as well as currently being required by Section 1.6.24 (F) of the AASHTO Specification, as previously explained, it is expected this practice will diminish.

In applications where the conditions of the jobsite permit, multi-span cast-in-place box-girder bridges should be provided with end spans which are of the order of 80% of the length of the interior spans. This ratio of end span length to length of interior spans results in a favorable distribution of moments along the structure.

In simple span box-girder bridges, the tendons are generally placed in the structure in such a manner that the center of gravity of the prestressing steel, and hence the prestressing force, lies on a second-degree parabolic trajectory. The center of gravity of the tendons may or may not have a nominal eccentricity at the ends of the structure. This is illustrated in Fig. 3.18. Trajectories formed of parabolic curves com-
pounded with tangents, such as shown in Fig. 3.19 may have merit in some instances because the straight portions of the tendons are more easily tied in position during construction and the curvature of the tendons can be confined to the areas where it is most beneficial in reducing shear stresses on the concrete. The friction loss during stressing will be less for the trajectory of Fig. 3.18 than for that of Fig. 3.19.

In continuous structures, the tendons are generally placed in such a manner that the center of gravity of the prestressing steel is on a trajectory.
Fig. 3.21 Tendon trajectory geometry for an end span of a continuous box-girder bridge.

Fig. 3.22 Tendon trajectory geometry for an interior span of a continuous box-girder bridge.
tory formed of compounded second-degree parabolas. This is illustrated in Fig. 3.20 from which it will be seen that the trajectories in the end spans are formed of three parabolic curves while that in the interior span is composed of four parabolic curves. The geometry of the trajectories must satisfy the requirements shown in Figs. 3.21 and 3.22.*

Although it is somewhat less efficient from a *flexural standpoint, the tendon layout shown in Fig. 3.23 is more efficient in reducing shear stresses on the concrete section than the trajectory of Fig. 3.20.

Different jurisdictions have different clearance requirements for *post-tensioning* ducts. The requirements of the California Department of Transportation which are applicable to precast post-tensioned girders are illustrated in Fig. 3.24. The clearance requirements for *post-tensioned* ducts in cast-in-place box-girder construction as specified by the California Department of Transportation are shown in Fig. 3.25.

---

*The designer must evaluate the friction losses and effects of anchorage deformation which occur during stressing continuous tendons of this type in order to ascertain the minimum amount of prestressing that will satisfy the design criteria (Ref. 45).*
The pleasant appearance of simple and continuous span box-girder bridges is illustrated in Figs. 3.26 and 3.27. Note the fact that the bent cap is hidden within the depth of the bridge superstructure in Fig. 3.26. The ease with which horizontal and vertical curvature as well as superelevation is accommodated with this mode of construction is illustrated in Fig. 3.27.

Fig. 3.24 Clearance requirements for post-tensioned ducts in precast girders according to the requirements of the California Department of Transportation.
Fig. 3.25 Clearance requirements for post-tensioned ducts in cast-in-place box-girder bridges according to the standards of the California Department of Transportation.
Fig. 3.26 Typical box-girder bridge with bent cap within the superstructure. (Courtesy of California Department of Transportation.)

Fig. 3.27 Mission Valley Viaduct in San Diego, California. (Courtesy of California Department of Transportation.)
4 Segmental Box-Girder Bridges

4.1 Introduction

The term “segmental box-girder bridge” or “segmental bridge” has been used in this book to describe special forms of box-girder bridges. The special forms are characterized by three basic cross sections which are illustrated in Figs. 4.1 through 4.3. The fundamental difference between the cross sections of the segmental bridges and box-girder bridges is the

Fig. 4.1 Typical cross section of a segmental bridge.
relatively few webs incorporated in the segmental bridge cross sections. Another difference is that in segmental bridges, as in the case shown in Fig. 4.3, the bottom slab may not be completely continuous between the most exterior webs. Additionally, segmental-girder bridges generally employ deck slabs of variable depth and are constructed without intermediate diaphragms except at locations of hinges. Hinges are sometimes provided within a span in order to accommodate length changes resulting from concrete shrinkage, creep and temperature variations. The typical cross sections used in segmental girder bridges have evolved in Europe where they have been optimized with a view toward reducing their dead load. The special importance of superstructure dead load with long span bridges is well known.

Torsional stiffness, as in the case of box-girder bridges, is important in the manner in which segmental bridges behave structurally. With the exception of superstructures formed of two or more tubular girders connected with a common slab, the structural behavior of a segmental-girder bridge is basically the same as that of a box-girder bridge. The special considerations for the multi-tubular girder bridge with connecting slabs between the individual girders is limited to the special analysis required for the transverse distribution of moments discussed in Section 4.4.

Because of the unique cross sections and erection techniques used
with this mode of bridge construction, special design considerations must be employed in their proportioning and analysis.

### 4.2 Longitudinal Flexural Analysis

The fundamental principles of structural analysis used in the design of bridges of other types are used for the determination of the maxima and minima moments and shears which can occur along the length of segmental box-girder bridges. However, certain types of flexural analyses not required in the design of bridges of other types are required in the design of segmental bridges that are erected in cantilever or with other special methods of erection. These include the dead load analysis for the statically determinate cantilevers, the determination of the secondary moments which occur at the various stages of construction as a result of the continuity tendons being installed and prestressed, and the analysis of the redistribution of moments (see Section 4.3) which occurs in the structure as a result of creep, shrinkage and relaxation. The effects of superimposed dead and live loads are determined as for any other structure.

The cantilever erection technique is illustrated in Fig. 4.4. It will be seen that the procedure consists of first setting the pier segment on top of the completed pier after which span segments are placed first on one side and then on the other. The segments may be cast-in-place or may be precast. In either case, the segments are prestressed to the previously constructed work, after they are placed.

It should be noted that some means of accommodating the unbalanced moment must be provided with a superstructure erection technique such as this one. The unbalanced moment can be accommodated by securely attaching the pier segment to the pier, either permanently or temporarily during construction, if the pier itself is sufficiently strong to accommodate the unbalanced moment. A discussion of special considerations for moment resisting piers to be used in bridges erected in cantilever will be found in Section 5.6. In some cases temporary bracing struts or temporary ties are used to accommodate the unbalanced moment. These methods are all illustrated in Fig. 4.5 and 4.6. Temporary struts and ties can be relatively costly. The removal of a temporary tie or strut results in the application of a load to the structure. The load in ties and struts can be quite large and must be considered in the stress analysis of the structure when they are used. It should be recognized that the foundations must be able to withstand the unbalanced moment if the temporary struts or ties are connected to them. Specially designed erection gantries
Fig. 4.4 Cantilever erection scheme.
Fig. 4.5 Superstructure-substructure temporarily fixed for unbalanced moment.

Fig. 4.6 Temporary struts for unbalanced moments.
which are capable of erecting precast segments as well as bracing the superstructure for unbalanced moments during erection have been used successfully. The unbalanced moment which occurs in cast-in-place segmental construction, due to the weight of the travelling forms, is shown in Fig. 4.7.

The computation of the cantilever moments which occur during the erection process is simple and straightforward. They must be computed for the location of every joint between the segments because the stresses in the concrete due to dead load and the initial prestressing forces must be checked at each joint at each step in the erection procedure. A diagram illustrating the cantilever moments which occur during erection is given in Fig. 4.8.

The overall erection of a typical bridge superstructure consisting of five spans is shown in Fig. 4.9. This particular superstructure is supported by hinged bearings at the top of each pier and abutment. The superstructure erection commences with the erection of the cantilevers which extend from each side of the first pier. The second step consists of constructing the end portion of the first span. When the end portion is completed, it is connected to the cantilevered superstructure constructed in the first step by the placing of the closure joint. The first span is completed by the installation and stressing of the “continuity tendons” which extend between the two portions of the span. Because there are only two simple supports, the stressing of the first “continuity tendons” does not create secondary moments in the structure. The erec-
tion progresses with the construction of the cantilevers which extend outwards on each side of the second pier (Fig. 4.9c). When the cantilevers near midspan of the second span are joined together and the closure joint has been placed and cured, the continuity tendons joining the two cantilevered portions of span two are inserted and stressed. Because the structure is continuous over three supports at the time the second group of continuity tendons is stressed, secondary moments, which have a distribution as shown in Fig. 4.10a, are created. Repeating this procedure, the construction continues until span five is completed and rendered continuous to the previously erected structure. The secondary moments resulting from the stressing of the various groups of continuity tendons together with their sum, are shown in Fig. 4.10. It should be noted that an extension to the cantilever of the first portion of the third span must be placed after continuity is established in the second span. This is the result of the unequal span lengths.

Other erection techniques which lend themselves to precast segmental bridges also result in secondary moments, as well as primary moments, which must be taken into account in the superstructure design. Consider the two span freeway overcrossing structure shown in Fig. 4.11. This structure is erected using temporary towers together with temporary prestressing. The erection procedure consists of erecting the three assemblies of precast units on top of the towers using the temporary prestressing tendons as shown in Fig. 4.12(a). After the three assemblies
Fig. 4.9 Construction sequence for a five-span bridge erected in cantilever.
Fig. 4.10 Secondary moments which occur during erection of the bridge shown in Fig. 4.9.
Fig. 4.11 Two-span freeway overcrossing structure.

Fig. 4.12 Erection of a freeway overcrossing structure using temporary towers and temporary prestressing.

Fig. 4.13 Two span highway overcrossing structure.
are erected, the two outermost assemblies are slid towards the middle assembly until they are in contact as shown in Fig. 4.12(b). It should be recognized that all of the precast units fit together perfectly because they are precast one against the other in the same order as they are to be assembled in the structure. This procedure also results in the outer assemblies of precast units fitting the inner assembly of precast units perfectly and eliminates the need for cast-in-place joints between the assemblies. In addition, hydraulic jacks are provided between the assemblies of segments and the temporary erection towers. (See Fig. 4.12a). The hydraulic jacks allow realignment of the assemblies during erection in the event it is necessary to correct for settlement of the temporary towers or make other minor adjustments. After the assemblies have been slid together, the first stage permanent prestressing tendons, having the shape shown in Fig. 4.13, are installed in preformed ducts in the segments and prestressed. Because the superstructure is continuous over six supports at the time the first stage tendons are stressed, a secondary moment as shown in Fig. 4.14 is created by the stressing. (Note: The hydraulic jacks at the top of each set of twin towers are interconnected during this operation with the result they act as a "hydraulic

Fig. 4.14  Secondary moment due to first stage prestress for the bridge of Fig. 4.11.

Fig. 4.15  Moment due to removal of the temporary erection towers shown in Fig. 4.12.
The next step in the erection consists of removing the erection towers. This results in a primary moment, having the shape shown in Fig. 4.15, being imposed upon the structure. Removal of the temporary prestressing tendons constitutes the next step in the procedure. This results in the secondary moment distributed as shown in Fig. 4.16. The secondary moment due to the second stage permanent prestressing tendons is shown in Fig. 4.17. Installation and prestressing of the second stage tendons completes the erection procedure.

From the above description of the various different moments which occur in bridges that are erected segmentally it should be apparent that the design of bridges of these types requires a considerable amount of engineering effort. Because the moments are dependent upon the construction methods that are to be used, the designer must specify the construction sequence that is to be followed. Because the stresses at each joint between the segments must be considered for each step in construction more design effort is required for segmental bridges than for more conventional bridge types. It should also be recognized from the above discussion that the basic elastic analysis involves methods familiar to all structural engineers.

The effect of live load on a segmental bridge must be found using fundamental engineering principles together with the basic provisions of Section 1.2 of the AASHTO Specifications. The provisions of Section 1.3 are clearly not intended to apply to bridge superstructures composed
of a single tubular girder. There are few complexities involved in the theoretical analysis of the distribution of wheel loads to members of this type. Consider the cross section of the single tubular girder shown in Fig. 4.18 which is subjected to an eccentrically applied vertical load. The eccentrically applied load can be analyzed as a concentric load and a torsional moment. Due to the great stiffness of the section, the rotation is small with the result the deflection of point A is only slightly greater than the average vertical deflection of the section.

It can be shown that for eccentrically applied live loads that are large in comparison to the total load, longitudinal warping stresses can be significant (Ref. 27). For most applications where segmental bridges will be considered for use in North America, the live loads used in design will not be large in comparison to the total load. Hence, this effect is generally ignored and, as far as the consideration of longitudinal flexural stresses are concerned, the loads are assumed to be applied concentrically. The torsional stresses should be considered using the greatest torsional moment that can be obtained with the live loads positioned as provided in the AASHTO Specifications.

As in the case of box-girder bridges, it will be found that segmental-girder bridges should be designed for longitudinal moments resulting
from the greatest number of design traffic lanes that can be placed upon the structure. The design for torsional stresses should be based upon the number of design traffic lanes which results in the greatest torsional moment.

The usual “beam” or “flexural” theory is adequate for the analysis of longitudinal flexural and shear stresses of most bridges which are to be built segmentally. The effect of the transverse flexural and torsional deformations on the longitudinal flexural and shear stresses is small and hence can safely be ignored. Bridge superstructures which are wide in comparison to their span or depth, require special consideration because of the transverse deformations which result when concentrated loads act upon them. This is not unique to bridge superstructures which are to be constructed segmentally but is also true for slab, girder or box-girder bridge superstructures which are constructed using conventional methods.

Shear lag is not normally a problem in segmental bridge superstructures of normal proportions (Ref. 41). Segmental bridge superstructures of usual proportions, which is intended to mean relatively narrow in width in comparison to the span length, are generally analyzed for flexure using the section properties of the gross section. Experience and analytical studies have shown this to be a reasonable assumption (Ref. 42).

Although the torsional stiffness of the tubular girders eliminates the need for intermediate diaphragms, diaphragms are normally needed at the points of support. Support diaphragms are used to transfer vertical loads as well as longitudinal and torsional moments to the substructure. This important need should not be overlooked. In a number of small structures constructed in France, diaphragms have been provided at the abutments alone with the bearing details at the pier-superstructure connections being such as to preclude the transfer of longitudinal bending moments to the substructure.

### 4.3 Creep Redistribution of Moments

The redistribution of moment due to concrete creep constitutes an important design consideration for segmental bridge superstructures that are erected in cantilever. This phenomenon is unique to structures which are erected in one structural configuration which is subsequently altered into another as is done with the cantilever erection technique. There is no redistribution of the moments in structures which are constructed at one time as is the case with conventionally cast-in-place con-
crete structures or with segmental bridges that are assembled and stressed in one operation. This phenomenon is generally characterized by a relatively small reduction in the effective negative moment at the piers and a relatively great increase in the effective positive moment at midspan. Effective moment is defined as that which is due to the combined effects of dead load and prestressing.

The cause of the redistribution of moment can be illustrated by considering two cantilever beams as shown in Fig. 4.19. These beams are rendered continuous when the cast-in-place joint between them is con-

![Two Cantilever Beams](image)

**Fig. 4.19** Two cantilever beams used to illustrate the cause of moment redistribution due to creep

structured and the continuity tendons between the two cantilevers are installed. It should be apparent that if the two beams were not rendered continuous, the effect of creep would be to cause vertical deflection and a rotation of the ends of the beams with the passing of time. Because this rotation is prevented by the provision of continuity, a positive moment is created near midspan. The creep-caused positive moment is of significant magnitude and is accompanied by a reduction of the negative moment at the supports which is normally of negligible magnitude.

The existence of creep-induced moment redistribution was not recognized in the early applications of segmental bridges. The result was that
some months after being put into service, cracks developed in the tensile flanges of some of these bridges in the areas of high positive moment. Although the bridges were not considered to be dangerous as far as their ultimate strength was concerned, it was felt their durability might be adversely affected by the existence of the cracks. For this reason, additional positive moment prestressing was introduced to close the cracks.

An early discussion of this phenomenon appeared in a paper by Jean Muller (Ref. 46) in 1967. In this discussion Muller has shown an example of a structure where it is estimated (by approximate calculations) that the effect of creep would reduce the effective negative dead load moment by 3% and increase the effective positive dead load moment by 20%. A more sophisticated method of evaluating the effect of this phenomenon is to use a numerical integration process in which concrete strength, modulus of elasticity, shrinkage and creep as well as relaxation of the prestressing steel are all treated as time functions using the methods first proposed by Subcommittee 5 of ACI Committee 435 (Ref. 47). The results of such an analysis, together with the effect of superimposed dead loads, can be used to determine the time-dependent rotations at various sections along a span. The rotations can then be used to determine incremental changes in the distribution of moments throughout the bridge structure as well as the changes in stress which result therefrom. If one makes these computations using numerical integration with a small time interval, and carries out the computations for a total time period (for the youngest concrete) of the order of 1,000 days, substantially limiting results will be obtained. An electronic computer is required for these computations due to the large number of variables involved in the analysis.

An approximate method which has been used in France to provide for the positive moment which results from the redistribution is to simply provide a residual compression of at least 225 psi under the effects of dead plus live plus impact loads in areas of positive moment. Because of the smaller design live loads used in this country, this latter method may not be conservative in the United States.

As an alternative to attempting to compute or estimate the magnitude of the redistribution of moments resulting from the creep of structures erected in cantilever, provision for future additional positive moment prestressing can be made during construction. This is accomplished by providing extra ducts for prestressing tendons in the positive moment areas during the original construction. In this manner, additional positive moment prestressing tendons can be added to the structure anytime after its completion in the event distress is subsequently detected in the areas of positive moment.
A method of computing the creep-induced positive moment which will develop at midspan of two cantilever beams which have been rendered continuous has been proposed by M. Thenoz (Ref. 48). The relationship resulting from this method gives the following expression for the positive moment due to the redistribution due to creep:

\[
M = - \frac{\omega_1 \Phi \left[ 1 - r (t_1 - t_0) \right]}{\omega_2 \left[ 1 + \Phi \right]} \tag{4.1}
\]

in which

- \(\omega_1\) is the rotation in radians at the end of the cantilever due to the combined effects of dead load and prestressing which the cantilever would undergo if not rendered continuous, with the elastic modulus being taken as unity. (See Fig. 4.20 for sign convention).
- \(\omega_2\) is the rotation at the end of the cantilever due to a unit moment applied at the end of the cantilever, with the elastic modulus being taken as unity. Units are radians (per unit moment).
- \(t_0\) is the time in days at which the dead load and prestressing are applied.
- \(t_1\) is the time in days at which continuity is established.
- \(\Phi\) is a coefficient dependent upon the concrete quality, creep maturity (age at loading) and theoretical thickness of the concrete as well as the creep-time coefficient.

\[
\Phi = K_n r(t_x - t_o) \tag{4.2}
\]

\(K_n\) and \(r(t)\) are from the Appendix I of the French prestressed concrete code which appears as Appendix A in this book.

The numerator of Eq. 4.1 is proportional to the amount of rotation the end of the cantilever would undergo under the action of dead load and

![Fig. 4.20 Sign convention for creep redistribution calculation method proposed by Thenoz.](image)
prestressing subsequent to the time continuity was established [from time $t_1$ to $t_1$]. The denominator is proportional to the total amount of rotation the end of the cantilever would undergo (elastic and time dependent) under the effect of a unit moment applied at time $t_1$. Hence, the numerical integration procedure for the computation of time dependent deflections and rotations (Ref. 47) could easily be adapted to determine the moment resulting from the actions of concrete creep and shrinkage together with relaxation of the steel using the principal proposed by Thenoz.

When applying Thenoz’s method, one must recognize that there is no provision for distribution of moment to the supports and to adjacent spans. The method is based upon the assumption that the supports of the cantilevers are fixed. The effect of distribution to adjacent spans should be considered.

4.4 Transverse Flexure

The design of superstructures incorporating a single box or tubular girder for stresses which exist in the transverse direction, consists of determining the adequacy of the concrete section as well as the amount of prestressing or reinforcing steel required to resist the flexural and shear stresses which are induced in the section by dead and live loads as well as the prestressing and temperature effects. A second consideration is the determination of the torsional stresses which could exist in the section due to live loads which might be applied eccentrically with respect to the longitudinal axis of the structure or due to horizontal curvature which is to be built into the structure. Superstructures which are formed of two or more tubulargirders connected together by a slab spanning between them require special study because the vertical deflection and rotation of the tubular girders due to the applied loads induce stresses in the superstructure for which provision must be made.

The analysis of the single tubular segment for transverse bending moments consists of solving a tube-frame composed of four or more components each of which frequently has variable depth. The determination of the moments for dead load is relatively straightforward. The analysis for live load and impact is somewhat more complicated due to the complication of determining the effect of a series of wheel loads. The wheel loads are applied to an elastic plate of variable depth and great width. The plate being one of several components of the tube-frame. The solution of this problem is greatly facilitated by the use of charts of influence surfaces such as those compiled by Homberg (Ref. 18).

The use of the charts in the determination of live load moments for the
Fig. 4.21  Moment diagram for a single-cell tubular girder (a) dead load of section plus live load on cantilevers, (b) dead load of section plus live load on center span only, and (c) for transverse post-tensioning of the upper slab, only.
cantilever overhangs is straightforward. One simply computes the volume the footprint each of the wheel loads makes on the influence surface. The sum of the products of each of these volumes together with the magnitude of the corresponding wheel load is equal to the moment per unit length for the point under study. One must use a trial and error procedure to determine the maximum value. (See Section 1.4).

In the case of interior spans, the influence surfaces are used to determine the fixed end moments for various positions of the load. The fixed end moments are then used in a frame analysis to determine the effect of live and impact loads on the frame. A convenient method is to construct influence lines for moment at the critical points in the span, based upon the results of a series of frame analyses using fixed-end moments from the appropriate influence surface chart.

This method of designing the structure for the effect of concentrated wheel loads is relatively simple and straightforward. It is based upon fundamental elastic design principles rather than upon empirical coefficients. In addition, it permits one to take the effect of haunches into account.

The results of such an analysis are shown in Fig. 4.21. The condition of moment within the tube-frame for maximum load on the cantilevered slab overhangs is shown in Fig. 4.21(a). The moment diagram for maximum positive moment for the interior deck span is shown in Fig. 4.21(b). It should be noted that the moments in the webs where they intersect the deck are of opposite sign under these different conditions of loading.

It is sometimes preferable to post-tension the deck transversely rather than employ conventional reinforcing steel to resist the transverse bending moments. When the deck is post-tensioned it undergoes an elastic shortening and moments are created in the tube-frame. Due to the effects of concrete creep, shrinkage and the relaxation of the prestressing steel, these moments diminish with the passing of time. Depending upon the creep characteristics of the concrete, the moments due to this effect may eventually be as little as one-third of their initial value. The moment diagram shown in Fig. 4.21(c) is typical of those due to elastic shortening of the deck.

If the deck and bottom slab of a box section have different temperatures, moments are induced in the tube-frame. The moments so induced are similar to those shown in Fig. 4.21(c) but may be of opposite sign. Because temperature differentials of this type are transient rather than permanent in nature, creep of the concrete does not reduce the moments caused by them. This phenomenon should be considered in the design of the tube-frame.

It should also be noted that reinforcing steel, that is in addition to the reinforcing steel required for shear stresses, must be provided in the webs
Fig. 4.22  Moment diagram for concentrated live loads on the center slab of a single-cell tubular girder.
in order to control the flexural tensile stresses in the webs. The number and positions of live loads which result in the greatest requirement for shear reinforcing at a section along the length of a beam, may be different from that which produces the maximum web flexural stresses. Because of this, the amount of web reinforcing required for the critical combinations of loading at any section might be less than the sum of the amounts required for shear and flexure if acting alone. This is illustrated in Fig. 4.22. The loading condition for maximum shear, for the cross section under consideration, is shown in Fig. 4.22(a) while those for two different conditions of web flexure are shown in Figs. 4.22(b) and 4.22(c).

Special attention must be given to the manner in which the web reinforcing and deck reinforcing are anchored in the web-deck joints. The details used must insure that the reinforcing that terminates in the joint is adequately anchored for the use intended and that continuity of the reinforcing steel is provided through the joint for the transfer of positive and negative moments which act upon the joint. This is illustrated in Fig. 4.23. Reinforcing steel bars must not be bent around re-entrant corners as shown in Fig. 4.24 unless properly tied, so that lateral force generated by a tensile force in the bar cannot spall the concrete.

The dimensions selected for the web-deck joint are not always completely dictated by considerations of transverse flexure. In the case where the longitudinal prestressing tendons may consist of a number of seven-wire strands, the size of the joint may have to be increased to accommodate the large number of tendons that must be concentrated in or near the joint. This is illustrated in Fig. 4.25. This problem is not as severe in structures utilizing a large number of short straight bar tendons because the short bar tendons are terminated in the joints between the segments, including the deck area, rather than being bent downward from the deck into the web to their anchorages as is normally the case with multi-strand tendons. A typical cross section for a tubular girder with bar tendons is shown in Fig. 4.26.

As mentioned above, the bridge designer has the option of reinforcing the bridge deck with ordinary reinforcing steel or with prestressed reinforcement. It is impossible to give specific limitations as to which of the two should be used because there are so many variables which should be considered. If designed without tensile stresses in the concrete under service loads, prestressed decks would be expected to be virtually crack-free and more durable than those utilizing non-prestressed reinforcement. On the other hand, segmental girder bridges generally have many transverse construction joints in the structure and hence cannot be assumed to be crack-free. Some engineers believe the existence of the many construction joints necessitates the use of a waterproof membrane on the deck of all
bridges constructed segmentally. One might conclude that the presence of a waterproof membrane renders the existence of fine flexural cracks in a non-prestressed deck unimportant. The relative economics of prestressed and non-prestressed reinforcement must be considered for each structure taking into account the parameters of the specific project and the economic conditions prevailing at the time the project is undertaken.

As previously stated, superstructures composed of two or more tubular girders connected together by a common slab as shown in Fig. 4.27, are not normally provided with intermediate diaphragms. Flexurally and torsionally stiff diaphragms are, however, almost invariably provided at the points of support for such superstructures.

Under the action of concentrated live loads, eccentrically applied between the supports, a structure of this type deforms as shown in Fig. 4.28. The deformation of the structure under this loading is characterized by vertical deflections and rotations of the individual tubular girders. These deformations induce secondary moments in the connecting slab which may or ‘may not be additive to primary moments which result from live loads!
Fig. 4.25 Web-deck joint showing longitudinal and transverse reinforcement.

Fig. 4.26 Details of prestressing in the deck of a bridge prestressed with bar tendons.

Fig. 4.27 Bridge superstructure composed of two tubular girders connected together with a common slab and without intermediate diaphragms.
applied directly to the connecting slab. The secondary moments in the connecting slab can be of significant magnitude near midspan of the structure and must be considered in the design of bridge decks of this type. Near the supports of the superstructure the secondary moments normally disappear as a result of the vertical and rotational restraints afforded by the supports.

An approximate elastic method of analysis for the transverse distribution of the loads in bridge superstructures consisting of two or more connected tubular girders has been reported by Muller (Ref. 49). This method of analysis utilizes the principle of superposition as well as the method of support constants which is explained in Appendix C. The basic principle consists of determining the primary bending moments in the deck independently of the secondary bending moments, determining the secondary moments independently of the primary moments and then combining the effects of the two.

The primary bending moments are determined by analyzing the struc-

![Fig. 4.28 Deformation of a bridge superstructure composed of two tubular girders connected by a common slab under the action of an eccentrically applied load.](image)

![Fig. 4.29 Idealized structure for the determination of the primary moments due to the live load.](image)
ture for live loads under the assumption that supports exist under the webs of the structure as shown in Fig. 4.29. The fictitious supports would prevent vertical deflection of the girders as well as their rotation. This type of analysis is well known to all structural engineers and does not require explanation. It will yield results that can be summarized as influence lines for primary transverse bending in the deck at the critical points.

The secondary bending moments for a line load applied to the structure are determined by replacing the single line load with two combinations of line loads and line moments which are equivalent to the single line load being considered. The replacement loads and moments are
applied directly above the webs in order to avoid primary bending. The replacement loads and moments can be made to be in two distinct forms which are termed symmetrical and asymmetrical. These are illustrated in Figs. 4.30 and 4.31. As an example, the load shown in Fig. 4.32, can be replaced by the equivalent loading conditions shown in Figs. 4.33 and 4.34.

The transverse deformations for the symmetrical portion of the equivalent loading are shown in Fig. 4.35. Because of the symmetry, the two girders have identical deflected shapes in the longitudinal direction. This shape is as shown in Fig. 4.36. The reversals in curvature near the supports are due to end restraints on the girders. The end restraints may be
due to the **flexural** and torsional stiffness of the supports, due to adjacent spans, or both.

In the case of symmetrical loading, there are three unknowns which vary along the span length. These are the vertical deflection ($y$) of the girders, the angle of twist of the girders ($\omega$) and the moment ($m$) in the connecting slab. Because of the symmetry, the connecting slab is subject to a constant moment, as shown in Fig. 4.37 and is without a shear

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**Fig. 4.34** Asymmetrical component of loading shown in Fig. 4.32

**Fig. 4.35** Transverse deformations for symmetrical loading.
force. The determination of the unknowns requires the solution of three equations which are:

\[
EI \frac{d^4 y}{dx^4} = p 
\]

\[
G \frac{d}{dx} \left( J \frac{d \omega}{dx} \right) = -y + m 
\]

\[
m = \frac{\omega}{K + a + b} 
\]

In the above equations, the terms are defined as follows:

- \( E \) = elastic modulus of the concrete (psi).
- \( G \) = shear modulus of the concrete (psi).
- \( I \) = moment of inertia of each tubular girder (in”).
- \( P \) = a uniformly distributed line load of constant magnitude extending from support to support (plf).
- \( \omega \) = rotation of a tubular girder (radians).
- \( \gamma \) = uniformly distributed line moment applied to the tubular girder from support to support (ft-lbs).
- \( m \) = moment in the connecting slab (ft-lbs).
- \( a, b \) = elasticity constants as defined in Appendix C for the method of support constants.
- \( K \) = flexural elasticity of the joint at each end of the connecting slab.
- \( J \) = torsional constant.

For members having a non-variable torsional constant \((J)\), Equation 4.4 becomes

\[
GJ \frac{d^2 \omega}{dx^2} = -\gamma + m 
\]
Fig. 4.37 Shear and moment conditions for the connecting slab under symmetrical loading.

Fig. 4.38 Transverse deformation for asymmetrical loading.

Fig. 4.39 Shear and moment conditions for the connecting slab under asymmetrical loading.
Under the asymmetrical portion of the equivalent loading, the structure deforms as shown in Fig. 4.38. In this case, there is no moment in the connecting slab at its midpoint and the slab is subjected to a constant shear of \( q \). The end moments for the connecting slab, as shown in Fig. 4.39, are equal to:

\[
m = qd
\]

There remain three unknown variables. These are the girder deflections \( y \), the girder rotations \( w \) and the shear force \( q \) in the connecting slab. The equations for the three unknowns are:

\[
EI \frac{d^4y}{dx^4} = p - q
\]

\[
G \frac{d}{dx} \left( J \frac{d\omega}{dx} \right) = -\gamma + d'q
\]

which for a non-variable torsional constant \( J \) becomes

\[
GJ \frac{d^2\omega}{dx^2} = -\gamma + d'q
\]

and

\[
y = -d'\omega + d^2q \left( K + a - b \right)
\]

All of the terms in the above equations have been defined except for \( d \) and \( d' \) which are defined in Fig. 4.33.

The longitudinal deformation of the two girders under asymmetrical loading may be as shown in Fig. 4.40. The torsional stiffness of the end diaphragms together with the flexural stiffness of adjacent spans account for the reversal of curvature in the curves.

A correct solution requires the restraints of the supports be taken into account in the analysis. For symmetrical loading, both girders deflect vertically as well as rotate equally at the supports. Therefore, they do not induce a torsional moment in the support diaphragms. Because the two girders have equal but opposite support torsional moments when symmetrical loaded, a circular bending will be induced in the support diaphragms. The bending deformation of the support diaphragms due to the torsional moments is generally small and can normally be neglected. In the case of asymmetrical loading the vertical deflections of the girders are in opposite directions and hence a torsional moment is induced in the support diaphragms. The torsional rigidity of each support diaphragm is additive to the flexural rigidity of the beams from the adjacent span, if any exists. The deformations of the support diaphragms are generally
very small and can be neglected without serious errors being introduced into the calculations. **Flexural** rigidity of adjacent spans or of a support, on the other hand, have an important influence on the deflection curve of a girder and hence should not be ignored.

The six equations, 4.3, 4.4, 4.5, 4.8, 4.9 and 4.11 can be transformed into linear equations by assuming a particular distribution of the shear force (q) and moment (m) in the connecting slab. It is known the distribution of the shear force (q) closely follows the shape of deflection curve (y) of the girders. Hence, the value of the shear force (q) can be assumed to vary parabolically from zero at the support to a maximum value (q_o) at midspan without the introduction of significant error. In this case, the magnitude of the shear force (q_x) at a distance of x from midspan is:

\[ q_x = q_o \left( 1 - \frac{4x^2}{L^2} \right) \]  

The angle of twist of a girder under a torsional moment (m) is equal to:

\[ \omega = K_t m \]  

in which \( K_t \) is the torsional elasticity constant of the girder. The actual value of \( K_t \) is maximum at midspan and is equal to zero at the torsion-
ally restrained) support. The value of $K_i$ can be assumed to be constant along the span without the introduction of significant error. The value of $K_i$ for a constant moment ($m$) along the span is

$$K_i' = \frac{L^2}{8GJ}$$

(4.14)

while for a moment which varies parabolically from $m$ at midspan to zero at the supports the value of $K_i$ is:

$$K_i'' = \frac{5}{48} \frac{L^2}{GJ}$$

(4.15)

With the simplifying assumptions described above, it can be shown the values for the shear $q$ and moment $m$ in the connecting slab can be determined for a single line load ($p$), uniformly distributed along the span, by first determining the primary moments for unit loads in the frame under the assumption fictitious supports exist under the webs of the tubular girders as shown in Fig. 4.29. From the primary moments so obtained, the reactions at points 1, 2, 3 and 4, as shown in Fig. 4.29, are determined. The terms $L_1$ through $L_5$ are defined in Fig. 4.29. Using these reactions, one computes the equivalent loads and moments on girders 1 and 2 as follows:

$$p_1 = R_1 + R_2$$

(4.16)

$$\gamma_1 = (R_2 - R_1) \frac{L_2}{2}$$

(4.17)

$$p_2 = R_3 + R_4$$

(4.18)

$$\gamma_2 = (R_4 - R_3) \frac{L_4}{2}$$

(4.19)

From these, the values of the corresponding symmetrical and asymmetrical girder loads and moments are determined from the following:

$$p_1' = \frac{p_1 + p_2}{2}$$

(4.20)

$$\gamma_1' = \frac{\gamma_1 - \gamma_2}{2}$$

(4.21)

$$p_1'' = \frac{p_1 - p_2}{2}$$

(4.22)

$$\gamma_1'' = \frac{\gamma_1 + \gamma_2}{2}$$

(4.23)
From these values, the moment due to symmetrical loading $m_1$ is computed from:

$$m_1 = \frac{\gamma_1'K_t'}{K + a_3 + b_3 + K''} \quad (4.24)$$

in which $K$, $a_3$ and $b_3$ are the elasticity constants for the connecting slab.

The shear in the connecting slab due to the asymmetrical vertical loads are determined from:

$$q_2 = \frac{\gamma'' + K''(L_2 + L_3)^2 + \frac{L_3^2}{4}(K + a_3 - b_3)}{4} \quad (4.25)$$

in which $\gamma'$ is the midspan deflection of one of the tubular girders, with due regard to the restraint of the supports, under a uniformly distributed unit load and $\gamma''$ is the midspan deflection of one of the tubular girders due to a parabolically distributed load having a value of unity at midspan and zero at the supports, with due regard to the elasticity of the supports. From this, one obtains,

$$m_2 = q_2 \frac{L_3}{2} \quad (4.26)$$

The shear and moment in the connecting slab due to the asymmetrical moments are computed from:

$$q_3 = \frac{\gamma''''K_t''(L_2 + L_3)^2 + \frac{L_3^2}{4}(K + a_3 - b_3)}{4} \quad (4.27)$$
and

\[ m_3 = q_3 \frac{L_3}{2} \]  

(4.28)

The total support secondary moments at points 2 and 3 are equal to:

\[ m = m_1 + m_2 + m_3 \]  

(4.29)

and the total secondary shear in the connecting slab is:

\[ q = q_2 + q_3 \]  

(4.30)

The secondary moment at the critical section (See Fig. 4.41) for negative moment is:

\[ m' = m_1 + (q_2 + q_3) d \]  

(4.31)

The results of such an analysis can be plotted as influence lines as shown in Fig. 4.42 in which it will be seen the primary and secondary moments at the support, as well as their sum, are plotted together. A similar diagram for the midspan moment is given in Fig. 4.43.

The influence lines shown in Figs. 4.42 and 4.43 were developed for line loads rather than for concentrated loads and are intended for use with lane loadings rather than with truck loadings. The lane loadings of the AASHTO Specifications include a concentrated load in addition to the uniform loads. The effect of the concentrated load can be included in the analysis by increasing the magnitude of the uniform lane loading used in the design as follows:

\[ w' = w - \left( \frac{y_u + y_c}{y_u} \right) \]  

(4.32)

in which \( w' \) is the adjusted lane load in pounds per linear foot.

\( w \) is the lane load provided in the AASHTO Specification

\( y_u \) is the midspan deflection of a girder, with due regard for end restraint, due to the AASHTO lane load.

\( y_c \) is the same as \( y_u \) except for the appropriate concentrated load applied at midspan of the girder.

The lane loading generally controls for bridges of long span.

For bridges of short and moderate span, the truck loading controls the design and the secondary moments due to girder deflection and rotation are small and normally neglected.
Fig. 4.42 Influence line for negative moment at the critical section of the connecting slab.

Fig. 4.43 Influence line for the positive moment at midspan of the connecting slab.
For beams which have variable moments of inertia and torsional constants, a trial and error method of analysis must be used. The procedure can be illustrated by considering the case of asymmetrical loading which is defined by equations 4.8, 4.9 and 4.11. The specific procedure is as follows:

1. Calculate the ordinates to the elastic curves for one of the girders under a uniformly distributed load with due regard to end restraints.
2. Assume the variation in the intensity of the shear in the connecting slab $q$ is proportional to the deflection computed in Step 1. Designate the unknown shear force at midspan as $q_0$.
3. Using the variation in the shear force determined in Step 2, solve equations 4.8 and 4.9 for the values of $y$ and $\omega$ respectively.
4. Using the values of $y$ and $\omega$ from Step 3, solve equation 4.11 and obtain a new value for $q_0$ at midspan and a new distribution of $q$ along the span.
5. If the distribution of $q$ obtained in Step 4 varies appreciably from the assumed distribution, the procedure is repeated using the distribution obtained in Step 4 as the new assumed distribution of $q$.

It is reported the method results in rapid convergence for the effect of the vertical load. Convergence is slower for the effect of the moments (Ref. 49).

Influence lines of the types shown in Figs. 4.42 and 4.43 can be constructed from the results of finite element or other methods of structural analysis in instances where one does not wish to employ the approximate method just described and the cost of the more sophisticated analysis can be justified.

### 4.5 Proportioning the Superstructure

Before considering the proportioning of segmental bridges, a review of a few typical bridge superstructure cross sections will be made. A review of this type is helpful in forming a basis for future design work. Four subject areas have been selected for consideration. These are constant depth superstructures; variable depth superstructures; superstructures with transverse floor beams and superstructures incorporating two or more tubular girders.

In European practice, for reasons of economy, superstructures of constant depth have normally been used for spans up to 200 feet. For spans in excess of 200 feet, superstructures of variable depth have more often been found to be more economical than constant depth structures.
On occasion, constant depth superstructures have been used for spans considerably greater than 200 feet, generally because of esthetic considerations. For optimum economy in Europe, superstructures of constant depth that are constructed in cantilever generally have depth-to-span ratios between one in eighteen and one in twenty. For superstructures of variable depth, depth-to-span ratios of the order of 1 to 18 at the piers and 1 to 40 at midspan are frequently used.

Constant depth superstructures with depth-to-span ratios approaching 1 to 30 are possible under some conditions even when the cantilever erection technique is used. Some sacrifice in economy would be expected when the shallower depths are used. When other erection techniques are used, such as casting the segments in place on falsework rather than using “travelers” or using temporary falsework towers in combination with temporary prestressing tendons in the erection of precast segments, the more slender sections should prove to be more feasible.
The lighter live loads used in United States bridge design, as compared to those used in other parts of the world, should also contribute to the feasibility of using less superstructure depth. The depth-to-span ratios which result in optimum economy in the United States will become more apparent as the use of this method becomes more common and widespread. “Americanization” of the construction details as well as the construction methods will undoubtedly occur. This evolution may have a profound effect on the depth of superstructure that is most economical for a particular structure as well as on the overall economy of the method.

Examples of cross sections of bridge superstructures having constant depth are those which were used on Interstate Highway I-70 over Vail Pass in Colorado. These are illustrated in Fig. 4.44. It should be noted that the dimensions of the upper slabs are identical for segments of all depths as are the slopes of the webs. The principal variable dimensions include the depth of the segments as well as the width and thickness of the bottom slabs. The bottom slab thickness was increased near the piers in order to accommodate the large negative moments which occur in these areas. This method of varying the principal dimensions was selected to facilitate the manufacture of segments having different depths. The depth-to-span ratios used on Vail Pass, due to esthetic con-
siderations, are generally more slender than that which would be expected by European practice to result in greatest economy.

Another example of a superstructure of constant depth is the Pine Valley Creek Bridge which is shown in Fig. 4.45. This structure has a main span of 450 feet and hence is in the span range that would normally be done with a variable depth in European practice. Note that the depth-to-span ratio for the main span of this structure is 1 to 23.7. It should also be pointed out that the thickness of the bottom flange in the main span of this structure varied from 6.5 feet at the piers to 10 inches at midspan. The exceptionally great bottom flange thickness at the pier was required as a result of using a relatively slender constant depth superstructure with a relatively great span length.

The Saint Cloud Bridge in Paris was constructed with precast segments having the shape shown in Fig. 4.46. This structure has a maximum span of 335 feet with a constant superstructure depth of 11.8 feet. This results in a depth-to-span ratio of 28.3. This cross section is not the most efficient from a structural viewpoint. The dimensions were selected in order to achieve a desired appearance.
Superstructures with variable depth can be designed with a wide combination of depth-to-span ratios at pier and midspan. The intrados can be formed of straight chords or of circular curves. Surprisingly, with segmental construction parabolic curves are not desirable due to construction considerations. Spans in excess of 750 feet have been constructed segmentally with cast-in-place superstructures of varying depth.

An interesting example of a bridge constructed with varying depth is the Oléron Bridge which is shown in Fig. 4.47. This structure has 26 central spans of 259 feet as well as a number of shorter spans at each end of the structure. The depth of the segments used in the central spans are 8.25 feet and 14.75 feet at midspan and at the piers respectively. This results in depth-to-span ratios of 1 to 17.5 at the pier and 1 to 31.6 at midspan. This structure was constructed using precast segments erected with a specially constructed gantry.

The typical cross section of the segmental alternate design of the Napa River Bridge is shown in Fig. 4.48. This bridge has a main span of 250 feet and depth-to-span ratios of 1 to 32.26 at midspan and 1 to 20.83 at the piers adjacent to the longer spans. The bridge also has 7 spans of 150 feet each which have a constant depth of 7.75 feet. (Depth-to-span ratio of 1 to 19.4.) Lightweight concrete was specified for the Napa River Bridge.

Fig. 4.47 Oléron Bridge between the European Continent and the island of Oléron in France.
Fig. 4.49 Typical cross section of the Napa River Bridge, California.

Fig. 4.49 Typical cross section of the Saint André de Cubzac Bridge in France.
The viaduct at Saint Andre de Cubzac is an interesting example of a bridge superstructure which has transverse floor beams in the deck. The cross sectional dimensions of the segments are shown in Fig. 4.49 from which it will be observed that the segments are quite wide (54.5 feet) and have only two webs. The provision of the transverse floor beams results in a relatively thin deck which spans longitudinally rather than transversely. This structure has central spans of 312.5 feet and depth-to-span ratios of 1 to 17.3 and 1 to 33.5 for the pier and midspan sections respectively.

As mentioned above, bridge superstructures can be formed of two or more tubular girders which are connected together by a deck slab as shown in Fig. 4.50. This scheme can be used to facilitate providing roadways of variable width. For superstructures of constant width, this scheme permits the use of segments which are smaller, and hence easier to cast, transport and erect, than would be the case if a single full-width segment were used.

Because of the unique distribution of moments that exists in a superstructure that is erected with the cantilever technique, the desirable end span length of multi-span bridges is of the order of 65 to 70% of the length of the interior spans. If shorter end spans are used, uplift can be experienced at the abutment end of the end span due to live load on the first interior span. Hence, special (and usually costly) support details must be provided at the ends of short end spans so that the ends can move neither up nor down. End spans longer than 70% of the interior spans can of course be used with some sacrifice in economy.
4.6 Proportioning the Segment

It is interesting to compare typical cross-sections of conventional cast-in-place box girder bridges such as have been used so effectively in the United States to those of typical segmental bridges of similar overall width and span length. Such a comparison is made in Fig. 4.51 for a bridge with a span of 150 feet. It will be observed that the depth-to-span ratio of the conventional cast-in-place superstructure is of the order of 1 in 25 while that for the segmental bridge is 1 in 18.8. There is a simple explanation for this difference. Firstly, the segmental bridge is erected in cantilever and hence is subjected to a greater dead load negative moment than is the conventional cast-in-place superstructure. In addition, the bottom flange of the segmental bridge, which is the compression flange in areas of negative moment, has less width than is provided in the conventional structure. Hence, more depth is required to compensate for these factors.

In establishing the dimensions of the cross-section of a segment, one must consider factors in addition to the longitudinal flexural stresses. The flexural stresses are of course a major consideration but considera-
tions other than the longitudinal \textit{flexural} stresses may control the dimensions of a cross-section. If one examines the function of each of the components of the cross-section, the following observations will be made:

Flanges With respect to the longitudinal bending moments, the width and thickness of the flanges, as well as the distance between them, have direct bearing on the magnitude of the \textit{flexural} stresses. In other words, the dimensions of the flanges and the distance between them must be great enough to confine the \textit{flexural} stresses within the allowable limits. This accounts for the fact that the bottom slab thickness frequently must be greater in the vicinity of the piers than at \textit{midspan}. The large negative dead load moment which exists at the pier makes this factor especially important when cantilever erection procedures are used. The bottom flange must also be of \textit{sufficient} thickness to resist the transverse dead and live load moments to which it is subjected. The top flange must act as the deck and hence be of adequate thickness to resist the transverse moments and shears due to the dead and live loads. An important practical consideration is that the flanges must be \textit{sufficiently} thick to accommodate the reinforcing steel and prestressing tendons that must be embedded in them. A minimum thickness for the bottom slab of 5.5 inches is frequently specified while for the top slab a minimum thickness of 6 inches is often used.

Web Layout The number and location of the webs has significant influence on the transverse bending moments in the deck as well as in the bottom slab. Optimum deck design is achieved when there is a good balance between the effects of the minima moment which occurs in the deck at the location of the web and the maxima moment which occurs in the deck between webs. This balance is not only affected by the number and spacing of the webs but also by the thickness of the deck at \textit{midspan}.

\begin{center}
\includegraphics[width=0.5\textwidth]{Fig_4.52.png}
\end{center}

Fig. 4.52 Top slab proportions.
and at the support. The designer has the option of determining the amount of slab overhang he wishes to provide in a particular design. This is illustrated in Fig. 4.52 in which it will be seen that coefficient “a”, which is a measure of the slab overhang, is a major factor in achieving an efficient design for the deck. The designer also must establish the thickness of the deck at the supports as well as at intermediate points and in the deck overhang. These decisions also affect the efficiency of the deck design.

Exterior webs are often inclined as a means of attempting to enhance the appearance of a segmental bridge. This is illustrated in Fig. 4.53. It will be recognized that inclining the web reduces the transverse span of the bottom slab. This in itself is not of major importance. The fact that the bottom slab is reduced in width by using inclined webs is an important consideration in areas of negative moment. In areas of negative moment the bottom slab is the compression flange. It is frequently necessary to increase the bottom slab thickness in areas of negative moment as a means of confining the longitudinal flexural stresses to an acceptable level. A narrow bottom flange will require a greater increase
in thickness to control the compressive flexural stresses than would be required for a wider bottom flange.

**Web Thickness** A primary function of the webs is to resist the longitudinal shear stresses. Because these stresses are the greatest near the piers, webs of superstructures having constant depth are frequently made thicker in the vicinity of the piers than they are at midspan. Thickening of the webs near the piers to accommodate shear stresses is not normally required in superstructures of variable depth due to the reduction of the shear force acting upon the webs that results from the vertical component of the compressive force in the inclined bottom flange. This is known as the Résal effect. (See Section 5.2.) The reduction in the shear force acting upon the concrete section as a result of the curvature of the longitudinal tendons should be taken into account in bridges of constant as well as variable depth.

For segments having cross sections which are symmetrical about a vertical axis and which have only two webs, the shear force resulting from the dead and live loads, as well as from the longitudinal prestressing, is equally divided between the two webs. For segment cross-sections which are not symmetrical about a vertical axis or which have more than two webs, the distribution of the shear forces to the webs
may not be equal and must be determined using special methods. This was explained in detail in Section 3.5. The webs also must resist flexural stresses which are induced by transverse dead and live loads. In establishing the web thickness that is to be used in a particular design, in addition to consideration of stresses, one must also consider the space required to accommodate the embedded items as well as to provide for the necessary minimum concrete thicknesses over and between the embedded items. The provision of sufficient space for placing the concrete during construction of the segments is very important. If the end anchorages of the longitudinal prestressing tendons are to be terminated in the web at the joints between the segments, as has been done extensively in the past, the web must also perform the function of an end-block. The web thickness may have to be made greater than would be required by other design considerations when the end anchorages are placed in the webs.

Web Stiffeners A relatively recent innovation in precast segmental bridge construction has been the provision of web stiffeners inside the segment as shown in Fig. 4.54. The stiffeners are provided in order to achieve a convenient means of anchoring temporary prestressing tendons that are used in some erection techniques as well as to accommodate permanent prestressing anchorages. They can also be used to strengthen precast segments with respect to stresses which are induced during transportation and erection.

Fillets The fillets which are provided between the webs and flanges serve an important function in relieving stress concentrations which could result from transverse flexure in these locations. They also serve the important function of providing necessary space for longitudinal pre-stressing tendons together with longitudinal and transverse reinforcing. An illustration of the concentration of tendons in the joint between a web and top flange is given in Fig. 4.25.

![Fig. 4.55 Forces on a joint between precast segments during erection.](image)
Shear Keys  Shear keys are provided in the webs and flanges of precast segments. They serve two functions. The first is to align the segments when they are erected. The second is to transmit the shear force between segments during construction when the epoxy bonding compound applied to the segment joint is still plastic and behaves like a lubricant. The web shear keys alone must be relied upon to transfer the shear force across the joint. The plastic bonding compound precludes the existence of friction between the segments. The forces acting on a joint during erection are as shown in Fig. 4.55. Although in the past the epoxy bonding compound provided in the joints between precast segments has been relied upon to transmit shear stresses in the completed structure, the use of multiple shear keys as shown in Fig. 4.56 makes this unnecessary. The web shear keys should be designed using the stresses permitted in Chapter 17 of the Building Code Requirements for Reinforced Concrete (Ref. 7).

Anchorage Blocks  In precast segmental construction the longitudinal prestressing tendons are frequently anchored inside of the segments at
internal anchorage blocks as shown in Fig. 4.54 rather than in the webs at the joint between segments as has been traditional in cast-in-place construction. Two advantages evolve from this practice. Firstly, the webs do not have to function as end blocks and resist the high tensile stresses that occur in the concrete which is immediately under the end anchorages. Secondly, this procedure permits the construction crews that are installing and stressing the permanent prestressing tendons to work independently of the crews that are erecting the segments and installing the temporary prestressing. Internal anchorage blocks are frequently, but not always, accompanied by web stiffeners which were described above.

Segment Length In designing and detailing a bridge that is to be constructed segmentally, one needs to know the length of the segments in order to completely analyze the structure and check the stresses in the joints between the various segments. One cannot be certain his design is complete unless this is done. On the other hand, the designer generally prefers to leave decisions such as this to the contractor with the purpose of not being unnecessarily restrictive and obtaining the lowest cost for the owner. One solution to this problem is to select a segment length that seems reasonable under the circumstances and prepare the complete design on this basis. An appropriate clause in the specifications can be provided to allow the contractor to use other than the detailed segment length and prestressing details providing he submits proof of the adequacy of any changes he wishes to make.

With these thoughts in mind, one should consider the factors which dictate segment length. A primary consideration influencing the size of the segment is the unbalanced moment the substructure, substructure-superstructure connection, or substructure foundation, can withstand. In cast-in-place construction, the design of the traveling forms is affected by the weight of the segments they support. One large company that has constructed many cast-in-place segmental bridges has devised a standard adjustable traveler that will accommodate a segment having a maximum length of 16 feet. This then seems like a reasonable upper limit for cast-in-place segments. Precast segments which must be hauled over the public highways, should not exceed lengths and weights that would require special fees, transportation costs or special permits. A length of 8 feet would normally be able to be hauled without exceeding legal load widths but segment weights that can be accommodated must be investigated for each locality. Precast segments which are to be job-site cast and not hauled over public roads or highways are limited only by the practical considerations of fabrication, handling and erection. For segments which are to be cast-in-place on falsework, the practical limi-
tions of available falsework, construction sequence and span lengths will dictate the lengths of segments to be used. Needless to say, not all contractors who bid any one job will agree as to the optimum segment length.

From the above it will be apparent that many basic considerations other than stresses resulting from longitudinal bending should be taken into account in establishing the cross-sectional dimensions of the segment. Some of these considerations are interrelated. If, for example, it is found that the thickness of the webs or flanges of a section must be increased in order to accommodate the necessary embedded items or to properly control transverse bending stresses, the dead load of the structure will be increased as will be the stresses resulting from longitudinal dead load bending moments and shears. It should also be apparent that the method of erection that is to be used can have a very significant influence on proportioning segments for longitudinal flexural stresses. Hence, it should be apparent that proportioning for longitudinal bending involves the commonly used procedure of adopting a trial section which

![Diagram of elastic curves](image)

Fig. 4.57 Comparison of the elastic curves for no hinge within a span to those for one at midspan and one at the quarter point.
has dimensions that are assumed to be adequate for the conditions at hand and reviewing the adequacy of the assumed section. If inadequacies are found, the dimensions of the section are adjusted as necessary and the process is repeated until a complete and satisfactory solution is developed.

4.7 Intermediate Hinges

Variation in length due to the effects of temperature, creep and shrinkage must be accommodated in bridge design. This factor becomes in-

Fig. 4.58 Procedure in providing an intermediate hinge within a span at other than midspan.
creasingly important with the length of the bridge. This is most often done by providing joints within one or more spans which include provision for translation and rotation. The joints are referred to as hinges.

The provision of a hinge within a span results in a discontinuity of the structure. Hence, abrupt angles between the tangents to the structure on each side of the hinge may develop due to applied loads or the effects of creep. Elastic curves for three hinge conditions in an interior span, which is a part of a multi-span continuous beam, are shown in Fig. 4.57. The curves are for the condition of a uniformly distributed live load with a hinge located at mid-span, at quarter point and without a hinge. Note that the angle between the tangents to the elastic curve on each side of the hinge is nearly three times as great when the hinge is at midspan than when it is at the quarter point. Hinges should be avoided where possible. When required, it is better to place them at the quarter point than at midspan.

Hinges can conveniently be provided between midspan and the support at any desired location in segmental construction. This is done by providing temporary prestressing and blocking across the hinge until such time as the abutting cantilevers are joined at midspan. The hinge is rendered free to translate and rotate by the removal of the temporary prestressing tendons and blocking. This procedure is illustrated in Fig. 4.58.

It should be recognized that releasing a hinge which was fixed temporarily during construction results in a redistribution of the moment that existed at the joint while it was restrained. The effect of releasing the hinge on the stability of the structure, with the configuration it has at the time the hinge is released, must be investigated. At the time the hinge is released, the portion of the structure from the hinge back toward the direction from which the construction proceeded is continuous while that portion from the hinge toward the direction the construction is proceeding may be simply supported and unable to support the reaction from the continuous structure without auxiliary supports. In addition to the moment which results from the releasing of the hinge, moments due to the removal of temporary hinge prestressing tendons must be taken into account when the hinge is released.

4.8 Support Details

Tall slender piers are frequently relatively flexible in comparison to the superstructure and hence the moments induced in the piers due to vertical loads as well as due to superstructure length and rotation variations are
relatively small. The result is that the taller piers can frequently be constructed with a moment-resisting (fixed) connection to the superstructure. On the other hand, short stiff piers frequently must be provided with bearing devices in the joint between the top of the pier and the superstructure. Properly selected bearing devices permit the designer to control the moments and shears that are induced in the pier due to vertical loads, superstructure length variations and horizontal forces. This, of course, is true in bridges of all types and is not unique to segmental bridges.

In segmental bridges, the design of the superstructure segments over the piers must be such as to be able to transfer the required forces and moments between superstructure and substructure.

Consider the pier proportioned with a width that is less than the width of the superstructure soffit as shown in Fig. 4.59. It is apparent that the

Fig. 4.59 Pier with a width less than the width of the superstructure
Fig. 4.60  Pier with a width equal to the width of superstructure

Fig. 4.61  Pier cross section.
diaphragm provided in the pier segment must be sufficiently strong to transfer the vertical loads from the superstructure webs to the pier. If the connection is designed to fix the top of the pier and the superstructure with respect to rotations, the diaphragm must also be capable of

Fig. 4.62 Pier cross section

Fig. 4.63 Pier cross section.
withstanding the torsional moments which occur between the superstructure webs and the pier shaft. The torsional stresses in the diaphragms could be avoided by using a pier that is as wide as the superstructure soffit as shown in Fig. 4.60.

The cross-sections of three pier shafts are shown in Fig. 4.61, 4.62 and 4.63. The cross section of Fig. 4.61 consists of two intersecting wall-like elements. The short elements which project along the center line of the bridge may be considered desirable from an architectural viewpoint but such projections can seriously restrict the moment capacity of a section and can create special problems in effecting a moment transfer between the superstructure and substructure. The cross section shown in Fig. 4.62, although it may be less desirable from an appearance standpoint, is far more efficient structurally than that of Fig. 4.61. This is particularly true when moment transfer between pier and superstructure is to be made. A pier cross section such as is shown in Fig. 4.63 is offered as one of many possible compromise solutions for obtaining a structurally efficient section that still provides protrusions and hence shadows to enhance the appearance of the finished structure. Piers which are of variable dimension, as shown in Fig. 4.64, are sometimes used in an effort to enhance the appearance of a structure. The variable

![Fig. 4.64 Pier with variable dimension. (Courtesy California Department of Transportation).](image)
Fig. 4.65  Wide pier for moment transferring ability.

Fig. 4.66  Wide pier for moment transferring ability.
dimensions are not normally desirable from the standpoints of cost and structural efficiency.

Supports which have a reasonable width in the direction of the span at the junction with the superstructure, say of the order of eight to ten feet, as shown in Fig. 4.65, facilitate superstructure erection when the cantilever erection method is to be used. When the width is adequate, the unbalanced moment which occurs at the support with the cantilever erection technique can be transferred with temporary prestressing tendons together with temporary bearing pads. Provision of adequate space and means of access for inserting hydraulic jacks, as shown in Fig. 4.66, permits removal of the temporary bearings and insertion of the permanent bearings upon the completion of the cantilever. Provision of space and access for the hydraulic jacks also permits adjustment of the position of the completed cantilever before the continuity joint between adjacent cantilevers are cast. After final adjustment of the cantilever and insertion of the permanent bearings, the connection can be left free to rotate, and hence act as a hinge, or can be fixed against rotation by grouting the joint and inserting permanent prestressing tendons across the joint.

4.9 Construction Details

When using the cantilever erection technique together with tendons composed of high strength wires or strands, tendon layouts similar to those shown in Fig. 4.67 are frequently employed. The tendons are generally referred to as “cantilever” tendons and “continuity” tendons.

The cantilever tendons are those which are located in the areas of negative moment and are normally placed and stressed during the erection of precast segments or casting of cast-in-place segments. They are generally on a trajectory that is symmetrical about the centerline of the pier. Cantilever tendons may be extended downward from the upper surface of the structure as shown in Fig. 4.67 or may terminate near the upper surface of the structure. When extended downward into the webs of the segments, the cantilever tendons exert a vertical component of prestress which reduces the shear force imposed upon the concrete section. When the tendons (either cantilever or continuity) are extended into the webs, the French Code (Ref. 50) requires the shear design be based on the net section.

The continuity tendons are those that provide prestressing in the areas of positive moment. They are placed and stressed after the closure joint has been placed. The continuity tendon may or may not be ex-
Fig. 4.67 Typical layout of longitudinal wire or strand tendons with the cantilever erection technique.

Fig. 4.66 Tendon layout for tendons which do not extend into the webs.
Fig. 4.69 Recommended bridge rail detail.

Fig. 4.70 Bridge rail detail that is not recommended.
tended up into the webs of the superstructure as shown in Fig. 4.67.

A tendon layout in which the continuity and cantilever tendons are not extended into the webs is shown in Fig. 4.68. In the layout in Fig. 4.68 the tendons are shown to be anchored at anchorage blocks inside of the segments. The cantilever tendons could be anchored in the ends of the segments (at the joint between segments) if the internal anchorage blocks are not provided.

When bar tendons are employed, the prestressing force is varied by terminating bars at various locations along the span in much the same manner as is done in reinforced concrete. Curvature of bar tendons is kept to a minimum due to the fact it is much easier to place bar tendons on straight trajectories.

The webs of segmental bridges are sometimes prestressed as a means of reinforcing for shear. This may be accomplished to various degrees by extending wire or strand tendons into the webs as shown in Fig. 4.67. It can also be accomplished by placing tendons, either vertically or diagonally, in the webs. Some engineers feel that the webs of all long-span concrete bridges should be prestressed as a means of controlling shear-induced web tensile stresses. It must be recognized that many existing concrete bridges having webs reinforced with non-prestressed web reinforcing are giving very satisfactory service.

Internal anchorage blocks, together with “web stiffeners” as shown in Fig. 4.54 are sometimes employed in precast segmental construction as a means of avoiding terminating the tendons at the joints between segments and providing a means for attaching temporary prestressing tendons. These details have been found to be useful in reducing the time required to erect precast segments. Basically it permits the workmen who are erecting the segments to work almost independently of those installing and stressing the cantilever tendons.

The detail used for the bridge barrier rail deserves mentioning. The recommended detail is shown in Fig. 4.69. It is characterized by the fact the cast-in-place railing extends below the edge of the deck. In this manner any difference in grade between the ends of cantilevers which may exist at the cast-in-place closure joints, will be hidden. The detail shown in Fig. 4.70 is obviously less costly because the forms are more easily erected and stripped. This is because they do not extend below the top of the deck. The latter is less desirable from an appearance standpoint for the reason previously mentioned.
5 Additional Design Considerations

5.1 Introduction

Included in this Chapter are brief discussions of several topics which may be completely understood by the reader and hence need not be given further consideration. On the other hand, experience has shown that not all bridge designers are familiar with these subjects. It is for this reason that this chapter has been included.

5.2 Design for Shear

Contemporary shear design provisions of the ACI 318 Building Code Requirements for Reinforced Concrete (Ref. 51) for prestressed concrete flexural members involves mathematical relationships that are more complex than the relationships contained in the AASHTO Specification (Ref. 6). The AASHTO Specification permits the use of the ACI criteria. The current provisions of ACI 318 may more accurately reflect the true shear strength of prestressed concrete than was predicted by previous design
criteria. Many engineers, however, question the necessity for the more complex relationships and point out that the new relationships do not show obvious economy with respect to shear design nor were the previously used relationships obviously inadequate.

The fact that some engineers believe the ACI criteria are unconservative when applied to large concrete beams is worthy of note. This group of engineers is of the opinion that when the shear capacity of a concrete beam is exceeded by the design load, the entire shear force should be carried by reinforcing steel. They believe the combined effects of dowel action, aggregate interlock and friction in the compression flange can be significant in the shear strength of small beams but not significant in large beams (Ref. 52). They point out the fact that shear strength relationships contained in ACI 318-71 have been confirmed by results obtained from tests of small beams.

The contemporary shear design criteria of ACI 318-71 lends itself to computer analysis. A mini-computer or programmable calculator of moderate size will suffice. Once the computer has been properly programmed, the fact that the computations are somewhat complex is unimportant. One cannot, however, properly program a computer unless he completely understands the meaning of the terms which are included in the relationships involved.

There is considerable confusion in the meaning of the terms in Eq. 1-1-1 of ACI 318-71. This equation, as revised in 1973, is:

$$v_{ci} = 0.6 \sqrt{f_c} + \frac{V_d + \left( \frac{V_i M_{ct}}{M_{max}} \right)}{b_w d} \quad (5.1)$$

The official ACI definitions of the terms in the second part of the equation, are as follows:

- **Vd** = Shear force at section due to dead load.
- **Vi** = Shear force at the section considered due to externally applied design loads occurring simultaneously with **M_max**.
- **MU** = Bending moment causing flexural cracking at the section considered due to superimposed loads.
- **M_max** = Maximum bending moment at the section considered due to externally applied design loads.
- **b_w** = Web width, or diameter of circular section, in.
- **d** = Distance from the extreme compression fiber to centroid of reinforcement. in.

It is the opinion of the authors that the definition of **V_d** should be the shear force at section due to service dead load because it is not intended that this
load be factored. The term “externally applied design loads” which is used in the definition of $V_i$ and $M_{\text{max}}$ should be defined as the design loads less the service (unfactored) dead load. It should be recognized that design loads are factored service loads.

It should be remembered that $V_{ci}$ is intended to be a measure of flexurally initiated shear cracks while $v_{cw}$ is intended to be a measure of cracks resulting from excessive principal tensile stress. The relationship for $v_{cw}$ (Equation 11-12 of ACI 318-71) is as follows:

$$v_{cw} = 3.5 \sqrt{f_{c}'} + 0.3 \ f_{p} + \frac{V_p}{b_x d} \quad (5.2)$$

in which $V_p$ is the vertical component of the effective prestressing force at the section being considered and $f_{p}$ is defined as follows:

$$f_{p} = \text{compressive stress in the concrete, after all prestress losses have occurred, at the centroid of the cross section resisting the applied load or at the junction of the web and flange when the centroid lies in the flange, psi.}$$

(In a composite member, $f_{p}$ will be the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone.)

As an alternative to Eq. 5.2, one may compute the value of $v_{cw}$ as being the shear stress which results in a principal tensile stress of $4 \sqrt{f_{c}'}$ ($3 \sqrt{f_{c}'}$ should be used in lieu of $4 \sqrt{f_{c}'}$ for all-lightweight concrete while $3.4 \sqrt{f_{c}'}$ should be used for sand-lightweight concrete). For a member stressed longitudinally only, this can be expressed as follows:

$$v_{cw} = \sqrt{4 \sqrt{f_{c}'} (4 \sqrt{f_{c}'} + f_{p})} + \frac{V_p}{b_x d} \quad (5.3)$$

In the case of a member having a vertical prestress $f_{x}$ in addition to a longitudinal prestress of $f_{p}$, the shear stress $v_{cw}$ which results in a principal tensile stress of $4 \sqrt{f_{c}'}$ can be determined from:

$$v_{cw} = \sqrt{(4 \sqrt{f_{c}'} + f_{p}) (4 \sqrt{f_{c}'} + f_{p})} + \frac{V_p}{b_x d} \quad (5.4)$$

Although Eqs. 5.3 and 5.4 would have been considered much too difficult for every day use in the day of the slide rule, they are easily solved today on inexpensive electronic calculators.

It has been pointed out that prestressing tendons which are inclined to the gravity axis of a beam have a vertical component of force which usually...
is acting opposite in direction to the flexural shear force. Hence, the vertical component of the prestressing force should logically be taken into account when computing the actual stress on the concrete. Equation 5.2, which is an “allowable stress,” includes this effect as does Equation 5.4. It should be observed that the relationship for $V_{ci}$ (Eq. 5.1) does not include the effect of the vertical component of prestressing. The explanation for this given by ACI Committee 426 is that the effect of $V_p$ is included in the flexural cracking moment, $M_{cr}$ (which is included in Eq. 5.1) and hence $V_p$ does not have to appear in Eq. 5.1 (Ref. 53). In continuous bridge structures $V_p$ can be quite large in areas of negative moment and it seems logical that its effect be incorporated in the analysis. Research is needed on large continuous prestressed concrete beams with the objective of obtaining a better understanding of the shear strength of such members in the areas subject to negative moment.

Beams of variable depth and normal configuration have less shear force on their webs in areas where the compression flange is inclined to the gravity axis than would be revealed from a usual analysis of the flexural shear forces. The principle is illustrated in Fig. 5.1 in which a freebody

Fig. 5.1 Freebody of a portion of a bridge superstructure having variable depth. \( (\text{Resal effect}) \)
Diagram of a portion of a variable depth continuous beam is shown. The portion of the beam shown is near the support where both the shear force and the negative moment are large. If the angle of inclination of the bottom slab with respect to the gravity axis of the member is taken as \( \alpha \) and the force in the compression flange is designated as \( C \), there is a vertical component of the force \( C \). The vertical component is equal to \( C \sin \alpha \). The vertical component of the force \( C \) acts in a direction that reduces the shear force acting on the webs of the member. When applying this effect in an analysis under design loads, the force \( C \) should be determined based upon the design moment that is concomitant with the design shear force being considered;

Secondary moments and shear forces frequently exist in continuous prestressed concrete members. It is recognized that the effect of these should be included in strength calculations (Ref. 54). A load factor of unity is generally accepted as being appropriate when including these effects in strength calculations.

In view of the above, it would seem the provisions for shear design of prestressed concrete members, following the principles of ACI 318, be done by including the effects of variable depth and secondary reactions (with load factors of unity) in the computation of the design shear force at each section considered. Hence, with the AASHTO load factors, this relationship becomes:

\[
v_{ci} = 1.44V_D + 2.41 (V_{L+1}) + V_v + V_s
\]

in which

- \( V_D \) = Total dead load shear
- \( V_{L+1} \) = Shear due to live and impact loads
- \( V_v \) = Effect of variable depth
- \( V_s \) = Shear due to secondary effects of prestressing

The values of \( v_{ci} \) and \( v_{cw} \) would then be computed using equations 5.1 and 5.4.

In using the lane load design criteria of the AASHTO Specification, it will be remembered that the magnitude of the concentrated loads used in determining maximum values of moment and shear differ. Furthermore, in continuous spans, two concentrated loads are used in determining the maximum negative moment. Only one load is used in determining maximum positive moments (even in continuous structures) and maximum shears. Because the shear \( V_i \) is the shear at the section under consideration when the loads are of the magnitude and positioned to produce the greatest value of \( M_{max} \), the correct value of \( V_i \) is less than would be obtained if the
largest live load shear force which occurs at the section were used in determining \( V_i \). It should also be recognized that computing \( V_i \) using the largest live load shear force which occurs at a section, rather than the shear force which is concomitant with the loading which produces \( M_{\text{max}} \), is unconservative because it will result in a greater value of \( v_i \).

5.3 Horizontally Curved Bridges

Bridges which have closed cross sections, such as the box-girder bridges described in Chapter 3 and the segmental bridges described in Chapter 4 are efficient in resisting torsional as well as flexural moments. For this reason bridges of these types are especially desirable in structures which must have a horizontally curved alignment. The horizontal curvature results in torsional moment from both dead and live load. Intermediate diaphragms are generally provided in box-girder bridges that have significant horizontal curvature. Diaphragms are normally required at each support in order to transfer shears and moments from the superstructure to the substructure.

A method for the rigorous analysis of a horizontally curved structure, which requires the use of a computer, is described in the literature (Ref. 55). This rigorous method has been used to confirm the adequacy of an approximate method for usual design applications. The approximate method, which has been described by Witecki (Ref. 56) is simple to apply and does not require a computer.

The approximate method described by Witecki is based upon the relationship:

\[
m = \Delta s \sum \left( \frac{M}{R} + T \right)
\]

(5.6)

in which

- \( M \) = total external bending moment produced by dead load, superimposed dead load, live load and any other anticipated load-all computed as for a straight structure.
- \( R \) = radius of curvature.
- \( M/R \) = torque per ft. produced by bending moment.
- \( T \) = \( qe \) = torque per ft. produced by eccentrically applied load.
- \( s \) = length on which the torsional moment is being computed.

The method is used to determine the torsional moments as follows:

1. Determine the vertical dead, superimposed dead and live load.
2. Determine the uniformly applied torque moment which must be accounted for in the design. This torque moment should include the
effects of dead and live load eccentricity as well as centrifugal force and wind.

3. Assuming the bridge is on tangent (without horizontal curvature) compute the longitudinal bending moments for the applied loads.

4. The summation of the bending moments at each section should then be divided by the radius of curvature and the quotient added to the torque determined in step 2. The result so obtained is the term \( \frac{M}{R} + T \) for each section.

5. The spacing of the supports along the structure which are capable of restraining the torsional moment determines the torque spans. If the piers are incapable of restraining the torsional moment, all of the torsion must be resisted at the abutments and the length of the bridge becomes the torque span. The results of step 4 are applied as a distributed load on the torque spans, which are assumed to have simple supports, and the beam reactions and distribution of shear forces are determined. The shear forces so determined are equal to the torsional moment at the various sections and the reactions are the concentrated torsional moments which the supports must resist. Where two torque spans frame into a single support, the support must restrain the torsional moment that is equal to the algebraic sum of the reactions from each span.

6. The longitudinal shears and bending moments are determined as if the bridge were on tangent. It has been found that the horizontal curvature has little influence on them under normal conditions (Ref. 57, 58).

An easily followed numerical example for the approximate method can be found in Reference 56.

### 5.4 Strength Analysis

Continuous structures, particularly those which have short spans and relatively large moving live loads, can experience moment reversals in unloaded spans which are adjacent to loaded spans. This phenomenon is well known and is not unique to prestressed concrete structures.

A related strength consideration in the design of prestressed concrete continuous structures which is not as well known is illustrated in Figs. 5.2 and 5.3. Moment capacity envelopes are shown in the two figures together with a moment diagram that is associated with a particular combination of design loads. In Fig. 5.2 the moment diagram is for the condition of secondary moment due to prestressing plus design dead and design live loads on all the spans. For the moment diagram shown in Fig. 5.3, the loading consists of secondary moment due to prestressing plus service
dead load on all the spans together with design live load on alternate spans. It will be seen that with design dead and design live load on all the spans, the simple span moment capacity for each span is 46,300 kip-ft. and the negative moment capacity of -35,400 kip-ft. at each support can be fully utilized. With service dead load on all spans together with design live load on alternate spans the simple span moment capacity for the loaded span is 34,200 kip-ft. In the latter case the negative moment capacity at the ends of the loaded spans is restricted to -23,300 kip-ft. due to negative moment capacity of -5,200 kip-ft. at midspan of the unloaded spans. It will be recognized that when the moment of -5,200 kip-ft. is attained at midspan of the unloaded spans, plastic hinges will form. Further increase in load will increase the positive moment at midspan of the loaded span until the value of 10,900 kip-ft. is obtained at which time an additional plastic hinge would form and the structure will no longer be stable.

This type of strength analysis should be made for all continuous prestressed concrete bridges. It is especially imperative that this type of analysis be made for segmental bridges which are erected in cantilever. This is because there is a tendency in this type of structure to have little, if any, negative moment capacity at midspan. It is frequently found to be necessary to place steel that is not required at service load in the upper

Fig. 5.2 Ultimate moment capacity diagram for a continuous prestressed concrete structure under design dead and live loads on all spans. (After J. Muller Ref. 67.)
Fig. 5.3 Ultimate moment capacity diagram for a continuous prestressed concrete structure under service dead load on all spans and design live load on alternate spans. (After J. Muller Ref. 67.)

Fig. 5.4 Elastomeric bearing pad used to explain European design practice.
portion of the section near midspans in order to provide the required ultimate moment capacity.

In girder bridges formed of precast prestressed concrete beams in combination with acast-in-place deck which contains continuity steel, positive moments can sometimes develop near the supports as a result of temperature, creep and shrinkage as well as due to live loads in remote spans. Provision of non-prestressed reinforcement is recommended in these areas as a means of controlling the effect of these moments.

5.5 Elastomeric Bearing Pads

It is interesting to compare the criteria contained in the AASHTO Specification for the design of elastomeric bearing pads to that which is used in Europe (Ref. 59). Such a comparison is made in Table 5.1. The obviously important differences are in the allowable compressive stress and the allowable translation.

The European design criteria can be explained by considering a pad having the dimensions shown in Fig. 5.4. The following notation is used:

\[
\begin{align*}
G &= \text{Shear modulus of the elastomer} \\
n &= \text{Numbers of layers} \\
a &= \text{Dimension parallel to the longitudinal axis of the bridge} \\
b &= \text{Dimension perpendicular to the longitudinal axis of the bridge} \\
t &= \text{Thickness of one layer of elastomer (between bonded steel plates)} \\
T &= \text{Total thickness of the bearing pad} \\
A &= \text{Area of the bearing pad} \\
S &= \text{Shape factor} = \frac{ab}{2t(a + b)} \\
\tau_0 &= \text{Shear stress due to vertical load} \\
\tau &= \text{Total shear stress} \\
\tau_h &= \text{Shear stress due to horizontal load} = \tau_s + \tau_d \\
\tau_s &= \text{Shear stress due to static horizontal load} \\
\tau_d &= \text{Shear stress due to dynamic horizontal load} \\
\tau_o &= \text{Shear stress due to rotation} \\
\sigma &= \text{Normal stress} \\
\delta_s &= \text{Horizontal deflection due to static loads} \\
\delta_d &= \text{Horizontal deflection due to dynamic loads} \\
H_s &= \text{Static horizontal load} \\
H_d &= \text{Dynamic horizontal load}
\end{align*}
\]

The basic consideration in the design of elastomeric bearing pads is the shear stress which exists in them due to the action of vertical load,
### Table 5.1

<table>
<thead>
<tr>
<th>ITEM</th>
<th>AASHTO CODE</th>
<th>FRENCH PRACTICE</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Allowable compressive stress</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D. L.</td>
<td>500 psi</td>
<td>none</td>
</tr>
<tr>
<td>T. L.</td>
<td>800 psi</td>
<td>2000 psi</td>
</tr>
<tr>
<td><strong>Allowable shear stress</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>none</td>
<td>570 psi</td>
</tr>
<tr>
<td>Vertical load (approx.)</td>
<td>none</td>
<td>340 psi</td>
</tr>
<tr>
<td>Horizontal displacement</td>
<td>none</td>
<td>60 psi</td>
</tr>
<tr>
<td>Rotation</td>
<td>none</td>
<td>170 psi</td>
</tr>
<tr>
<td><strong>Translation</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Long term (temp., shrinkage, creep)</td>
<td>none</td>
<td>0.5 T</td>
</tr>
<tr>
<td>Short term (temperature)</td>
<td>See below</td>
<td></td>
</tr>
<tr>
<td><strong>Rotation</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>For each layer $\alpha_i$</td>
<td>none</td>
<td>$\frac{t}{0.7T}^2$</td>
</tr>
<tr>
<td>For total $\alpha_T$</td>
<td>$\frac{t}{0.7T}^2$</td>
<td>$3n_0\frac{t}{0.7T}^2$</td>
</tr>
<tr>
<td><strong>Vertical Deflection</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T. L.</td>
<td>0.07 t</td>
<td>none</td>
</tr>
<tr>
<td><strong>Edge Clearance</strong></td>
<td>3 in.</td>
<td>2 in.</td>
</tr>
</tbody>
</table>

The AASHTO Specification limits the total of the positive and the negative movements to 0.5 T or less.

Horizontal load and rotation, it should be recognized that the application of a vertical load on a solid block of elastomer results in bulging of the sides of the block as shown in Fig. 5.5. The bulging is accompanied with shear stresses. A commonly used means of controlling the shear stresses (and reducing the bulging) is to provide pads composed of several thin
layers of elastomer bonded to thin steel plates separating the layers of elastomer. This is illustrated in Fig. 5.6.

The total shear stress in the elastomer, $\tau$, should be limited to 570 psi. Under the vertical load, the shear stress is:

$$\tau_p = \frac{3P}{2abS}$$  \hspace{1cm} (5.7)

If $\tau_p$ is limited to a maximum value of 340 psi, the maximum value of the vertical stress is 225 $S$ psi where $S$ is the shape factor defined above. The normal stress is generally limited by the concrete bearing stress. Because only nominal reinforcement is generally found in the concrete near the bearings, this stress is limited to 2000 psi and a minimum edge distance of 2 inches is normally used.

Shear stresses due to static and dynamic horizontal loads must be considered separately. Static loads include the effects of temperature concrete shrinkage and concrete creep. Dynamic loads include the effects of live loads such as braking forces, traction, wind and earthquake. The shear stress due to a static horizontal load is:

$$\tau_s = \frac{G\delta_s}{T} - \frac{H_s}{ab}$$  \hspace{1cm} (5.8)

and the maximum allowable value is $0.5G$ (60 psi) which limits the horizontal deflection due to static loads to 0.5 $T$.

The shear modulus of the elastomer, is twice as great for loads of short duration as it is for long-term loads. Hence the shear stress due to dynamic loads is:

$$\tau_d = \frac{2G\delta_d}{T} - \frac{H_d}{ab}$$  \hspace{1cm} (5.9)
which can be written
\[ \delta_d = \frac{H_d}{2G} \frac{T}{ab} \]  \hspace{1cm} (5.10)

The combined shear stress due to horizontal loads is:
\[ \tau_h = \tau_s + \tau_d = \frac{G}{T} \delta_s + \frac{H_d}{ab} \]  \hspace{1cm} (5.11)

If the deformation of the bearing pad under horizontal loads is restricted to an angle \( \gamma \) whose tangent is 0.7,
\[ \tan \gamma = \frac{\tau}{G} \]  \hspace{1cm} (5.12)

and one can write
\[ \frac{\delta_s}{T} + \frac{\delta_d}{T} = \frac{\delta_s}{T} + \frac{H_d}{2G ab} \leq 0.7 \]  \hspace{1cm} (5.13)

The shear stress due to rotation is equal to:
\[ \tau_o = \frac{G}{2} \left( \frac{a}{T} \right) \tan \alpha \]  \hspace{1cm} (5.14)

and the maximum value is limited to 1.5 G. Because for small angles, the tangent of the angle is equal to the angle, the rotation of the bearing is limited to
\[ \alpha_t \leq 3 \left( \frac{1}{a} \right)^2 \]  \hspace{1cm} (5.15)

for each layer of elastomer or
\[ \alpha_t \leq 3n \left( \frac{t}{a} \right)^2 \]  \hspace{1cm} (5.16)

for \( n \) layers.

If the vertical stress \( \sigma \) is less than 430 psi or if the total horizontal force equals or exceeds 20% of the vertical force, the bearing will slide unless it is bonded to the surface on which it bears.

Comparing the above design criteria to that shown from the AASHTO Specification in Table 5.1, will reveal our European counterparts are using compressive stresses that are over two and one-half times larger than those permitted by the AASHTO Specification. In addition, while the AASHTO Specification limits the sum of the positive and negative displacements of the bearing pads to one-half of the height of the bearing, the European criteria limits the maximum displacement under the combined effects of static and dynamic loads to 0.7 of the height of the pad.

It is interesting to note that elastomeric bearing pads were first used in
Fig. 5.7 Example of reactions at a pier which is provided with twin, spaced elastomeric bearings.

civil engineering works in Europe. The performance of the elastomeric bearing pads designed with the European criteria has been good and would indicate that it is adequately conservative.

The design criteria used in the U.S. results in bearing pads much larger than are required by the European criteria. This is best illustrated by an example. Consider the case shown in Fig. 5.7 in which plan and elevation views of a bridge are shown at a bent. The detail shown consists of two pads placed near the edges of the bent which is eight feet wide. Spaced as shown, the dead load reactions are 2200 kips on each pad and the live and impact loads result in an upward reaction of 905 kips on the side of the unloaded span and a downward reaction of 1335 kips on the side of the loaded span. The result is a maximum reaction of 3535 kips on one pad and a minimum reaction of 1295 kips on the other. The AASHTO Specification would require pads having an area of 30.7 square feet while the European method would require pads having an area of 12.3 square feet.

An increase in pad area requires a greater height for the pad if the pad must accommodate a particular movement and if the force which causes the pad deformation is to be unaffected by the increased area. From this it will be seen that the low allowable stresses due to vertical loads result in very considerably larger pads being required with the AASHTO
Specification as compared to that required with the European design criteria.

If properly detailed, the use of elastomeric bearing pads can allow the bridge designer to control the forces a support must withstand from a given translation. Increasing the height of the pad will reduce the horizontal force transferred to the support. A decrease in the pad height increases the force and the effective stiffness of the substructure. By using spaced bearing pads as shown in Fig. 5.7, the designer can transfer moment due to vertical loads from superstructure to substructure and yet control the moment induced in the bent as a result of translations. In effect, semi-fixed and semi-expansion bearings can be achieved by proper proportioning. Bearings of other types, such as conventional metal bearings and elastomeric “pot” bearings are not as versatile in forming semi-fixed or semi-expansion bearings.

The spaced bearing detail shown in Fig. 5.8 was used on the Oléron Bridge in France. The use of the spaced bearings resulted in a distribution of moments due to the design live load of four kips per linear foot as shown in the left side of Fig. 5.9. If a single row of bearings rather than two rows were used, the distribution of moments would have been as shown in the right side of Fig. 5.9. From this it will be seen the provi-

Fig. 5.8 Superstructure-substructure detail at a typical interior pier of the Oléron Bridge in France.
Consider the five-span bridge shown in Fig. 5.10. The structure traverses a deep valley which results in the two interior bents being relatively high while the outermost bents are relatively low. The prestressed concrete superstructure will shorten with the passing of time due to effects of shrinkage and creep. In addition, its length will vary with temperature changes. The changes in length require the provision of one or more expansion joints in the superstructure. The number and location of the expansion joints has significant influence on the moments which will exist in the structure as a result of length changes as well as due to lateral loads. Normally one prefers to use the minimum number of expansion joints possible because joints require maintenance during the life of the structure and may be costly to provide.

One joint layout that could be feasible for the bridge of Fig. 5.10, would consist of expansion joints at the abutments only. With this layout, if the superstructure were fixed to the bents, the changes of length of the superstructure would induce moments in the bents. The moments in the
taller central bents are frequently not critical under such circumstances whereas they frequently are critical for the shorter and stiffer outer bents. Provision of expansion bearings between the top of the shorter bents and the superstructure would eliminate the moments in these bents but would require the higher bents to resist all longitudinal lateral loads.

Another and perhaps better method of solving the problem is to place spaced elastomeric pads, as shown in Fig. 5.7, between the tops of the shorter bents and the superstructure. By carefully selecting the size and shape of the bearings, the designer can obtain almost any effective stiffness of the combined bearing and bent and still transmit moment due to vertical loads to the shorter bents. This is discussed further in the following section.

5.6 Substructure Considerations

The piers or bents of a bridge constitute an important part of the bridge from functional, cost and appearance standpoints. Too frequently the shapes of piers or bents are selected without each of these factors being given the consideration it deserves. Each of these is treated briefly in this section.

The primary function of a pier is to transmit the vertical dead and live loads of the superstructure to the ground through the foundation system. A secondary function which piers sometimes perform consists of transmitting lateral loads, both transverse and parallel from the longitudinal axis of the bridge, to the ground. Another secondary function piers sometimes perform is to resist moments induced in them either as a result of vertical dead and live loads acting upon the superstructure or due to dimensional...
changes (temperature, creep and shrinkage) the superstructure may undergo. These latter functions are secondary in nature but not necessarily in magnitude. Many times the bridge designer must take special precautions, such as using elastomeric bearing pads as was described in Section 5.5, to eliminate or reduce what could otherwise be excessive stresses.

Piers of various shapes and cross sections have been successfully used. This includes simple solid round or rectangular columns as shown in Fig. 5.11, simple hollow prismatic shafts as shown in Fig. 5.12, twin walls which may or may not be vertical as shown in Fig. 5.13 and flared shapes such as is shown in Fig. 5.14, to show only a few examples. Each type has certain characteristics and advantages which can be described as follows:

**Solid piers.** Solid round and rectangular columns have been extensively used in bridge construction and most particularly in bridges spanning roads, highways and railroads. Their use has generally (but not always) been confined to structures of moderate span and height. Primary advantages of these shapes include relatively low cost, unless they are large in cross section, and acceptable appearance under most conditions.

---

Fig. 5.11 Solid piers which are round in cross section. (Courtesy California Department of Transportation).
Hollow prismatic piers. Hollow prismatic piers are structurally efficient, are relatively low in cost and generally find application on structures of moderate to long spans. They can be used to best advantage for piers that are of medium to great height. Piers of this type are generally of good but simple appearance.

When provided with spaced elastomeric bearing pads at their tops, as described in Section 5.5, hollow prismatic piers of adequate proportions are especially functional in structures that are erected in cantilever. This is partially because their capability of resisting bending moments negates the
need of temporary struts during erection. In addition, as explained in Section 6.7, the provision of elastomeric bearing pads between superstructure and substructure provides a means of adjusting the structure during construction.

The structural analysis of piers which incorporate elastomeric bearing pads has been described by Mathivat (Ref. 60) who has shown the relationships between the rotation $\theta$, horizontal translation $\delta$, vertical deflection $v$ and the moment $M$, shear $Q$ and vertical load $N$ applied at the top of the pier can be written as follows:

$$E\theta = AM + BQ \quad (5.17)$$

$$E\delta = BM + CQ \quad (5.18)$$

$$Ev = KN \quad (5.19)$$

in which for a pier alone (no elastomeric bearing pads) the elastic constants for rotation (A due to moment and B due to shear), displacement due to shear (C) and vertical deflection (K)* are as follows:

*Note: B is both the coefficient for rotation due to shear and the coefficient for displacement due to moment. This can be explained by Maxwell’s law.
ADDITIONAL DESIGN CONSIDERATIONS

\[ A_x = \int_0^h \frac{dx}{l} \]  \hspace{1cm} (5.20)

\[ B_p = \int_0^h \frac{xdx}{l} \]  \hspace{1cm} (5.21)

\[ C_p = \int_0^f \frac{x'dx}{d} \]  \hspace{1cm} (5.22)

\[ K_p = \int_0^h \frac{dx}{A} \]  \hspace{1cm} (5.23)

Fig. 5.14 A flared pier.
\( G \) = shear modulus of elastomer  
\( p \) = number of pads per row  
\( n \) = number of laminations in each pad  
\( S = a \times b \) (area of each pad)  
\( t \) = thickness of each lamination

Fig. 5.15 Pier with spaced elastomeric bearing pads.
For a prismatic pier, Eqs. 5.20 through 5.23 become:

\[ A_{\varepsilon} = \frac{h}{I} \]  
\[ B_{p} = \frac{h^2}{2I} \]  
\[ C_{p} = \frac{h^3}{3I} \]  
\[ K_{p} = \frac{h}{A} \]

in which I and A are the moment of inertia and area of the pier cross section respectively and h is the height of the pier. If spaced elastomeric pads, having the dimensions shown in Fig. 5.15 are placed on top of a pier the values of the elastic constants A, B, C and K in Eqs. 5.17, 5.18 and 5.19 must include the effect of the bearing pads. These effects are:

\[ A_{e} = c \cdot \frac{2n}{pd^2} \cdot \frac{t^3}{GSw^2} \]  
\[ B_{e} = 0 \]  
\[ C_{e} = \frac{n}{2p} \cdot \frac{t}{GS} \]  
\[ K_{e} = c \cdot \frac{nt^3}{GSa^2} \]

in which all terms except c are defined in Fig. 5.15.

<table>
<thead>
<tr>
<th>b/a</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.75</th>
<th>0.8</th>
<th>0.9</th>
<th>1.0</th>
<th>1.2</th>
<th>1.4</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>10</th>
<th>( \infty )</th>
</tr>
</thead>
<tbody>
<tr>
<td>c</td>
<td>5.83</td>
<td>4.44</td>
<td>3.59</td>
<td>3.28</td>
<td>3.03</td>
<td>2.65</td>
<td>2.37</td>
<td>2.01</td>
<td>1.78</td>
<td>1.70</td>
<td>1.46</td>
<td>1.27</td>
<td>1.18</td>
<td>1.15</td>
<td>1.07</td>
</tr>
</tbody>
</table>

The coefficient c has the value shown in Table 5.2. It can be shown that for the combined effects of the pier and elastomeric pads, the elastic coefficients are as follows:
\[
A = A_0 + A_1 \quad E \quad (5.32)
\]
\[
B = B_p \quad (5.33)
\]
\[
C = C_p + C_e \quad E \quad (5.34)
\]
\[
K = K_p + K_e \quad E \quad (5.35)
\]

Using the above, it can be shown that for a pier which has no horizontal deflection \((\delta = 0)\) the moment at the base, \(M_b\) can be determined from

\[
M_b = \left(1 - \frac{B_p h}{C_p + C_e E}\right) M \quad (5.36)
\]
in which \(M\) is the moment at the top of the pier. The term

\[
\frac{B_p h}{C_p + C_e E} \quad (5.37)
\]
is the carry-over factor. Consideration of Eq. 5.37 will reveal that for the case of a prismatic pier without elastomeric bearing pads the value of Eq. 5.37 is -0.50 (the familiar carry-over factor). On the other hand if

\[
C_e E = h B_p - C_p \quad (5.38)
\]

the carry-over factor is equal to zero and there is no moment at the bottom of the pier. Finally, as the value of \(C_e\) approaches infinity, the carry-over factor approaches unity. These are illustrated in Fig. 5.15.

Consideration of these fundamental elastic principles will reveal the important advantages that can be realized through the use of spaced elastomeric bearings. Their use permits the engineer to control the elasticity of the substructure and transfer bending moments due to vertical loads between superstructure and substructure. In addition, the effects of length changes can be controlled.

**Piers having twin walls.** Rather than employing spaced elastomeric bearing pads as shown in Fig. 5.7 to control the effects of volume changes in bridges having piers of small to medium height, piers having twin vertical or inclined walls as shown in Figs. 5.13 and 5.16 have been used. This pier configuration behaves in a manner similar to that of a hollow prismatic pier with spaced elastomeric bearing pads (Ref. 60). The walls may be fixed to the superstructure as well as to the solid base of the pier. Piers have also been made with concrete hinges at one or both ends of the walls. Longitudinal length changes such as those caused by temperature variations, shrin-
kage and creep cause bending of the wall elements which in turn results in very little **resistance** to these movements. On the other hand, under the action of vertical loads, the structure containing twin wall piers behaves as a frame and moment is transferred from the superstructure to the substructure. Unbalanced superstructure moments can result in uplift forces being applied to one of the two walls of a pier and if the uplift force exceeds the dead load in the pier, the wall will act as a tension member and must be reinforced accordingly (Ref. 61). When converging walls are used, a portion of the longitudinal deformation of the superstructure is resisted by truss action as contrasted to bending alone in the case of vertical walls. If converging walls which have hinges at both their tops and bottoms are used on slopes that result in their axes converging at the bottom of the foundation, as shown in Fig. 5.16, the foundation is subjected to vertical load alone. On the other hand, if the axes of the walls converge at a point above or below the bottom of the foundation, the foundation must resist a bending moment in addition to the vertical load. Needless to say, thin pier walls of this type must be investigated for elastic stability. This type of pier is relatively economical in segmental construction because they are stable during construction with very little in the way of temporary bracing. The appearance of piers of this type can be quite acceptable as shown in Figs. 5.17 and 5.18. A detailed study of piers of this type is contained in Ref. 60.
Fig. 5.17  Choisy-Le-Roi Bridge, France (Courtesy of Société Technique pour L'Utilization de la Précontrainte).
Flared piers. Piers which are provided with flared upper portions, such as shown in Figs. 5.19 and 5.20 have been used on a number of bridges. Piers of this shape are not normally used on structures which are low (short piers). The pier shafts may be solid but they are usually hollow because the shafts themselves generally are quite large in cross sections. The piers may have openings, recessed areas or “special architectural details” which are intended to enhance their appearance. The flared outer edges of the piers near their tops as well as the openings, recesses and other architectural details often complicate the structural details of the piers. This in turn results in structural inefficiency, imposition of specific construction methods that must be used in the pier and superstructure and additional cost. The flares at the tops of piers are particularly a problem in precast segmental construction when the design is predicated upon a fixed connection between superstructure and substructure. The construction details required to effect such a connection are complicated and costly. The use of piers with flared tops is less objectionable with cast-in-place segmental construction. Many persons feel piers of these types are attractive and...
Fig. 5.19  Mission Valley Viaduct, San Diego, California. (Courtesy of State Department of Transportation).

Fig. 5.20  Pine Valley Creek Bridge, California. (Courtesy of State Department of Transportation).
hence the additional cost they entail is worthwhile. Others feel the basic shape of the pier should be chosen to satisfy its functional (structural) requirements with due consideration to cost and appearance.

5.7 Seismic Forces

The earthquake design provisions of the AASHTO Specification are contained in the Interim Specifications issued in 1975 (Ref. 62). These provisions are an outgrowth of the significant amount of bridge damage that occurred in the 1971 San Fernando earthquake. It was this event that prompted the California Department of Transportation to re-evaluate the seismic design procedures then being used as well as to re-examine details of construction which were being used as standard details (Ref. 64). This study resulted in the seismic design criteria that are currently used in California and which are contained in the AASHTO Interim Specifications (Ref. 63). The present criteria requires consideration of the relationship of the site to active faults, soil-structure interaction, and the dynamic response characteristics of the bridge as a whole. The criteria is among the more sophisticated in use today.
6 Construction Considerations

6.1 Introduction

Many factors affect the relative cost of the various modes of bridge construction. The relative importance of the factors varies throughout the country (and the world) and hence escape generalization. For this reason no attempt has been made to give specific cost data nor even relative cost data for the different bridge types. Construction considerations vary in importance between the bridge types and an attempt is made to discuss the basic considerations of falsework, formwork, concrete finishing and camber control.

Girder and box-girder bridges have had wide use in the United States for a number of years and it is doubtful if major innovations will evolve with these modes of construction. Segmental bridge construction has not as yet been widely used in the United States, in spite of the fact that several hundred segmental bridges have been constructed in other parts of the world. It is expected that many innovations will be made in this country when the use of this method becomes more common.
6.2 Falsework

Bridges which are cast-in-place on falsework are normally of the type that are classified as T-beam or box-girder bridges. Bridges have, however, been constructed on falsework, even though they have been designed to be erected segmentally in cantilever. This is illustrated in Fig. 6.1. This permits advantage to be taken of the economies of both methods. The falsework normally consists of timber or steel beams supported by timber or steel columns. A typical bent which utilizes steel beams supported on timber posts is shown in Fig. 6.2(a). Typical details for falsework composed of steel beams supported by steel towers are shown in Fig. 6.2(b).

In recent years the trend has been toward greater use of steel falsework and less use of timber falsework. This has partially been the result of improved modular systems of steel falsework becoming commercially available and partially due to timber becoming more expensive than it was in previous years.

The costs related to falsework can represent a major portion of the total cost of a bridge. The falsework related costs, which are in addition to the purchase or rental of the basic falsework materials, include the following:

1. Engineering costs for design and inspection of the falsework.
2. Preparation of the foundation. This might consist of preparing soil

Fig. 6.1 Falsework for use with the segmental erection technique.
Fig. 6.2 (a) Typical timber falsework

Fig. 6.2 (b) Falsework incorporating steel beams and steel towers
and setting wood mudsills, concrete pads or perhaps driving temporary piling. In many cases wood mudsills can be used on level, fine graded natural soil or thin blankets of sand placed over natural soil. In some locations the natural soil has low strength and it is removed and replaced with well-compacted soil that is adequate for the loads that are to be imposed by the mudsills or concrete pads. Temporary piling are used in cases where it is not feasible to use mudsills.

3. Fabrication or assembly and erection of the falsework bents, together with adequate longitudinal and transverse bracing.

4. Setting the beams, whether of wood or steel, which span between falsework bents.

5. Fabricating and setting camber strips on the beams, if required by anticipated beam deflection. (This is often done before the beams are erected.)

6. Adjusting the assembled falsework to grade. (Generally done after the formwork has been partially assembled on the falsework.)

7. After the concrete has been placed, cured and stripped of the formwork, lowering the falsework, removal of the beams and striking the falsework bents.

8. Removing the mudsills or temporary piles and restoring the site. This may include reconstructing abutment slopes in areas where falsework bents must be set on the slopes.

Falsework which must be erected over an existing highway or railroad is considerably more difficult to erect if traffic must be maintained. It also involves considerably more risk to the contractor. Furthermore, some agencies require especially conservative designs and details for falsework which is to be erected over a traveled way. For these reasons, falsework costs are greater under such circumstances.

Bodies of water, mudflats and deep valleys can add materially to the problems associated with the construction of falsework. Costs are also affected by obstacles of these types.

Falsework, being a very temporary structure, is normally constructed using materials, connections and details which would not be permitted or even considered in a permanent structure. Because of this, falsework is more subject to failure, whether originating by accident, or inadequacy of materials, connections and details, than are permanent structures. This fact is reflected in the cost of structures erected on falsework.

Bridge superstructures which utilize precast girders or segmental gir- ders which are erected in cantilever, frequently do not require any falsework. As was explained in Section 4.2, falsework at the piers of segmental bridges is sometimes needed to accommodate the unbalanced
moments which exist during erection. Falsework of this type is quite special and requires careful consideration in its design in order to achieve adequate strength together with details which facilitate its erection and removal.

6.3 Formwork

The cost of formwork, per unit of area, varies significantly between the various modes of prestressed concrete bridge construction. Surprisingly, the unit price (cost per square foot of bridge) of bridge types which require more *jobsite constructed formwork* may be lower than bridges of other types. The phenomenon is attributable to the quality of the *formwork* required together with the degree of prefabrication that is possible and the working conditions available for the assembly of *non-prefabricated* forms.

Box-girder bridges of normal configuration have more formwork contact area per unit area of bridge deck than T-beam or precast prestressed concrete girder bridges. On the other hand, the formwork for box-girder bridges is relatively easy to construct and much of it does not need to be of high quality material nor fabricated with high precision. The soffit formwork generally consists of 4 x 4 wood sleepers placed transversely on top of the falsework beam camber strips over which 5/8-inch plywood is nailed (nominally). The soffit formwork is easily constructed because there is little (if any) cutting and fitting of material and the work can all be done “down-hand”. High quality formwork is required for the deck slab overhang and the exterior surface of the fascia girder. The surfaces of the interior webs do not require high quality formwork because they are hidden from view in the completed structure. Formwork for the interior webs is usually reasonably well made with material of good quality because they are frequently reused many times. The formwork for the upper slab of the interior cells of box-girder bridges is normally not stripped after the slab has cured and for this reason it is frequently referred to as the “lost-deck-form”. Because the lost-deck-form is used only once and because the surface it forms is not subject to view in the completed structure, material of very low quality together with only adequate workmanship is normally used in its construction. Stripping of all the formwork (webs, slab overhangs and soffit) is relatively easy and little must be done “over-head”. This combination of requirements results in relatively inexpensive formwork and even though the quantity which is required is relatively large, the overall formwork cost is relatively low.
The quantity of formwork required for cast-in-place T-beam girder bridges is frequently less per unit of deck area than is required for box-girder bridges. The soffit formwork is relatively expensive because it must be made to a specific width, must be vertically cambered and made horizontally straight. The side forms for the diaphragms and all the webs, both interior and exterior, must be well constructed of high quality material in order to obtain straight smooth lines and surfaces. The same is true of the deck formwork. The forms do lend themselves to a reasonable amount of prefabrication. Stripping T-beam forms is difficult because at least the deck form must be done “over-head”. All surfaces of a T-beam bridge are visible in the completed structure.

Although the amount of field assembled formwork required for bridges which incorporate precast prestressed concrete girders is relatively small, it is also rather difficult and costly to construct. The interior diaphragms are normally relatively thin and the girders must be tied together when the diaphragm concrete is placed in order to prevent the pressure of the plastic concrete from deflecting the girders laterally. The interior diaphragm forms must be field cut and adjusted to fit the girders as built and as erected and hence do not lend themselves to complete prefabrication. Little exists in the way of safe or convenient working space for the construction of the diaphragms. They are normally constructed (and stripped) before the deck is formed. The deck forms are frequently supported from adjustable metal brackets as shown in Fig. 6.3. Because the space between the girders is never exactly straight nor with the precise width shown on the construction drawings, the deck

Fig. 6.3 Forms for the deck of a girder bridge incorporating precast girders.
forms must be field cut to fit the actual space between girders. The reinforcing steel stirrups which project from the tops of the girders hinder the construction of the deck form. Stripping the deck forms is always over-head work and frequently must be done under adverse and potentially dangerous conditions. Formwork quality must be high because the surfaces are visible in the completed structure and leakage during placing of the concrete must be minimized as is explained below in Section 6.4. These conditions result in relatively costly formwork.

The traveling forms, or simply the travelers, which are used in cast-in-place segmental bridge construction perform two functions. These are the forming of the desired shape of the concrete segments and holding the forms, falsework, concrete and other materials in the desired position, cantilevered from the previously constructed work. These functions may be done by two independent portions of the travelers or may be done by a single device which performs both functions. The formwork function of the travelers is normally done with steel forms although particularly in the vicinity of the piers, where transverse dia-
phragms must generally be provided, the internal formwork may be done with wood. The structural cantilevering function of the travelers is normally done with steel trusses which incorporate hydraulic equipment for advancing and adjustment for line, grade and superelevation. When the traveler consists of a single device, the side forms act as structural girder cantilevered from the previously constructed work. Needless to say, the cost of the travelers is great and must be amortized over a number of reuses. When used in erecting bridges over deep valleys, dismantling the traveler after completing one cantilever and moving it to another bent to begin a new cantilever can be a somewhat difficult and time consuming task. A typical traveler is shown in Fig. 6.4.

The equipment and method of casting precast segments for segmental bridge construction has gone through three generations. Innovations in these methods will certainly come about in the future. The method first used is shown in Fig. 6.5, and is sometimes referred to as the “long-line method”. In this, the simplest of the methods, the segments are cast on a fixed concrete soffit form, each segment being cast against the adjacent one, with the internal and external forms moving down the soffit form as the construction progresses. The procedure of casting one segment against a previously cast segment is referred to as match-casting and the segments are referred to as match-cast segments. The forms are generally of steel but they can be made of wood. The segments are removed to the bridge site and reassembled in the same relative position in which they were cast. A variation on this technique which was employed on the bridge between the European Continent and the Island of Oléron in France is shown in Fig. 6.6. This variation permits the start

![Fig. 6.5 Principle of the precasting technique for segmental box girder construction utilizing a long soffit form.](image-url)
Fig. 6.6 Principle utilized in precasting the segments for the bridge between the continent and the Island of Oleron.
HIGHWAY BRIDGE SUPERSTRUCTURES

PIER SEGMENT
(CAST LEVEL AND PLUMB)

END FORM

ADJUSTABLE CART

PERMANENT BULKHEAD

(a) FIRST STEP - CAST PIER SEGMENT

/ - MATCH CAST JOINT

FIRST SPAN SEGMENT,

PIER SEGMENT
(ADJUST TO PROPER ORIENTATION WITH RESPECT TO SEGMENT 2

(b) SECOND STEP - CAST FIRST SPAN SEGMENT

(c) SUBSEQUENT STEPS - CAST REMAINING SPAN SEGMENTS

Fig. 6.7 Casting cell for precasting segments.
of precasting the segments for a second cantilever before the completion of the first. This is accomplished by the provision of a means of removing the pier segment before the first cantilever has been completed. The cost of the concrete soffit forms is relatively great and the method is only considered practical on bridges which have constant horizontal and vertical curvature (if any at all). The second generation of precasting segments involves the use of a casting machine or casting cell, and can accommodate variable horizontal and vertical curvature as well as variable superelevation. The method is sometimes referred to as the “short-line method”. The principle of the casting cell is illustrated in Fig. 6.7. It consists of adjustable internal and external forms which are always used in a level and plumb position. The previously cast segment is set next to the form in the proper relative position with respect to the segment being made to give the required relative position in the final structure. The new segment is cast against the older segment insuring perfect fit when later it is being reassembled. The cost of the casting cells used in the second method varies between wide limits but is high and necessitates amortization over a number of reuses. The third precasting technique consists of match casting the segments in a vertical position, the new segment being cast on top of the previously cast segment as shown in Fig. 6.8. The method can be thought of as the “vertical short-line method”. The basic steel form system is somewhat like that which

![Diagram of precasting segments](image)

Fig. 6.6 The technique used in match casting segments in a vertical position.
Fig. 6.9 Floating crane erecting a precast girder. (Courtesy of Freyssinet Company, Inc. New York).

Fig. 6.10 Launcher for erecting precast girders.
is used in the manufacture of concrete pipe. The third method has been used exclusively on the production of a large number of freeway overcrossing structures in southeastern France and is particularly useful in producing segments which have small depths.

6.4 Concrete Finishes

Specifications for the finishing of concrete bridges vary between the various jurisdictions. Hence, the statements made herein in this regard, which are based upon the standard specifications used in highway bridge construction in California (Ref. 65), may not be universally applicable.

In general two types of finishes are required. These are termed ordinary surface finish and Class 1 surface finish. Ordinary surface finish consists of filling all holes or depressions in the surface of the concrete, repairing rock pockets and repairing honeycomb as well as removing fins, stains and discolorations which are visible from traveled ways. Ordinary surface finish is the final finish for most surfaces on a bridge and is a preparatory finish to surfaces required to have a Class 1 surface finish. The Class 1 surface finish includes eliminating any bulges or depressions or other imperfections which exist due to the forms not being of high quality. The removal of bulges requires grinding the surface after which they must be “sacked” to give a uniform appearance.

The Class 1 surface finish is required for the deck overhang and exterior faces of the exterior beams or webs of all bridge types. Hence the cost of this portion of the work is roughly the same for bridges of all types.

The inside surfaces of box-girder bridges do not require fin, stain or discoloration removal. Ordinary finish is required for the deck, and beam soffits as well as the sides of the beams in girder bridges. Only the bottom deck surface of a box-girder bridge requires the ordinary finish.

In the construction of girder bridges in which the deck is not placed monolithically with the girders, care must be exercised during the placing of the deck concrete to insure that concrete does not leak through the deck-form-to-girder joint and become deposited upon the side of the girder. If such deposits do occur, they should be washed off with fresh water before they harden. Otherwise they increase the effort (and cost) required to properly finish the concrete surfaces.

As a result of the above, concrete finishing costs are usually less for a box-girder bridge than for girder bridges. Segmental bridges should have finishing costs comparable to those of a box-girder bridge.
6.5 Erection of Precast Girders

The erection of precast girders to be incorporated in structures crossing over an existing roadway is normally accomplished with the familiar truck crane. For very large girders or for larger than usual reaches, crawler cranes may be used. Girders which are to be used in bridges which cross bodies of calm water are frequently erected with floating cranes as shown in Fig. 6.9. The erection of precast girders under these circumstances is straightforward and does not require further explanation.

For precast girders to be used in bridges crossing deep valleys or bodies of water subject to sudden changes in conditions (such as a river in the tropics) or those having strong and variable current, the use of a girder launcher may be the best approach. A girder launcher is shown in Fig. 6.10. In using a girder launcher the launcher is first cantilevered over the first span until it can be supported near the first abutment and the first pier. The girders for the first span are then picked up by the launcher and moved into position between the first abutment and the first pier. Some launchers are equipped to move the girders transversely while others are not. When the launchers are not equipped to move the girders transversely, they are moved horizontally by placing them on greased steel plates and jacking them laterally. Before a launcher can be advanced, it is frequently necessary to install the permanent or temporary diaphragms between the erected girders in the first span, placing a lateral bracing system in the plane of the top flanges of the girders to prevent buckling of the compression flanges and finally placing a temporary wood deck on the first span to provide a surface over which the launcher and girders can be moved without interference from the bracing system and reinforcing steel dowels which usually protrude from the tops of the erected girders. The launcher is then advanced and the girders in the second span are erected and the procedure is repeated until the bridge is erected from one abutment to the other. It is generally not possible from a structural viewpoint, nor feasible from a construction scheduling viewpoint, to cast the permanent deck slab in place and thereby avoid the temporary top flange bracing and wood deck.

6.6 Erection of Precast Segments

Precast segments have been erected using a variety of methods. In selecting an erection method one must consider three major factors; the cost of the equipment required in the method, the rate at which the equipment will erect segments, and the constraints imposed by the site. Virtually all large structures have been erected with a gantry designed ex-
pressly for the particular structure. The specially designed gantries are the preferred method because they are capable of erecting the segments more rapidly than any other equipment and they can be used regardless of site conditions. The relatively high first cost of the gantries preclude their use on less important structures.

The simplest method of erecting precast segments, and the one which involves the least capital investment, is to employ a conventional truck or crawler crane. Cranes of these types are readily available in virtually all parts of the country and most bridge contractors are familiar with their use. There are, however, several reasons why the use of conventional cranes may not be feasible on a particular structure. The load capacity of a crane is a function of the size of the crane, the length of boom required to reach the required height and the load radius (measured from the axis about which the crane cab revolves). The capacity of a crane reduces rapidly as the height of lift and load radius increases. Some combinations of segment weight, pier height and minimum possible load radius preclude the use of even the largest conventional crane. It is normally not feasible to “walk” a crane which is under a large load. Hence, the crane must be positioned directly below the final position of

Fig. 6.11 Erection of the Choisy-Le-Roi Bridge with floating cranes. (Courtesy of Société pour L’Utilisation de la Précontrainte, Paris, France).
the erected segment and the segment must be transported to the crane. Site restrictions may preclude such access. It should also be recognized that conventional cranes are relatively flexible, particularly with the longer booms, and because of oscillations and the difficulty of raising or lowering a load slowly in minute increments, segment erection with conventional cranes is generally slow. Segment erection requires greater precision in positioning the load being erected than is required in the erection of precast beams.

Bridges which traverse bodies of water can often be erected with floating cranes as shown in Fig. 6.11. Floating cranes of very large capacity exist, and very long booms can be used with them where necessary. The effects of wind, current, waves, tide and wakes of moving vessels can result in conditions which render the use of floating cranes difficult and slow, if not impossible. One solution for this problem is to use the floating crane to erect the segment on a secondary piece of erection equipment that is supported on the bridge superstructure, the secondary equipment being used to adjust the segment into final position. Secondary equipment of this type was used in the bridge shown in Fig. 6.11. It would be practical for use with conventional truck or crawler cranes.

When it is possible to transport the segments to a position directly below their final position in the completed structure, equipment of the type shown in Fig. 6.12 can be used to erect the segments. This type of equipment is considerably less costly than a gantry but is also much slower.

An erection gantry was used for the first time in precast segmental bridge construction in the Oliron Bridge. This gantry is shown in Fig. 6.13. There have been a number of bridges constructed by gantry since the construction of the Oléron Bridge. The design of the gantries has gone through a constant evolution and their efficiency has been progressively increased. As an example, the gantries used in the construction of the Rio-Niteroi Bridge in Brazil were able to erect a typical span of 262 feet every seven days. The Rio-Niteroi Bridge is shown during construction in Fig. 6.14. Gantries can be used to erect segments which are transported to the gantry over the top of the previously erected superstructure as was the case with Oléron or which are transported to a point below the gantry as was the case with the Rio-Niteroi. In instances where the pier or pier-superstructure connection is not sufficiently strong to resist the unbalanced moment induced by the cantilever erection technique, the magnitude of the unbalanced moment can be minimized by erecting two segments simultaneously as shown in Fig. 6.15 or its effect can be nullified by using the gantry itself to brace the structure during erection as shown in Fig. 6.16. Erection gantries are
Fig. 6.12 Erection procedure with segments transported to a position directly below their final position.

Fig. 6.13 Gantry erecting precast segments on the Oléron Bridge. (Courtesy of Société pour L'Utilisation de la Précontrainte, Paris, France).
very expensive but no other erection method is as versatile or as fast.

The erection of bridge superstructures is normally done by starting at
one end of the structure and working toward the other. There are many
reasons for this. In the case of erecting precast segmental bridges with a
gantry, it is especially important to erect the superstructure in this man-
ner. One of the reasons for this involves the stability of the structure due
to its own dead load and is illustrated in Fig. 6.17. It will be seen in Fig.
6.17 that the structure is rendered stable after each new cantilevered section has been erected. The stability is achieved by connecting the newly erected cantilever to the previously erected portion of the superstructure. In the case of the first cantilever, the stability is achieved by completing the first span. If all the cantilevers were to be erected before continuity was established between the cantilevered portions of the bridge, the construction cost and risk would be greater. This

1. ERECT FIRST CANTILEVERS

2. COMPLETE END SPAN

3. ERECT SECOND CANTILEVERS AND EFFECT CONTINUITY

4. COMPLETE END SPAN

Fig. 6.17 Recommended erection sequence with **gantry-erected** precast segments.
procedure is illustrated in Fig. 6.18. The procedure shown in Fig. 6.18 involves a more elaborate and costly gantry than is required when continuity is established as the erection progresses. This is because the gantry must advance without imposing any load on the cantilevers.

An erection method which utilizes a temporary cable-stayed technique was used in the summer of 1974 for the first time on the Rombas Bridge in France (Ref. 66). This method, which is illustrated in Fig. 6.19, permits the bridge to be erected from the top with the segments being transported to the point of erection over the top of the previously erected superstructure. It has two major disadvantages; (1) the first end span must be erected on falsework or with some other special method and, (2) the bridge substructure is subjected to larger dead loads during erection than in the completed structure and hence must be designed therefor. The cost of the equipment with this erection method is modest and the speed of erection is not as fast as with a gantry. Considerable engineering effort is required to determine the adjustments required in the loads in the temporary cables during the progression of the bridge erection.

A large number of small grade separation structures were constructed using precast segments on the Rhône-Alps Motorway in France. The structures have central spans up to 100 feet long. The erection of the segments was accomplished with conventional truck cranes together with a minimum amount of steel falsework and a clever temporary prestressing scheme. The erection procedure is illustrated in Fig. 6.20 in
which it will be seen that the first segment is erected on top of four 25-ton hydraulic jacks after which the second and third segments are connected to the first with temporary prestressing. The temporary prestressing which connects the first three segments includes top and bottom prestressing. Four 100-ton jacks, which are equipped with low-friction slide plates, are then installed to support the segments and the four 25-ton jacks are removed. The segment erection is continued using the cantilever technique with the prestressing being provided by temporary bar tendons attached to the tops of the segments. After each half of the segments have been erected, the assemblies of segments are pushed together thereby eliminating the need of a cast-in-place closure joint. The erection procedure is completed by installing the permanent prestressing tendons (in two stages), erecting the abutment segments, and removing the temporary prestressing tendons as well as the falsework. Superstructures can be erected in as little as four days with this technique.

6.7 Camber Control

Prestressed concrete structures deform under the effect of short duration loads in much the same manner as structures composed of other materials. Because concrete is a material which is subject to time-dependent deformations as a result of creep and shrinkage, and because different prestressing steels exhibit relaxation to different degrees, the long-term deformation and deflection of prestressed concrete members under dead load alone is the result of a complex interaction of several
Fig. 6.20  Erection technique employed on the Rhône-Alps Motorway overcrossing structures.
phenomena. In order to achieve desirable results during the service life of a structure, the structure must be constructed with the effects of time-dependent deformation being taken into account. Considerable information is available in the literature regarding the computation of long-term deformation of prestressed concrete members and hence the fundamentals of this subject are not repeated here.

The effect of time-dependent deformations on the deflection of cast-in-place prestressed concrete structures is taken into account by building camber in the formwork used to construct the structures. This is a well-known procedure which does not require further explanation. In a similar manner provision is made for camber in structures utilizing precast girders through the use of a haunch of variable depth over the girders as was described in Section 2.7. No further consideration of this subject is needed.

The estimation of the deflection of bridges which are constructed segmentally is more complex than for more conventional prestressed concrete structures. The additional complexity partially emanates from the fact that the age of the concrete varies from segment to segment and hence the elastic modulus is variable along the length of a segmentally constructed member. In addition, because the tendons are prestressed at different time intervals, the various tendons are relaxing on a different time basis. When the cantilever erection technique is employed, the deflections must be computed for each step in the construction. The most logical approach to analyzing a structure with so many time-dependent variables is to use a numerical integration procedure in which the many variables can be treated as independent time functions.

In the case of cast-in-place segmental bridges, from four to seven days will normally be required for a cycle in the fabrication of segments of usual size. For larger segments greater periods are required. Because a cast-in-place cantilever may have from six to twenty segments, it will be realized that the difference in age between the first and last segments cast in any one cantilever might be as great as 130 days. Using the strength and elastic modulus relationships proposed by Committee 209 of the American Concrete Institute which are:

\[ f'_{ct} = \left( \frac{t}{c + dt} \right) f'_c \]  \hspace{1cm} (6.1)

\[ E_{ct} = 33 \ w^{1.5} \sqrt{f'_{ct}} \]  \hspace{1cm} (6.2)
In which

\[ f'_{et} = \text{concrete strength at the age of time } t \]
\[ f'_{c} = \text{concrete strength at the age of 28 days} \]
\[ c, d = \text{coefficients for concrete strength which are taken to be 2.24 and 0.92 respectively for water cured concrete of usual quality. (Type III Cement).} \]
\[ E_{et} = \text{elastic modulus of the concrete at the age of } t \text{ days} \]
\[ w = \text{unit weight of the concrete which is taken as 145 pcf for normal concrete.} \]

one can show that in any one cantilever the elastic modulus can vary from 3,660,000 psi to 4,210,000 psi. The oldest concrete has an elastic modulus equal to 115% of that of the youngest. Obviously variations in elastic properties as great as this should be taken into account in computing the deflections of the cantilevers. Furthermore, in the case of cast-in-place construction, because the concrete is prestressed and incorporated in the structure at a relatively early age, a high portion of the total concrete shrinkage is effective in reducing the prestressing force. Additionally the total creep strain is much greater for concretes which are stressed at early ages. This is illustrated graphically in Fig. A.6 in Appendix A where it will be seen that concrete stressed at the age of 3 days exhibits a creep strain that is 160 percent of that for the same concrete being stressed at an age of 28 days, all other conditions being equal. This effect also has a tendency to increase the loss of prestress, add to deflection and render the accurate prediction of the deflection more difficult.

In the case of bridges composed of precast segments, the segments of any one cantilever are frequently cast one per day with the result that a cantilever which has 20 segments may have a difference in age between the oldest and youngest segment of the order of 25 to 28 days (including an allowance for weekends). Because the youngest segments would normally be 30 days old or more at the time of erection, the ratio of the elastic modulii of the oldest and youngest concrete at the time of erection would be of the order of 1.02 or less which is negligible. Because of the rapid rate of erection normally achieved with this mode of construction, the amount of creep which takes place during erection is less than that experienced with cast-in-place segments and the total creep strain is less because the concrete has a greater age at the time of stressing. For these reasons the loss of prestress is relatively low with precast segments as is the total deflection of the cantilever at the time of erection.

Studies of the relative deflection of cantilevers composed of cast-in-place and precast segments have been reported in the literature. (Ref.
Fig. 6.21 (a) Deflection of a segmentally constructed cantilever in which no provision has been made for camber. (b) Deflection of a segmentally constructed cantilever in which provision is made for camber.
These studies have shown the deflection of the former can be as great as 2.5 times that of the latter for identical cantilevers constructed under normal conditions for each construction mode.

Deflection control for a bridge erected in cantilever involves the determination of the deflection of each cantilever as it is assembled followed by the determination of the deflection of the total structure as new portions are added to it. These are illustrated in Figs. 6.21 and 6.22. A typical deflection curve for a cantilever that is erected segmentally without camber but including the effects of prestressing is shown in Fig. 6.21(a). The deflection curves for each stage of construction for the same cantilever in which the appropriate correction for camber has been built into the member, is shown in Fig. 6.21(b). The cantilever camber that is to be constructed into the structure at any point, such as point 3 in Fig. 6.21(a), is the deflection the point undergoes from the subsequently erected segments. In other words, point 3 must be constructed with a camber equal to the sum of the deflections due to the effects of segments 4 and 5. These are shown graphically in Fig. 6.21 in which they are designated 83.4 and 63.5. The deflection which must be taken into account in the construction of a four-span continuous bridge that is erected in cantilever, in addition to the cantilever deflection, is shown in Fig. 6.22. The downward deflection of the cantilevered ends results from the stressing of the continuity tendons in each span as the construction proceeds. The downward deflection of the cantilever accounts for the discontinuities in the deflection curves.

In cast-in-place construction the deflection analysis must also include the effects of the traveling form because it contributes significantly to the deflection of the cantilever. Because the traveler is moved forward as the construction progresses, the amount of deflection attributable to the traveling form progressively increases. When the construction of the cantilever is completed, the traveling form is removed and the elastic deflection caused by the traveling form is recovered.

The camber is built into the structure in cast-in-place cantilever construction much in the same way that it is in conventional cast-in-place construction. That is to say the elevation of the traveling form is field adjusted by a crew composed of surveyors and workmen in such a way that the desired camber is built into the structure. In addition, as the construction progresses, the surveyors periodically measure and plot the camber in order to compare the actual values to the computed values. If it is found the actual values differ significantly from the computed values, the camber provided in subsequently constructed segments can be adjusted to correct for the deviation between the two values.

In the case of precast segments which are provided with relatively
CONSTRUCTION CONSIDERATIONS

(a) Elevation

(b) Completion of Span 1 including continuity prestressing.

(c) Completion of Span 1 and Span 2 including continuity prestressing of Span 1 and 2.

(d) Completion of Spans 1, 2 and 3 including continuity prestressing of Spans 1, 2 and 3.

(e) Completion of Spans 1, 2, 3 and 4 including continuity prestressing of Spans 1, 2, 3 and 4.

(f) Completed structure including effect of continuity tendon extending from abut. 1 to Abut. 5.

(g) Completed structure including superimposed dead load.

Fig. 6.22 Deflections occurring during construction of a four-span bridge erected in cantilever, in addition to the cantilever deflection.
thick joints of mortar or concrete between the segments (a procedure which is not normally employed in contemporary segmental bridge construction), camber control can be handled with the precast segments in much the same manner as described above for cast-in-place construction.

Precast segments which are match cast are normally erected with thin joints which are of the order of 0.02 inches thick. Hence the camber must be provided in the shape of the segments themselves. To accomplish this when the segments are precast on rigid soffit forms (long-line method), the camber is built into the soffit form in much the same way as is done with conventional cast-in-place construction. When casting cells of either the horizontal or vertical type are used, the camber adjustment is made by orienting the previously cast segment, with respect to the segment in fabrication, in such a way as to provide the required relative position between the two segments. Horizontal and vertical Curvature as well as superelevation, if any, are provided for in the same manner. The adjustment of the previously cast segment with respect to the one under fabrication is done under the direction of a surveying crew working in the precasting yard. After each new segment is cast, the segments are again measured before they are removed from the casting cell and the data plotted as a means of insuring the work is

![Diagram of bridge pier and section](image)

Fig. 6.23 Procedure for adjusting the final position of a cantilever.
being done as planned. If errors are detected during segment fabrication, corrections can be built into subsequent segments as is done with cast-in-place segmental construction.

If, during erection of the precast segments, it is found the anticipated camber is not being achieved with the desired accuracy, the position of a segment with respect to the previously erected one can be altered by placing shims of metal or wire fabric in the joints in addition to the epoxy adhesive. Alternatively, in some instances epoxy mortar has been used to alter the position between precast segments. These procedures have not frequently been used in precast segmental construction and should not be needed if proper attention is directed to segment fabrication.

Another means of adjusting for deviations between the actual and anticipated positions of completed cantilevers is shown in Fig. 6.23. The procedure cannot be used if the superstructure and substructure are permanently connected from the time the pier segment is first placed. The procedure consists of providing space for hydraulic jacks, which can be conventional hydraulic jacks or flat jacks, in such a manner that the structure can be slightly raised, lowered or tilted and then shimmed into the desired position before the permanent superstructure-substructure connection is installed or completed.

6.6 Quantity of Prestressing Material

Prestressing steel, ducts and anchorages constitute important cost items in prestressed concrete bridges. The number of anchorages required is an especially important parameter for use in comparing the relative costs of two or more designs. Each anchorage represents a cost made of the following:

1. The cost of the anchorage itself.
2. The formwork required to accommodate the anchorage.
3. The labor required for placing the anchorage as well as for stressing and grouting the tendon that terminates in the anchorage.
4. Providing the necessary permanent protection for the anchorage (concrete, formwork, curing, etc. for the concrete cover).

When the balanced cantilever erection technique is used, a considerably greater number of anchorages is required than is needed when conventional continuous cast-in-place construction methods are employed. This is due to high cantilever dead load moments and the sequential construction process that requires a few tendons to be stressed at each segment. For the segmental construction technique to be the more
economical, the cost of the greater number of prestressing anchorages must be offset by savings in falsework costs and savings which might occur as a result of more rapid erection. A comparison of the quantities of prestressing materials required for different solutions for a bridge superstructure 600 feet long and 32.5 feet wide is shown in Table 6.1.

<table>
<thead>
<tr>
<th>DESIGN</th>
<th>SPANS FEET</th>
<th>NPE</th>
<th>ANCHORAGES* EACH</th>
<th>STRAND L.F.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>180-240-180</td>
<td>CIP Box</td>
<td>48</td>
<td>174,000</td>
</tr>
<tr>
<td>2</td>
<td>170-260-170</td>
<td>CIP Segmental</td>
<td>216</td>
<td>156,000</td>
</tr>
<tr>
<td>3</td>
<td>90-140-140-140-90</td>
<td>Precast Segmental</td>
<td>232</td>
<td>120,000</td>
</tr>
</tbody>
</table>

*12-1/2" φ 270 strand anchorages.
In the case of Portland Cement Concrete, class 325 or 400, hardened under normal conditions, (in particular without heat curing), and subjected to working stresses at most equal to 0.35 $\sigma'_{j}$**, in the absence of more precise experimental results, the long-term deformations of the concrete may be evaluated as follows:

1. Deformation due to shrinkage.

For concretes which have been protected against excessive loss of moisture at early age, the shrinkage, as a function of time, may be determined using the expression $r(t)$, in which $r(t)$ is the function of time defined below in section 1.5 and $r$ is the product of the four factors given in the following paragraphs.

$$r = k_b \varepsilon_c k_c k_p$$

*From Reference 14, with minor editing.

**$\sigma'_{j}$ = Compressive strength of concrete or the age of j days.
1.1 The coefficient $k_b$ depends on the composition of the concrete; it is defined in Fig. A.1.

![Fig. A.1 Chart showing relationship between coefficient $k_b$ and water-cement ratio for various cement contents. (Note that 100 Kg/m$^3$ equals 169 pounds per cubic yards or 1.79 sacks per cubic yard).](image1)

1.2 The coefficient $\varepsilon_c$ depends on climatic conditions; it is defined in Fig. A.2.

![Fig. A.2 Shrinkage coefficient $\varepsilon_c$ shown as a function of relative humidity of average ambient air during service.](image2)
For heated floors, ovens etc., the values of \( c \) should be drawn from experience.

1.3 The coefficient \( k_{e1} \) depends on the theoretical thickness of the member; it is defined in Fig. A.3.

![Fig. A.3 Coefficient \( k_{e1} \) versus the theoretical thickness \( e_m \).](image)

The theoretical thickness \( e_m \) is defined as the quotient of the area \( B \) of the section divided by the semi-perimeter \( p/2 \) in contact with the atmosphere; for a section with one dimension very large with respect to the other, the theoretical thickness is very close to the actual thickness.

If the dimensions are not constant along the member, it is possible to take a theoretical thickness which is an average, paying particular attention to sections subjected to the maximum stresses.

1.4 The coefficient \( k_p \) depends on the percentage of steel \( \omega = \frac{A}{B} \), the ratio of the sectional area of all the longitudinal steel (provided that it is bonded) to the transverse sectional area of the member. It is expressed by the formula:

\[
k_p = \frac{1}{1 + \frac{n\omega}{n}}
\]

with \( n = 20 \), having regard to the effects of creep.
1.5 The law $r(t)$, expressing the development of long-term deformation with time, as a function of the time, is given in Fig. A.4.

Fig. A.4 Rate of deferred deformation (shrinkage and creep) with respect to time. Time is actual time from application of load in the case of creep and is “theoretical time” (defined in text) for shrinkage.

In order to take into account the size of the member, instead of the actual time $t$ (age of the concrete) the “theoretical time” is used in the diagram, obtained from the following expression:

$$t_i = t \sqrt{\frac{10}{e_m}}$$

in which the theoretical thickness $e_m$ is expressed in cm.

That part of the deformation due to shrinkage in any interval of time $(t - t_i)$ is equal to:

- $r [r(t) - r(t_i)]$

At an early age, the shrinkage of concrete which is protected is lower than that in concrete which is not protected, (which is of importance in the avoidance of cracks in concrete which is young and thus has a low strength). This difference decreases and finally vanishes with time. The phenomenon is less noticeable for very thick members.
2. Deformation due to creep.

As a first approximation, it may be assumed that creep is of a linear nature, in evaluating the order of magnitude of long-term deformation due to creep. This assumption, in the case of a constant stress $\sigma'_b$, leads to the calculation of the final creep deformation $\epsilon_\text{f}$ from the formula:

$$\epsilon_\text{f} = \frac{\sigma'_b}{E_i} K_n \cdot r(t)$$

in which $\sigma'_b$ is the compressive stress in the concrete, $E_i$ is the instantaneous elastic modulus of the concrete at the age of $j$ days, $r(t)$ is the same function of time used in the shrinkage and $K_n$ is the product of the four coefficients described in the following paragraphs.

$$K_n = k_b k_c k_d k_e$$

2.1 The coefficient $k_b$ depends on the composition of the concrete; its value is identical with that given for shrinkage (see Section 1.1).

2.2 The coefficient $k_c$ depends on the climatic conditions; it is defined in Fig. A.5.

![Fig. A.5](image_url)

Fig. A.5 Creep ratio $k_c$ versus relative humidity of average ambient air during service.
2.3 The coefficient $k_d$ depends on the hardening of the concrete at the time of loading; it is defined in Fig. A.6, as a function of the age $t$ of the concrete at loading.

![Diagram](image)

A.6 Coefficient $k_d$ versus the age of the concrete at loading for ambient temperature of 20°C (68°F). (See text for adjusting for different average ambient temperatures.)

The values in the diagram correspond to an average surrounding temperature of 20°C (68°F); where the temperatures are below this value, a correction should be made to the age of loading and the new value $t_c$ is given in the following formula, valid for temperatures lying between -10°C (14°F) and +20°C (68°F);

$$t_c = \frac{\Sigma t(\theta + 10)}{30}$$

in which:

- $t_c$ represents the corrected loading age expressed in days,
- $t$ represents the number of days during which the hardening has occurred at a temperature $\theta$ expressed in degrees centigrade.

2.4 The coefficient $k_{ez}$ depends on the theoretical thickness $e_m$ of the member defined in Section 1.3; it is given in Fig. A.7.
2.5 The law $r(t)$ is the same as for shrinkage, but the number of days is counted from the application of the load; on the other hand, the actual time* is taken into account, since the development of creep with time is less sensitive than that of shrinkage, to the influence of a reduction in the theoretical thickness of the member.

2.6 At a given time $t$ counted from the application of loads, the effect of a stress $\sigma'_{bj}$ applied at a moment $j$ and subjected at any instant $i$ to variations in intensity $\Delta\sigma'_{bi}$ (algebraically) may be taken as equal to:

$$\epsilon_n = k_b k_c k_e \left[ \frac{\sigma'_{bj}}{E_j} \cdot k_e r(t - j) + \sum \frac{\Delta\sigma'_{bi}}{E_i} k_d r(t - i) \right]$$

an expression in which the moduli $E_j$ and $E_i$ are both instantaneous moduli, the subscripts $i$ and $j$ referring to the moments designated above.

*In contrast to the theoretical time.
ASSHTO-PCI Standard Bridge Beams types I through VI.

**TABLE B.1**

<table>
<thead>
<tr>
<th>Beam Type</th>
<th>Area $\text{in}^2$</th>
<th>Moment of Inertia $\text{in}^4$</th>
<th>$y_t$ in</th>
<th>Span feet</th>
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</thead>
<tbody>
<tr>
<td>I</td>
<td>276</td>
<td>22,750</td>
<td>15.41</td>
<td>30-45</td>
</tr>
<tr>
<td>II</td>
<td>369</td>
<td>50,980</td>
<td>20.17</td>
<td>40-60</td>
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<tr>
<td>III</td>
<td>560</td>
<td>125,390</td>
<td>24.73</td>
<td>55-80</td>
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<tr>
<td>IV</td>
<td>789</td>
<td>260,730</td>
<td>29.27</td>
<td>70-100</td>
</tr>
<tr>
<td>V</td>
<td>1013</td>
<td>521,180</td>
<td>31.04</td>
<td>90-120</td>
</tr>
<tr>
<td>VI</td>
<td>1085</td>
<td>733,320</td>
<td>35.62</td>
<td>110-140</td>
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TABLE 8.2

<table>
<thead>
<tr>
<th>Depth (in)</th>
<th>Area (in^2)</th>
<th>Moment of Inertia (in^4)</th>
<th>Yr (in)</th>
<th>Span (Feet)</th>
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<tbody>
<tr>
<td>36</td>
<td>432</td>
<td>63,300</td>
<td>18.9</td>
<td>50-55</td>
</tr>
<tr>
<td>48</td>
<td>516</td>
<td>137,300</td>
<td>25.2</td>
<td>65-75</td>
</tr>
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<td>54</td>
<td>558</td>
<td>187,800</td>
<td>28.3</td>
<td>75-80</td>
</tr>
<tr>
<td>60</td>
<td>600</td>
<td>248,600</td>
<td>31.4</td>
<td>80-90</td>
</tr>
<tr>
<td>66</td>
<td>642</td>
<td>318,000</td>
<td>34.4</td>
<td>90-100</td>
</tr>
<tr>
<td>72</td>
<td>684</td>
<td>400,600</td>
<td>37.5</td>
<td>100-110</td>
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</table>

Fig. 8.2 California standard I-girders.

Fig. 8.3 Colorado standard girders.
### TABLE 8.3

<table>
<thead>
<tr>
<th>Depth (in)</th>
<th>Area (in²)</th>
<th>Moment of Inertia (in⁴)</th>
<th>Yt (in)</th>
<th>Maximum Span (Feet)</th>
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<tr>
<td>54</td>
<td>630.5</td>
<td>242.59</td>
<td>21.33</td>
<td>105</td>
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<td>68</td>
<td>647.5</td>
<td>413,725</td>
<td>34.08</td>
<td>125</td>
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<tr>
<td>72</td>
<td>759.5</td>
<td>538,144</td>
<td>32.76</td>
<td>134</td>
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</tbody>
</table>

Fig. 8.4 Standard precast concrete girders, State of Washington.

### TABLE 8.4

<table>
<thead>
<tr>
<th>Depth (in)</th>
<th>Area (in²)</th>
<th>Moment of Inertia (in⁴)</th>
<th>Yt (in)</th>
<th>Nominal Span (Feet)</th>
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<tr>
<td>32.0</td>
<td>253</td>
<td>31,000</td>
<td>16.84</td>
<td>40</td>
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<tr>
<td>42.0</td>
<td>332</td>
<td>70,100</td>
<td>23.31</td>
<td>60</td>
</tr>
<tr>
<td>50.0</td>
<td>476</td>
<td>154,900</td>
<td>27.47</td>
<td>80</td>
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<tr>
<td>58.0</td>
<td>546</td>
<td>249,000</td>
<td>30.10</td>
<td>100</td>
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<tr>
<td>73.5</td>
<td>626</td>
<td>456,000</td>
<td>37.90</td>
<td>120</td>
</tr>
</tbody>
</table>
Definition of the Constants

1. Elasticity of the joints with respect to the beam (K).

Consider the structural system shown in Fig. C.1 which incorporates a beam spanning between joints 1 and 2. Rotations $\theta_1$ and $\theta_2$ occur at joints 1 and 2.
and 2 respectively, due to the applications of loads to beam 12. The rotations $\theta_1$ and $\theta_2$ induce moments $M_{12}$ and $M_{21}$ in beam 12 at joints 1 and 2 respectively. Moments of equal magnitude, but opposite direction are introduced in the supporting members at joints 1 and 2. The elasticity of joint 1 with respect to beam 12 is defined as follows:

$$K_{1/12} = \frac{\theta_1}{M_{12}}$$  \hspace{1cm} (1)

The moment $M_{12}$ is applied to the joint 1 as shown in Fig. C.2. In a similar manner the elasticity of joint 2 with respect to beam 21 is as follows:

$$K_{2/21} = \frac{\theta_2}{M_{21}}$$  \hspace{1cm} (2)
The stiffness (R) is reciprocal of the elasticity. The stiffness of the joint 1 with respect to beam 12 is equal to:

\[ R_{1/12} = \frac{M_{12}}{\theta_1} \]  \hspace{1cm} (3)

and that for joint 2 with respect to beam 21 is:

\[ R_{2/21} = \frac{M_{21}}{\theta_2} \]  \hspace{1cm} (4)

It should be recognized that stiffnesses, but not elasticities, can be added.

Considering the freebody of the joint 1 shown in Fig. C.3, the member 13 has a moment \( M_{13} \) at joint 1 equal to:

\[ M_{13} = \theta_1 R_{13} \]  \hspace{1cm} (5)

In a similar manner for member 14, there is a moment at joint 1 equal to:

\[ M_{14} = \theta_1 R_{14} \]  \hspace{1cm} (6)

and for member 15 the moment is:

\[ M_{15} = \theta_1 R_{15} \]  \hspace{1cm} (7)

Finally, one can relate the sum of the moments in the members which constitute the supports at joint 1, to the moment \( M_{12} \), as follows:

\[ M_{12} = M_{13} + M_{14} + M_{15} \]  \hspace{1cm} (8)

From which comes the equivalent expression:

\[ \theta_1 R_{1/12} = \theta_1 (R_{13} + R_{14} + R_{15}) \]  \hspace{1cm} (9)

and

\[ R_{1/12} = R_{13} + R_{14} + R_{15} \]  \hspace{1cm} (10)

Hence, the stiffness of the joint 1 with respect to the beam 12 is equal to the sum of the stiffnesses of all the members at joint 1 except for beam 12. Knowing the stiffnesses of the members which frame into the joints, one can determine the joint stiffness and elasticity.
2. Elasticity of the beam with respect to the joints (k).

If one assumes beam 12 of Fig. C.1 is hinged at joint 1 and a moment is applied at end 1 as shown in Fig. C.4, the elasticity of the beam with respect to the support is:

$$k_{12/1} = \frac{\theta_1}{m_1}$$  \hspace{1cm} (11)

and the stiffness of the beam with respect to the support is:

$$r_{12/1} = \frac{m_1}{\theta_1}$$  \hspace{1cm} (12)

In a similar manner the elasticity and stiffness of the beam with respect to the joint 2 are:

$$k_{21/2} = \frac{\theta_2}{m_2}$$  \hspace{1cm} (13)

and

$$r_{21/2} = \frac{m_2}{\theta_2}$$  \hspace{1cm} (14)

3. Properties of the Beam

If beam 12 is simply supported and a unit moment is applied at end 1, rotations which are designated a and b are obtained at ends 1 and 2 as shown in Fig. C.5. It can be shown that:

$$a = \frac{1}{l^2} \int_{0}^{1} (l-x)^2 \frac{dx}{EI}$$  \hspace{1cm} (15)

Fig. C.4 Freebody diagram with hinge at joint 1.
Fig. C.5 Freebody diagram for beam 12 simply supported with a unit moment at 1.

where \(x\) is measured from 1 toward 2; and

\[
b = \frac{1}{l^2} \int_0^l x(l-x) \frac{dx}{EI}
\]

In which \(E\) and \(I\) are the elastic modulus and the moment of inertia of the beam respectively.

In a similar manner if a unit moment is applied at end 2 as shown in Fig. C.6, rotations equal to \(b\) and \(c\) are obtained at ends 1 and 2 respectively. The value of \(c\) is:

\[
c = \frac{1}{l^2} \int_0^l x^2 \frac{dx}{EI}
\]

For a member of constant moment of inertia:

\[
a = c = 2b = \frac{l}{3EI}
\]
The rotation at the ends opposite to that at which the unit moment is applied is equal to \( b \) in each case. This can be explained by Maxwell’s Law of **Reciprocal** Deflections.

For loads applied to the simply supported beam 12, rotations occur at ends 1 and 2 which are designated \( \omega_1 \) and \( \omega_2 \) respectively. This is illustrated in Fig. C.7.

![Fig. C.7 Simply supported beam with transverse loads.](image)

4. **End moments on beam 12 including the effects of continuity.**

The rotations at the ends of beam 12, including the effects of loading within the span as well as the end moments due to continuity, as shown in Fig. C.8, can be expressed as follows:

\[
\theta_1 = \omega_1 + aM_{12} - bM_{21} \quad (19)
\]

and

\[
\theta_2 = \omega_2 - bM_{12} + cM_{21} \quad (20)
\]

Substituting equations (1) and (2) for \( \theta_1 \) and \( \theta_2 \) respectively in equations (19) and (20) and solving for \( M_{12} \) and \( M_{21} \), one obtains:

\[
M_{12} = \frac{(c + K_{221}) \omega_1 + b\omega_2}{(a + K_{112})(c + K_{221}) - b^2} \quad (21)
\]

and

\[
M_{21} = -\frac{b\omega_1 + (a + K_{112})\omega_2}{(a + K_{112})(c + K_{221}) - b^2} \quad (22)
\]

In the equations (21) and (22) “beam” moment sign convention, as shown in Fig. C.9 is **used.**
Fig. C.8  Rotations at the ends of beam 12 resulting from transverse loads and end moments.

Fig. C.9  Sign convention for rotation and moment (beam convention).
5. Computation of the elasticities of the beam.

With the elasticity of the joint 1 ($K_{1/12}$) being known, the elasticity of the beam at the support 2 ($k_{2/12}$) is determined by applying a moment $m_2$ in beam 21, at joint 2 (hinge assumed at 2 in beam 21). This is illustrated in Fig. C.10. The moment $m_2$ induces a moment of $M_{12}$ in beam 12 at joint 1. (See Fig. C.11.) Therefore:

$$\theta_1 = K_{1/12} M_{12}$$  \hspace{1cm} (23)

and

$$\theta_2 = k_{2/12} m_2$$  \hspace{1cm} (24)

but

$$\theta_1 = -aM_{12} - bm_2$$  \hspace{1cm} (25)

and

$$\theta_2 = bM_{12} + cm_2$$  \hspace{1cm} (26)

from which

$$K_{1/12} M_{12} = -aM_{12} - bm_2$$  \hspace{1cm} (27)

and

$$k_{2/12} m_2 = bM_{12} + cm_2$$  \hspace{1cm} (28)

Fig. C.10  Frame with hinge assumed at 2.
Therefore

\[(a + K_{1/12})M_{12} = -bm_2\]  \hspace{1cm} (29)

and

\[(c - km) m_2 = -bM_{12}\]  \hspace{1cm} (30)

which combined gives

\[(a + K_{1/12})(c - k_{21/2}) - b^2 = 0\]  \hspace{1cm} (31)

From which the general equations for the elasticities of the beam are determined to be:

\[k_{21/2} = c - \frac{b^2}{a + K_{1/12}}\]  \hspace{1cm} (32)

and

\[k_{12/1} = a - \frac{b^2}{c + K_{2/21}}\]  \hspace{1cm} (33)


Rearranging equation (29), one obtains:

\[M_{12} = \frac{-b}{a + K_{1/12}} \cdot m_2\]  \hspace{1cm} (34)

Therefore, the carryover factor from 2 to 1 is:

\[C_{21} = \frac{b}{a + K_{1/12}}\]  \hspace{1cm} (35)

\[\text{Fig. C.11} \hspace{1cm} \text{Relationship between joint moment } m_2 \text{ at beam moment } M_{12}.\]
and from 1 to 2

\[ C_{12} = \frac{b}{c + K_{2/1}} \]  

(36)

The carryover factors are illustrated in Fig. C.12.

The equations for the end moments in the continuous beam can be written from equations (21) and (22) as:

\[ M_{12} = - \frac{c_{21}}{b} \cdot \frac{\omega_1 + C_{12} \omega_2}{1 - C_{2,2} C_{2,1}} \]  

(37)

\[ M_{21} = \frac{C_{12}}{b} \cdot \frac{C_{21} \omega_1 + \omega_2}{1 - C_{1,2} C_{2,1}} \]  

(38)

**SUMMARY:**

*Elasticity of the Joints:* The elasticity of the joint is computed as the reciprocal of the sum of the stiffnesses of the members which frame into the joint, neglecting the beam under consideration. The elasticity of a member is equal to the angle of rotation of the member under a unit moment, assuming the member is hinged at the joint under consideration. For

![Fig. C.12 Diagram showing definition of carry-over factors.](image-url)
prismatic members, the relative stiffnesses for members hinged and fixed at their far ends are:

\[ \text{Hinged members } R = \frac{3EI}{L} \]  \hspace{1cm} (39)

\[ \text{Fixed members } R = \frac{4EI}{L} \]  \hspace{1cm} (40)

**Elasticities of the Beams**

\[ k_{12/1} = a - \frac{b^2}{c + K_{2/21}} \]  \hspace{1cm} (41)

\[ k_{21/2} - c = \frac{b^2}{a + K_{1/12}} \]  \hspace{1cm} (42)

**Carryover Factors**

\[ c_{12} = \frac{b}{c + K_{2/21}} \]  \hspace{1cm} (43)

\[ c_{21} = \frac{b}{a + K_{1/12}} \]  \hspace{1cm} (44)

**End Moments**

\[ M_{12} = + \frac{c_{21}}{b} \cdot \frac{\omega_1 + c_{12} \omega_2}{1 - c_{12} c_{21}} \]  \hspace{1cm} (45)

\[ M_{21} = - \frac{c_{12}}{b} \cdot \frac{c_{21} \omega_1 + \omega_2}{1 - c_{12} c_{21}} \]  \hspace{1cm} (46)

**Example Problem**

Compute the bending moments due to the applied live load of 10 klf as well as the secondary bending moments due to the prestressing force for the structure shown in Fig. C. 13. The structure can be idealized as shown in Fig. C. 14 where it will be seen the moments of inertia for the beams and piers are 400 ft.\(^4\) and 100 ft.\(^4\) respectively.

The properties of the structure are determined by first finding the unit moment coefficients a, b and c using Eq. 18. These are as follows:
For piers 25 and 36 (taking $E = 1$)

$$a = c = 2b = \frac{20}{3 \times 100} = 0.0666$$

For beams 12 and 34 (taking $E = 1$)

$$a = c = 2b = \frac{100}{3 \times 400} = 0.0833$$

For beam 23 (with $E = 1$)

$$a = c = 2b = \frac{130}{3 \times 400} = 0.108$$

Considering pier 25 (Fig. C.15, because joint 5 is hinged, $(K_{552} = \infty)$, from Eq. 33:

$$k_{525} = a = 0.0666$$

In the case of pier 36 (Fig. C.16), the joint 6 is fixed $(K_{636} = 0)$ and from Eq. 32:

$$k_{363} = 0.0666 - \frac{0.0333^2}{0.0666} = 0.05$$

Fig. C.14 Schematic layout for frame of Fig. C.13.
For beam 12, joint 1 is hinged \((K_{112} = \infty)\) and from Eq. 32:

\[
k_{21/2} = 0.0833 - \frac{0.0416^2}{0.0833 + \infty} = 0.0833
\]

For joint 2 with respect to span 23, by Eq. 10:

\[
\frac{1}{K_{2/23}} = \frac{1}{k_{21/2}} + \frac{1}{k_{25/2}}
\]

\[
K_{2/23} = 0.0833 + 0.0666 = 0.15
\]
The elasticity of span 23 with respect to joint 3, from Eq. 32 is:

\[ k_{32/3} = 0.108 \frac{0.0542}{0.037 + 0.108} = 0.0879 \]

and the elasticity of joint 3 with respect to span 34 is computed using Eq. 10 as follows:

\[ K_{3/34} = 0.032 \]

Joint 4 is fixed. Therefore \( K_{4/32} = 0 \). The elasticity of span 43 with respect to joint 3 is:

\[ k_{43/3} = 0.0833 - \frac{0.0416}{0.0833 + 0} = 0.0625 \]

and the elasticity of joint 3 with respect to span 32, is:

\[ K_{3/32} = 0.0277 \]

and

\[ k_{32/2} = 0.108 - \frac{0.0542}{0.108 + 0.0277} = 0.0866 \]

from which

\[ K_{2/21} = 0.0866 + 0.0666 \]

and

\[ K_{2/21} = 0.0377 \]

The joint elasticities can be summarized as follows:

\[ K_{1/12} = \infty \quad K_{2/21} = 0.0377 \]
\[ K_{2/23} = 0.037 \quad K_{3/32} = 0.0277 \]
\[ K_{3/34} = 0.032 \quad K_{4/43} = 0 \]
The distribution factors for each joint are determined from the stiffness of the members. For moment $M_{23}$ as shown in Fig. C.17:

$$k_{21/2} = 0.0833 \quad r_{21/2} = 12 \quad d_{21} = 0.444$$

$$k_{25/2} = 0.0666 \quad \frac{r_{25/2}}{\Sigma r} = \frac{15}{27} \quad d_{25} = 0.556$$

and for $M_{21}$ as shown in Fig. C.18.

$$k_{23/2} = 0.0866 \quad r_{23/2} = 11.54 \quad d_{23} = 0.435$$

$$k_{25/2} = 0.0666 \quad \frac{r_{25/2}}{\Sigma r} = \frac{15.00}{26.54} \quad d_{25} = 0.565$$

and for $M_{32}$ as shown in Fig. C.19

$$k_{34/3} = 0.0625 \quad r_{34/3} = 16 \quad d_{34} = 0.444$$

$$k_{36/3} = 0.050 \quad \frac{r_{36/3}}{\Sigma r} = \frac{20}{36} \quad d_{36} = 0.556$$

and for $M_{34}$ as shown in Fig. C.20

$$k_{313} = 0.0879 \quad r_{323} = 11.38 \quad d_{32} = 0.362$$

$$k_{36/3} = 0.050 \quad \frac{r_{36/3}}{\Sigma r} = \frac{20}{31.38} \quad d_{36} = 0.638$$
The carry over factors from right to left are computed from Eq. 35 while that from left to right from Eq. 36. These are:

\[ C_{21} = \frac{0.0416}{0.0833 + \infty} = 0 \quad C_{12} = \frac{0.0416}{0.0833 + 0.037} = 0.344 \]

\[ C_{32} = \frac{0.0416}{0.108 + 0.037} = 0.372 \quad C^{23} = \frac{0.054}{0.108 + 0.0277} = 0.40 \]

\[ C_{43} = \frac{0.0416}{0.0833 + 0.032} = 0.36 \quad C_{34} = \frac{0.0416}{0.0833 + 0} = 0.50 \]
This completes the computations of the properties of the structure.

Under the loading of 10 klf on span 23, the simple span values of $E\omega_2$ and $E\omega_3$ can be shown to be:

$$E\omega_2 = \sim 2288$$

$$E\omega_3 = + 2288$$

in which a counterclockwise rotation is positive. Using Eq. 37 and 38, the end moments in span 23 are found to be as follows:

$$M_{23} = + \frac{0.372}{0.054} \times \frac{2288}{0.372 \times 0.4} = \sim 11'113 \text{ K'Ft}$$

$$M_{32} = - \frac{0.054 \times 0.4}{0.372 \times 0.4} \times 2288 = \sim 12'507 \text{ K'Ft}$$

Using the distribution factors computed above for $M_{23}$ and $M_{32}$ one obtains a distribution of moments as shown in Fig. C.21.

The secondary moments for the portion of the tendon in span 23 is computed next. The same procedure should be used for the portions of the tendon in spans 12 and 34 and the results of the three analyses added together for the combined effect. Refer to Fig. C.22. Assuming the tendon
is on a trajectory of compounded second degree parabolas, the areas of the moment curves are computed as follows (E = 1):
Between 12 and 34:

\[
\frac{Fe}{l} = \frac{5000 \times 4}{400} = -50
\]

Area = \(-50 \times 30 \times \frac{2}{3} = -1000\)

Between 23:

\[
\frac{Fe}{l} = \frac{5000 \times 5}{400} = 62.5
\]

Area = \(62.5 \times 70 \times \frac{2}{3} = +2917\)

\[
\omega_2 = \frac{+2917 - 1000}{2} \frac{1000}{1000} = +458
\]

\[
\omega_3 = -458
\]

and the moments \(M_{23}\) and \(M_{32}\) are as follows:

\[
M_{23} = 0.372 \times 458 - 0.4 \times 458 \times 0.372 \times 0.4 = 2224
\]

\[
M_{32} = -0.4 \times 0.372 \times 458 + 0.372 \times 458 \times 0.4 = 2503
\]
and the secondary moment diagram as shown in Fig. C.23 is found using the distribution factors for moments $M_{23}$ and $M_{32}$.

Note: When the tops of the piers are free to translate horizontally, a general method must be used for the computation of the piers. With reference to Fig. C.24, it will be seen that the pier is subject to a bending moment of $M$ and a horizontal force $Q$ which are applied at the top. The rotation at the top due to $M$ and $Q$ is $\theta$ while the displacement is equal to 6. Using the notation:

- $A$ = the rotation due to the moment (elasticity for rotation.)
- $B$ = the rotation due to the horizontal force, which because of the Maxwell Theory is equal to the displacement due to the moment.
- $C$ = the displacement due to the horizontal force (elasticity for displacement.)
- $E$ = Young’s modulus.

Fig. C.24  Freebody diagram for typical pier.
it can be shown that

$$E\theta = AM + BQ$$  \hspace{1cm} (47) 

$$E\delta = BM + CQ$$  \hspace{1cm} (48) 

in which

$$A = \int_{o}^{h} \frac{dx}{I}$$  \hspace{1cm} (49) 

$$B = \int_{o}^{h} \frac{x dx}{I}$$  \hspace{1cm} (50) 

$$C = \int_{o}^{h} \frac{x^2 dx}{3I}$$  \hspace{1cm} (51) 

If the top of the pier cannot move, \( E\theta = 0 \), \( \delta = -\frac{B}{C} M \) \hspace{1cm} (52) 

and

$$E\theta = M(A - \frac{B^2}{C}) = kM$$  \hspace{1cm} (53) 

or

$$k = A - \frac{B^2}{C}$$  \hspace{1cm} (54) 

For a constant moment of inertia,

$$A = \frac{h}{I}$$  \hspace{1cm} (55) 

$$B = \frac{h^2}{2I}$$  \hspace{1cm} (56) 

$$C = \frac{h^3}{I}$$  \hspace{1cm} (57) 

In the case of pier 36, \( A = 0.20 \), \( B = 2.00 \) and \( C = 26.67 \) and

$$k = 0.20 - \frac{2^2}{26.67} = 0.05$$

This is the same value computed above.
For the case where the top of the pier cannot deflect horizontally ($E\delta=0$) it can be shown that the distance $x$ from the top of the pier to the point of zero moment is:

$$x = \frac{C}{B} \quad (58)$$

Under the effect of a lateral load, the tops of the piers deflect as shown in Fig. C.25. Considering the joint at the top of the pier on the left side, it can be shown that for the pier Eq. 47 applies while the rotations result in the following relationships for the beams:

span 1: \[ E\theta = -c_1 M' \quad (59) \]

Span 2: \[ E\theta = -(a_2 - b_2) M'' \quad (60) \]

and for the joint:

\[ M = M' + M'' \quad (61) \]

Fig. C.25 Deflected shape of a frame under lateral load.

From the above one obtains:

\[ M' = \frac{(a_2 - b_2)}{c_1 + (a_2 - b_2)} M \quad (62) \]

and

\[ M'' = \frac{c_1}{c_1 + (a_2 - b_2)} M \quad (63) \]

If

\[ \lambda = \frac{1}{(a_2 - b_2)} + \frac{1}{c_1} \quad (64) \]
one can show that

$$M = \frac{-B\lambda Q}{Ah + 1}$$  \hspace{1cm} (65)

and the distance $x$ from the top of the pier to the point of zero moment

$$x = \frac{B\lambda}{Ax + 1}$$  \hspace{1cm} (66)

The effect of a length change, which may be due to a combination of creep, shrinkage or temperature-induced strain, the piers deflect as shown in Fig. C.26. Eq. 47 remains correct for the pier and the rotational relationships for the beams at the top of the left pier are as follows:

![Deflected shape of a frame due to change of beam length](image)

**Fig. C.26**  Deflected shape of a frame due to change of beam length.

Span 1: $E\theta = c_1 M'$  \hspace{1cm} (67)

Span 2: $E\theta = -(a_2 + b_2) M''$  \hspace{1cm} (68)

and for the joint Eq. 61 applies. From this one can show that

$$M' = \frac{(a_2 + b_2) M}{c_1 + (a_2 + b_2)}$$  \hspace{1cm} (69)

$$M'' = \frac{c_1 M}{c_1 + (a_2 + b_2)}$$  \hspace{1cm} (70)

and if

$$\lambda' = \frac{1}{(a_2 + b_2)} + \frac{1}{c_1}$$  \hspace{1cm} (71)

$$M = \frac{-B\lambda' Q}{Ah' + 1}$$  \hspace{1cm} (72)
and the distance $x$ from the top of the pier to the point of zero moment is:

$$x = \frac{BX'}{A\lambda' + 1}$$  \hspace{1cm} (73)

Finally, it can be shown that the relationship between the shear force $Q$ and the deflection at the top of the pier is:

$$Q = \frac{E\delta}{C} = \frac{B^2\lambda'}{AA' + 1}$$  \hspace{1cm} (74)
When the temperature distribution within a member is known, the thermal stresses in the section can be easily computed (Ref. 23). Priestley has shown if plane sections are assumed to remain plane, for a section as shown in Fig. D.1 subject to the temperature distribution shown in Fig. D.2 the thermally induced strains and stresses are as shown in Fig. D.3. The equations for stress and for force and moment equilibrium are as follows:

\[ f_y = E \left( \epsilon_1 + \epsilon_2 \frac{y}{d} - \alpha t_y \right) \quad (D.1) \]

\[ \epsilon_1 + \epsilon_2 \frac{n}{d} = \frac{\alpha}{A} \int_{0}^{d} t_y b_y \, dy \quad (D.2) \]

\[ \frac{\epsilon_2}{d} = \frac{\alpha}{I} \int_{0}^{d} t_y (y-n) b_y \, dy \quad (D.3) \]
in which the terms are defined in Figs. D.1 through D.3 and \( \alpha \) is the linear coefficient of thermal expansion, \( A \) is the area of the section and \( I \) is the moment of inertia of the section about the horizontal axis passing through the centroid. The procedure consists of solving equation (D.3) which gives the curvature of the section and the value of \( \varepsilon_2 \). With \( \varepsilon_2 \) known, equation (D.2) can be solved for the value of \( \varepsilon_1 \) and the stresses found from equation (D.1). The curvature found with equation (D.3) is used to determine the additional stresses induced by continuity in structures which are continuous. Curvature is equal to \( M/EI \) and deflections can be computed therefrom.

The bridge designer generally does not have precise data relative to the extreme temperature gradient a particular bridge will experience in service. For this reason an approximate method of analysis will yield results that are adequate for most design work. The approximate method consists

---

**Fig. D.1 Cross section of a tubular beam.**
Fig. D.2 Temperature gradient in beam of Fig. D.1.

Fig. D.3 Strain and stress distributions in beam of Fig. D.1 due to temperature gradient of Fig. D.2.
of assuming the top deck is raised to a uniform temperature higher than the other parts of the cross section. If unrestrained, the top deck would experience an increase in length as a result of the temperature increase. The expansion of the top deck is resisted by the webs and bottom flange.

The first step in computations with the approximate method consists of computing the forces that would exist in the top flange due to the increase in temperature if the flange were completely restrained. The force results in a compressive stress in the top flange. The effect of the force in the top flange on the section as a whole is computed by applying a tensile force equal in magnitude to the compressive force, applied at the centroid of the top flange. The sum of the stresses from the two forces gives the approximate stresses due to thermal effects in the section.

Fig. D.4 Typical bridge cross section.

Fig. D.5. Stresses in the bridge of Fig. D.4, by approximate calculation, for a temperature differential of 30°F.
As an example, consider the bridge cross section shown in Fig. D.4. Assume the top flange temperature is 30°F higher than that of the webs and bottom flange. The compressive stress in the top flange, if fully restrained, would be:

\[ fc = (At)(\alpha)(Ec) \]  

(D.4)

in which \( \alpha \) is the linear coefficient of thermal expansion and \( Ec \) is the elastic modulus of the concrete. If \( \alpha = 0.0000065 \) and \( Ec = 3,000,000 \) psi, \( fc = 585 \) psi. The force in the top flange would be:

\[ p = 585 \times 5000 = 2925 \text{ kips} \]

The tensile force applied 5.29 inches from the top of the section produces the stresses shown in Fig. D.5(b) and the combined stresses are as shown in Fig. D.5(c). These are the approximate stresses that would exist if the beam were a single simply-supported span.

If made continuous over four spans as shown in Fig. D.6(a), it can be shown that secondary moments and reactions as shown in Fig. D.5(b) would exist. The secondary reactions are those required to prevent the beam from deflecting upward from its supports as a result of the thermal gradient. The secondary moment at the first interior support results in a stress of 314 psi (compression) in the top fiber and 510 psi (tension) in the bottom fiber. The net stress in the member due to the effect of the differential temperature would be those obtained by combining the stresses shown in Fig. D.5(c) with those resulting from the secondary moment of Fig. D.6.
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