STRUCTURAL ASPECTS OF
CABLE-STAYED BRIDGE DESIGN

by

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(1973)

Submitted in partial fulfillment
of the requirements for the degree of
Master of Science in Civil Engineering

at the

Massachusetts Institute of Technology
June 1975

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ABSTRACT

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Submitted to the Department of Civil Engineering
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This thesis discusses the behavior of the cable-stayed
bridge and the problems encountered in design, analysis and
construction. The major bridge components, the cables,
towers and deck, are discussed in detail. Existing and pro-
posed designs are considered and a comparison of the advan-
tages and disadvantages of each of the major structural com-
ponents is presented. Preliminary and final analysis proce-
dures are discussed with emphasis on the non-linear, sag and
beam-column effects. Consideration is also given to the site
requirements, material selection and cable anchorages. Con-
struction techniques are discussed including popular methods
as well as those suited to specific site requirements. Cable
stayed systems are shown to have distinctive characteristics
which present new problems in analysis and material selection.
Once these problems are recognized, the cable stayed system
is shown to be an overall economic alternative in bridge
design.

Thesis Supervisor: Professor Jerome Connor

Title: Professor in Civil Engineering

Acknowledgment

I am greatly indebted to Professor Jerome Connor for his invaluable assistance and counsel during the time of research, preparation and writing of this thesis. I owe a special thanks to my sister Karen and to Roy, who generously contributed so much of their time and efforts. I also wish to thank Wally, my friend and translator, and Marge for her patience and skill in typing this thesis.

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Introduction

The cable-stayed bridge is a highly indeterminate structure, dependent upon the proper utilization of high strength cables for efficient design and satisfactory behavior. The cables are anchored directly to the main bridge girder and act as supports for the girder. Consequently, the cables eliminate the need for intermediate piers producing much longer spans than previously deemed economical for bridge girders.

The cable-stayed system was first proposed by C. J. Loscher, a German carpenter, in 1784. His design consisted of a wooden deck supported by a wooden tower and stays (Fig. 1). In 1818, a chain stayed pedestrian bridge, crossing the Tweed River in England, collapsed due to fatigue failure of the chain stays subjected to wind oscillations. Poyet, a French architect, proposed a fan type arrangement of steel bar stays in 1821 (Fig. 1). A similar version of the fan type arrangement was proposed for the Gischlard-Arnodin Bridge (Fig. 2). The first harp type stayed bridge was proposed by Hatley, an English engineer, which utilized chain stays in a parallel configuration (Fig. 2). The disaster which led to the early condemnation of the cable-stayed system was the collapse of a 260 foot span bridge across the Saale River in Germany. In 1925, a year after the construction of the bridge, an overload of people
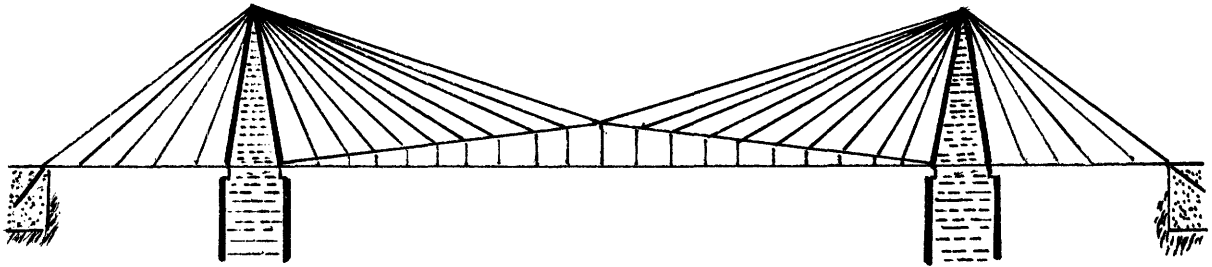
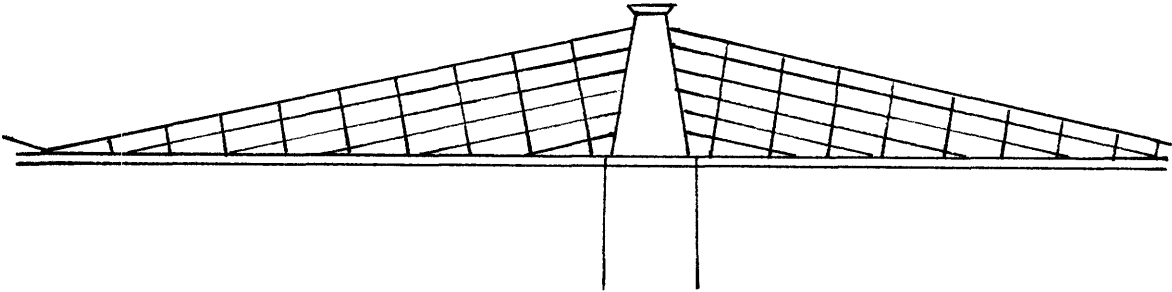5
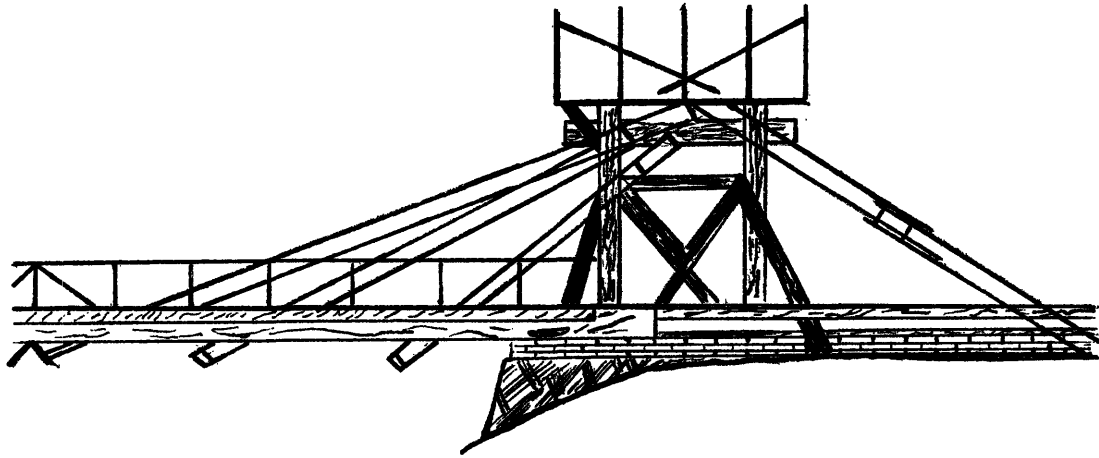
Timber Bridge by C. J. Loscher (1784)
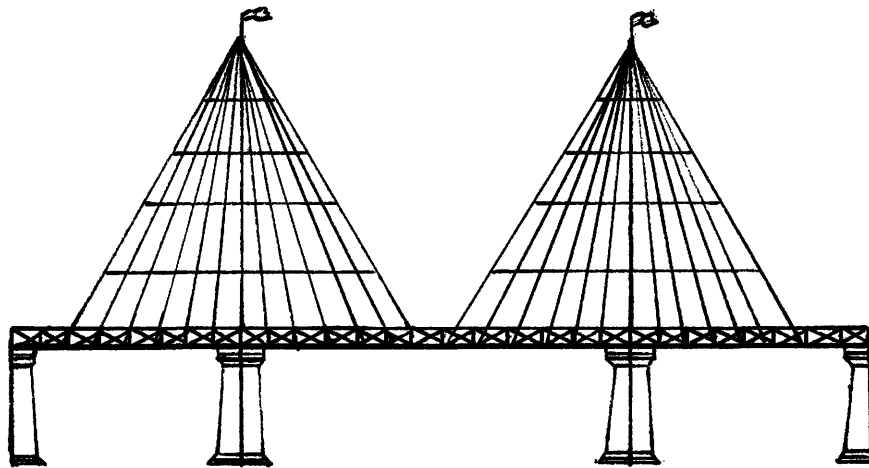
Design by Poyet

Figure 1. Early Designs

Gischlard-Arnodin Bridge, Fan Type

Harp Configuration by Hatley (1840)

Figure 2. Cable Arrangements

caused the collapse in which many lives were lost. Navier,
a French engineer, researched the early failures and his re-
ports led engineers to other alternatives for their bridge
designs.

The early failures were a result of the engineers in-
ability to understand the behavior of the cable-stayed
bridges. The design procedures used during this period were
not adequate for the complicated indeterminate stay system.
The stiffness of the structural system is highly dependent
on the stiffness of the stays. The materials used for the
stays in these early designs, such as steel bars and chains,
were not strong enough and could not be initially stressed
to a point where they would contribute to the behavior of
the bridge to the degree for which they were intended. The
development of high strength steel cables and the advance-
ment of analytical theories for statically indeterminate
structures have overcome the problems associated with these
early failures and have made the cable-stayed system a viable
alternative for bridge design.

There are many options available to the engineer in
the selection of the suitable materials and design of each
of the major bridge components. The purpose of this thesis
is to present these alternatives in a manner that will enable
the engineer to choose the most efficient design satisfying
his specific requirements.

Chapter 1 - Cable Stays

Section 1.1   Introduction

The most distinguishing feature of the cable stayed bridge
is the cables because the pattern chosen will define the overall
shape of the bridge. Numerous configurations have been used
that vary in both the longitudinal and transverse directions.

Historically, the longitudinal arrangement has been cate-
gorized into two main types, the fan and the harp. Most bridges
to date employ either one of these systems or a combination of
the two. Other systems such as the star pattern used in the de-
sign of the Norderelle Bridge (Fig. 3) and the unsymetrical
shapes shown in figure 4 have been successfully utilized to
enhance the appearance of the structure and to satisfy various
site requirements.

Section 1.2   Longitudinal Arrangements

The major criterior in determining the configuration of
the stays is the provision of sufficient stiffness to the main
bridge girder in an efficient and aesthetically pleasing manner.
From the engineering point of view, the fan shape is most ef-
ficient since it transfers the vertical load from the deck to
the towers with the minimum amount of steel and with the low-
est horizontal thrust to the girder. In addition, with all
the cables attached to the tower at one point, the fan type
has the advantage of requiring only one cable anchorage in the
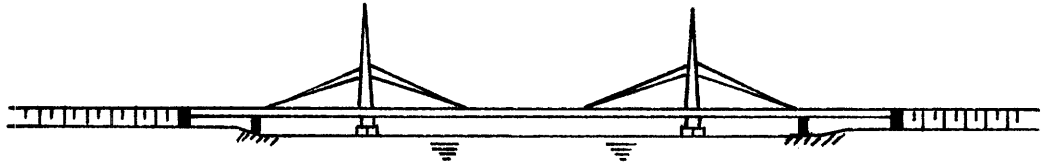
Norderelle Bridge
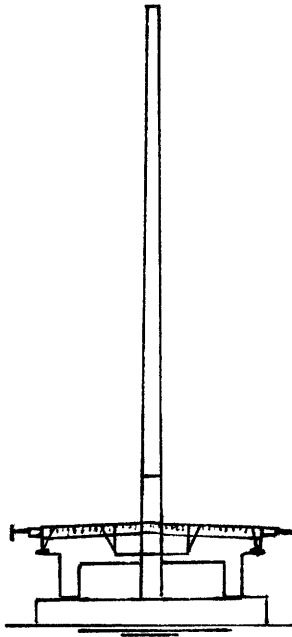
Tower Cross Section

Figure 3.   Star Configuration

Bratis Lava Bridge

Batman Bridge
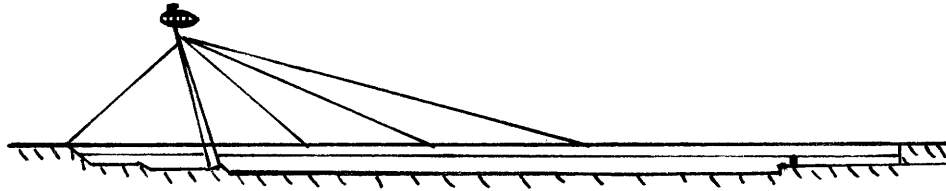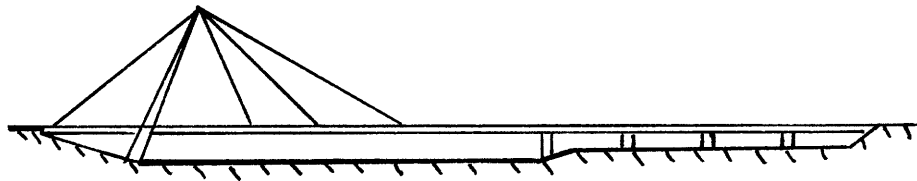
Figure 4. Unsymetrical Shapes

tower. Unfortunately, the fan type does not offer the best solution in terms of aesthetics. In the event that towers and cable supports are required on both sides of the deck, the cable lines will intersect when the bridge is viewed from most angles. This will detract a great deal from the appearance of the structure and has led many designers to lower the interior anchorages from the top of the tower.

The harp arrangement of the cables will generally be more aesthetically pleasing but has one major disadvantage. By anchoring the cables at different levels, large bending moments will be produced in the towers. These bending moments can be greatly reduced by allowing the cables horizontal movement at the tower connection. Due to friction at the anchorages and eccentricity of applied loads to the towers, the bending moment produced cannot be neglected. Also, by allowing movement of the cables the stiffness of the overall structure is reduced.

As previously mentioned, the stiffness of the structure is highly dependent on the stiffness of the cables but deficiencies in cable stiffness can be compensated for by various methods. One alternative is to fix the top cable on the side spans to abutments. This would greatly reduce the horizontal deflection of the tower and thereby increase the stiffness of the structure. It is appropriate to mention here that the major advantage of the cable stayed system over suspension bridges is that there is no requirement for massive foundations

to anchor the cables at each end of the span. Therefore, it
is very possible that site requirements may not allow anchor-
ing the top cable to side abutments. One excellent solution
is to anchor each of the cables on the side spans to piers as
was done for the Kniebrucke-Dusseldorf and Duisburg bridges
(Fig. 5).

Section 1.3   Transverse Arrangements

In the transverse direction there are three basic con-
figurations for the cables; the single plane, the double plane
and the A-frame. The single plane system has been used in many
recent designs such as the Leverkusen Bridge and the Papineau
Bridge in Montreal   (Fig. 6). This system offers great aes-
thetical advantages due to the overall impression of lightness
and the unobstructed view obtained by locating the towers in
the center of the deck. This arrangement is most suitable for
divided highways for which the required central meridian strip
is an excellent location for the towers and cable anchorages.
The major disadvantage of this system is that the cables do not
supply any rotational restraint to the deck and therefore the
deck must have high torsional rigidity in order to carry the
torsional moments induced by eccentric loadings. Due to this
requirement an excessive amount of material may be needed in
the deck structure.

The double plane system incorporated in the design of the
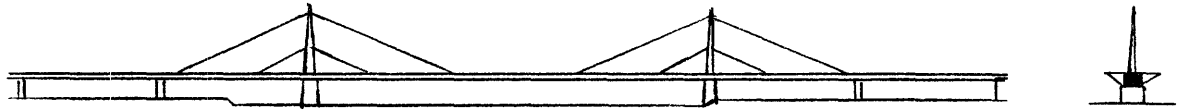Saint Florence Bridge in France (Fig. 7) has been most often
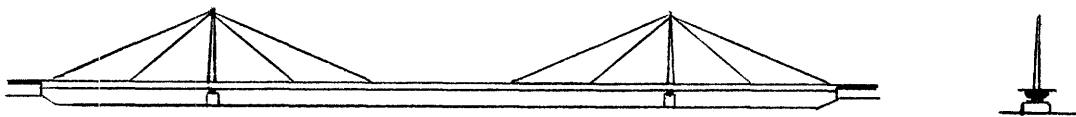
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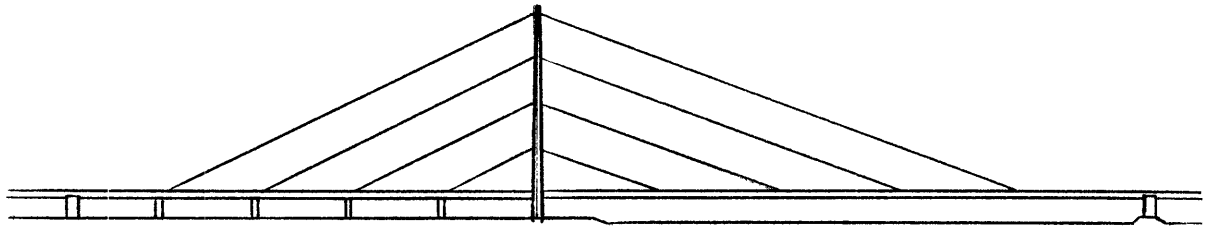
Kniebrucke-Dusseldorf Bridge

Duisburg Bridge

Figure 5.   Anchored Side Spans

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Leverkusen Bridge

Papineau Bridge

Figure 6. Single Plane Systems

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used because of its high torsional rigidity. Alternatively, for very long spans, such as required for the Cologne Bridge in Germany (Fig. 7), the A-frame is more appropriate because it supplies more torsional restraint to the deck and also has much more lateral resistance both from the cables and from the frame action in the towers.

The number of cables is a very important factor in determining the stiffness of the structure and therefore must be considered when determining the cable arrangement. It is sufficient to state here that a small number of cables will result in large cable forces transmitted to the towers and the deck which requires heavy and complicated anchorages and additional material in order to transmit the forces over the entire cross-section of the deck. A large number of cables will simplify anchorages and can be considered a continuous elastic support for the deck which will reduce resultant bending moments.

Section 1.4 Cable Properties

The efficiency of the cable stayed bridge is the major reason it has come into prominence in recent years. The shortage of steel in Germany after the war led to the rediscovery of the cable stayed system. A maximum utilization of steel and therefore a reduction in dead weight of the structure was needed to rebuild the bridges along the Rhine. The consequential increase in the live load to dead load ratio necessitated high fatigue strength for the structure, especially for the
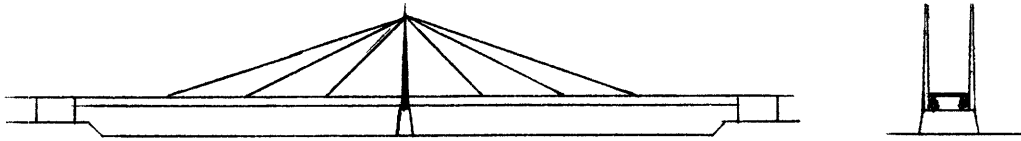
Saint-Florence Bridge
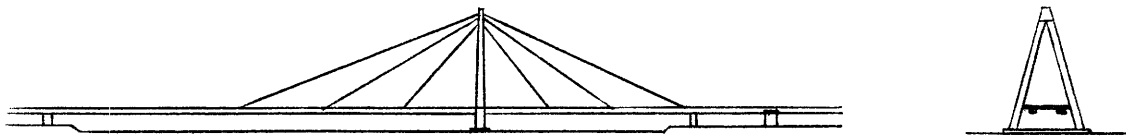
Cologne Bridge

Figure 7. Double Plane and A-Frame Systems

cables and cable anchorages. Many tests have been conducted
in Germany and the results are available in the edition of the
German Standard DIN 1073 and subsequent editions.

It is important to note that in a preliminary analysis
the configurations of the cables is not necessary but rather
the stiffness of the cables at the deck supports and tower sup-
ports must be estimated. The stiffness of the cables is not
only dependent on the modulus and area of the steel but also
the projected horizontal length of the cable and the applied
stress. The understanding of these relations is most important
and therefore the derivation is included in Appendix I. The
concept of an equivalent modulus for the cable is appropriate
for demonstrating the effects of various parameters on the
stiffness of the cables. Figure 8 has been constructed using
this approach in which:

$$E_{eff} = \frac{E}{1 + \frac{\gamma^2 L_p^2 E}{12 \sigma^3}}$$

where: E = Young's Modulus of the steel (30,000 ksi)

γ = specific weight of the cable (500 #/ft^3)

L_p = projected horizontal length of the cable.

σ = tensile stress in the cable.

It is obvious from this relationship that a high stress
state is needed in the cables to achieve a satisfactory effec-
tive modulus and consequentially a high stiffness. The ulti-
mate stress for cables can be as high as 220 ksi. With the
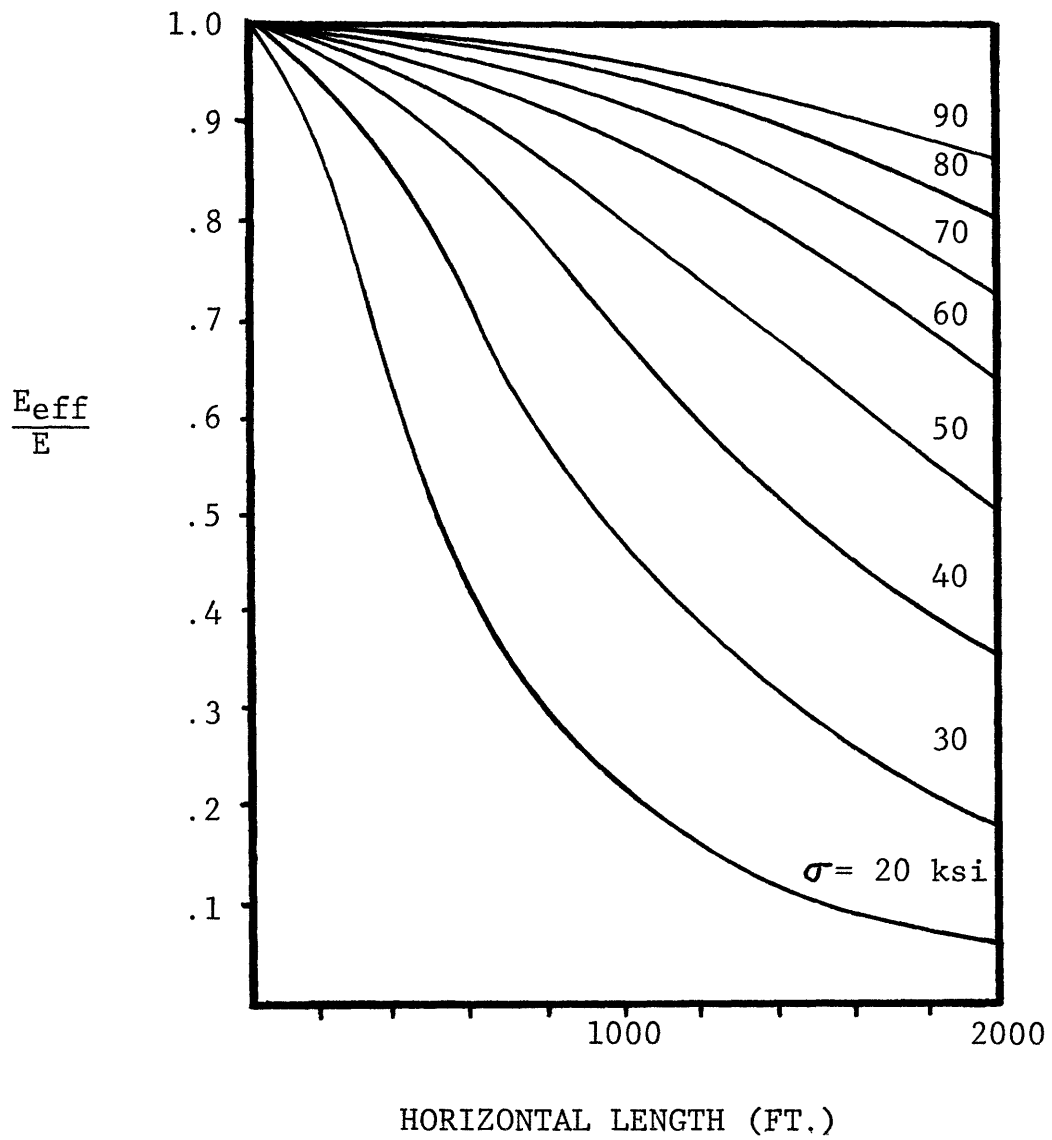
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Figure 8. Graph of Effective Modulus

appropriate factor of safety for cables of 2.5, the allowable stress is approximately 90 ksi. Unfortunately, this high stress state will be present only under critical design load conditions and therefore, under usual loadings, the effective cable modulus can be only a small fraction of the steel modulus. In the design of long span bridges, steps must be taken to reduce these effects.

Prestressing the cables can reduce this sag effect but has the disadvantage of producing larger bending moments and axial loads in the deck and towers which would increase the amount of required material in these areas. Appropriate construction techniques, to be discussed later, can induce a stress level in the cables, effectively producing satisfactory behavior. In extreme conditions it may be required to support the cables at intermediate locations by use of ropes or lightweight towers.

The purpose of the cables is to transmit the vertical loads from the deck to the towers. Therefore, the concept of effective modulus, which is independent of the cable inclination, may be misleading when determining cable effectiveness.

The actual vertical support provided by the cables to the main girder is,

$$k_y = \sin^2\theta \; (\frac{AE}{L})(\frac{1}{1+ \frac{\gamma^2 L_p^2 E}{12 \, \sigma^3}})$$

As can be seen in figure 9, the vertical support drops rapidly as the cable anchorages are moved out along the span. This is
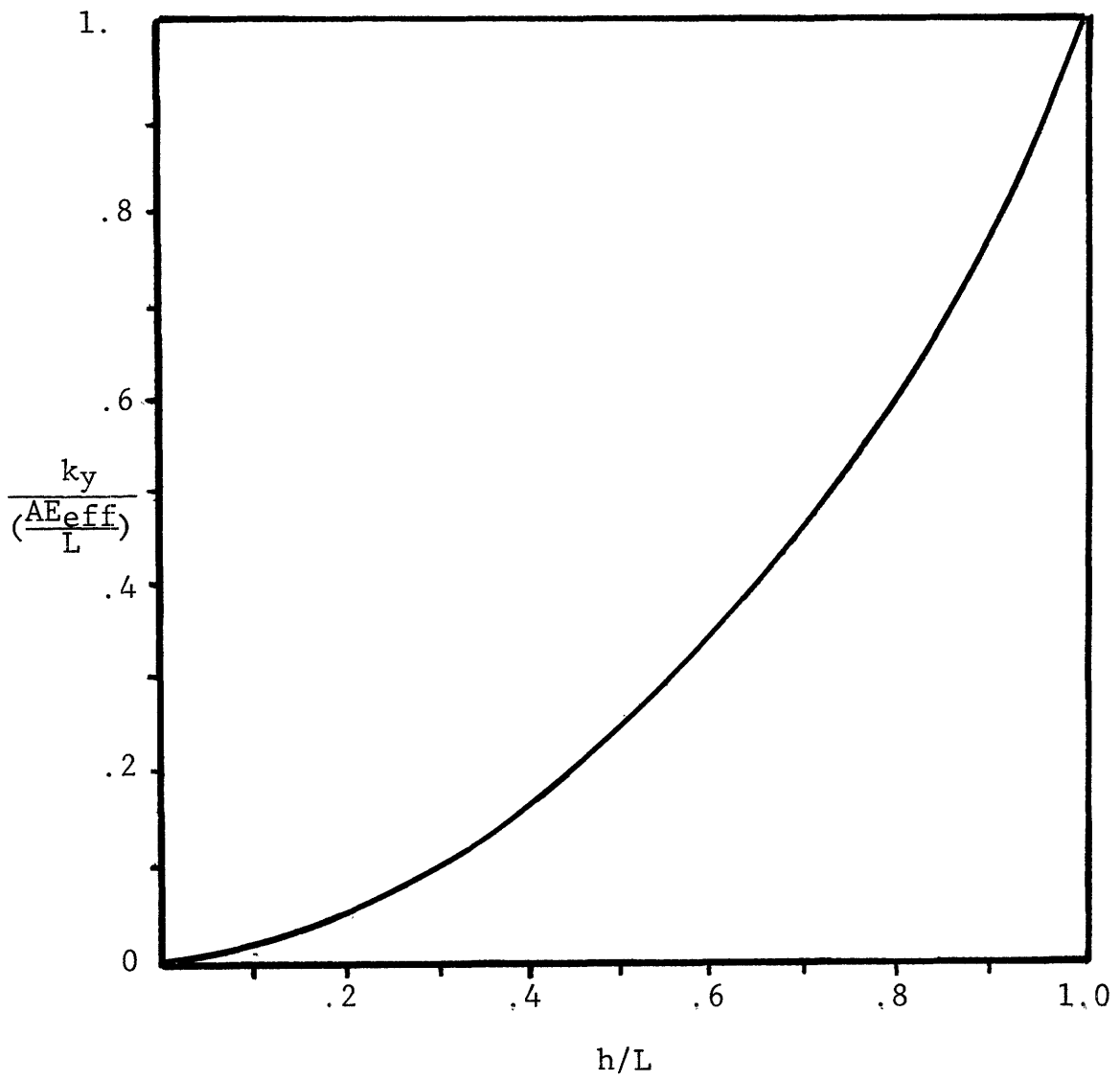
$\dfrac{k_y}{(\dfrac{AE_{eff}}{L})}$ vs h/L

Figure 9. Graph of Cable Vertical Stiffness

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the main reason that the cable to tower connections should be made as high on the tower as possible. In addition, the horizontal thrust transmitted to the girder also increase proportionately with h/L_p which suggests the higher cable-tower connection is desirable.

Chapter 2 - Main Girders

Section 2.1   Introduction

In the selection of the proper deck support system there are two basic alternatives:

a)   Truss Girder.
b)   Solid Web Girder.

In past years, truss girders were used extensively for long spans because of their low wind response and the ease with which member areas can be varied to produce an optimum design. However, streamlined solid web girders have proven to behave satisfactorily under wind loadings. In addition, other characteristics of the truss girders such as unfavorable visual appearance and rising fabrication, maintenance and corrosion protection costs have negated this alternative except for very special circumstances.

Section 2.2   General Characteristics

The continuous support that the cables provide along the deck allow for a much shallower and lighter deck support system as compared to other bridge designs. The total depth of the cable stayed bridge deck usually ranges from six to twelve feet for highway bridges and twelve to twenty feet for railroad bridges. The depth is almost independent of the length of the main span and is basically a function of the dead to live load ratio and the side to main span ratio. Experience gained from past designs indicates that for an optimum use of

cable stiffness, a side to main span ratio of approximately four tenths is desirable.

It has been the practice of many engineers to limit the deflection to span ratio on bridge designs. This may prove to be a harsh restriction for the cable stayed bridge system. Although the total deck deflection may be substantial under large distributed loadings, the cables are in a high stress state and therefore the stiffness of the structure is at a maximum. Due to the cable supports and the continuous main girder, the deflection under concentrated loads is much smaller for the cable stayed system than for other systems. Therefore, the important criterior for design is the change in slope of the deflection curve and resultant bending moments. The present limitations on deflection to span ratio may place unjust restrictions on design.

The main girder must carry large axial loads, transmitted by the cables, in addition to the vertical deck loadings. For all bridge designs to date, the axial load is compression which requires the non-linear beam-column effects be taken into account in the final analysis. In the preliminary design, it is sufficient to model the main girder as a beam on elastic supports.

Grimsing[8] has proposed that the deck be anchored at the ends of the side spans with the placement of expansion joints in appropriate positions in order to decrease the horizontal thrust transmitted to the girder. In this scheme, either

part or all of the main girder will be in tension   (Fig. 10).
Increased stiffness and material savings in the deck structure
are the major advantages of this system.   Because of the com-
plicated anchoring devices and expansion joints, this method
is uneconomical for most bridge designs.   Also, site conditions
may not permit large anchorages at the end supports.

Section 2.3   Solid Web Systems

A solid web system may be classified as either an open
section or a closed section.   The open sections may be twin
plate girders as used in the design of the Rees and Ludwig-
shafen Bridges   (Fig. 11), or they may be multiple plate
girders.   The disadvantage of open sections is the large amount
of transverse stiffeners required to support the plate girders
against load buckling and applied wind forces.   In addition,
an open section does not possess the high torsional resistance
needed for unsymmetrical live loadings.   Therefore, open sec-
tions can only be used in long span designs with a double or
A-frame cable system.

Rising fabrication, construction and maintenance costs
for plate girders have made the closed box sections a more
appropriate solution for long span deck support systems.   It
is no longer sufficient to design for minimum weight as the
most important criterior when other factors can have a much
more substantial effect on economy.   Although in many instances
the box section's required plate thickness is a result of local

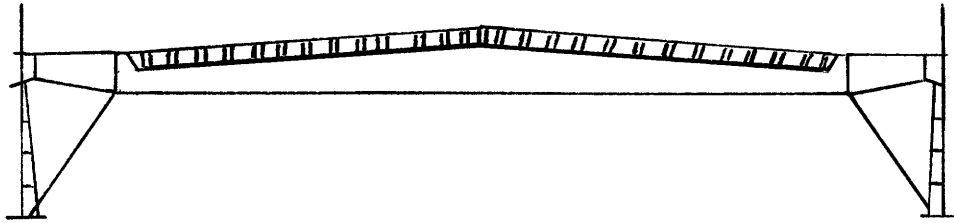expansion joint
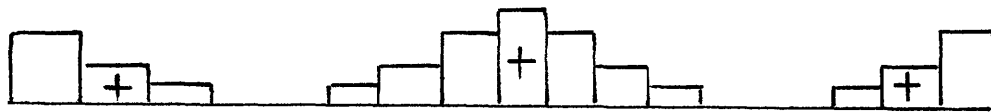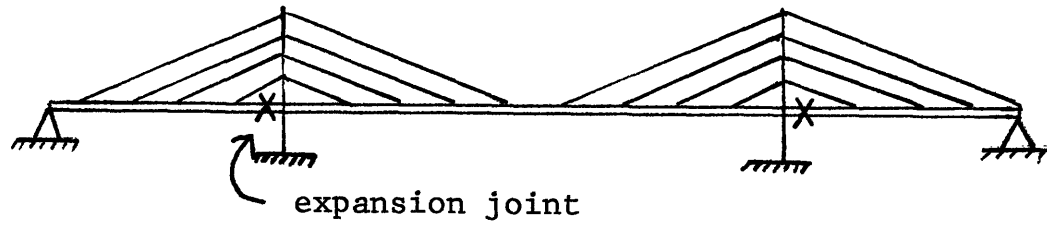
Fully Anchored System
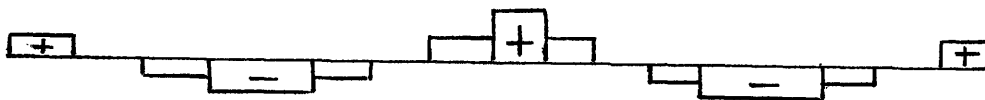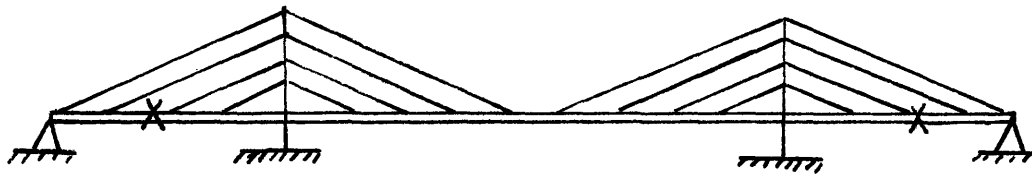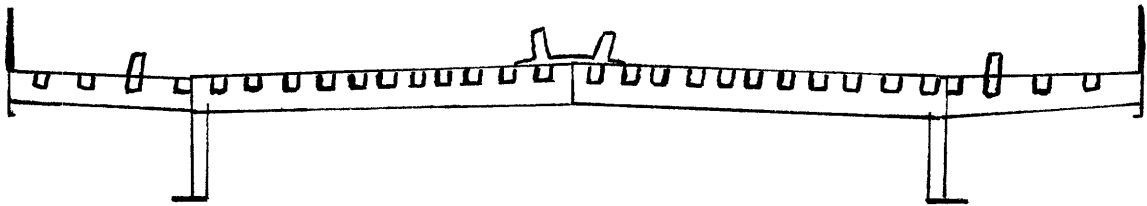
Axial Force in Deck

Partially Anchored System

Axial Force in Deck

Figure 10. Earth Anchored Systems

Rees Bridge (Deck Cross Section)

Ludwig Shafen Bridge (Deck Cross Section)

Figure 11.   Open Deck Sections

buckling and corrosion protection problems, there are also
many advantages to this system. Construction costs can be de-
creased due to less welding and the larger spacing of trans-
verse stiffeners. Maintenance is also much easier and a
minimum amount of corrosion protection treatment is needed
on the inside of the section. Closed sections have higher
torsional rigity and utilize the deck in the transmission of
bending and torsional moments more effectively.

Closed sections may be of the single box type used for
the Norderelle Bridge in Hamburg (Fig. 12), or the double box
type as used in the design of the Severins Bridge in Cologne
(Fig. 12). A trapazoidal section, such as proposed for the
Southern Crossing Bridge across the San Francisco Bay (Fig.13),
is often used to reduce the bottom flange area while support
to the deck remains at the optimum position. Fabrication
costs of these sections are higher than the single rectangu-
lar box section, but overall economy may be achieved by de-
creasing the amount of material.

Concrete sections are a viable alternative although
they are much heavier than their steel counterparts and there-
fore are not often used in long span bridge design. There has
been much recent research done on steel deck plates. Behavior
of this structural component has proven most satisfactory both
in tests and subsequent bridge designs. The orthotropic plate
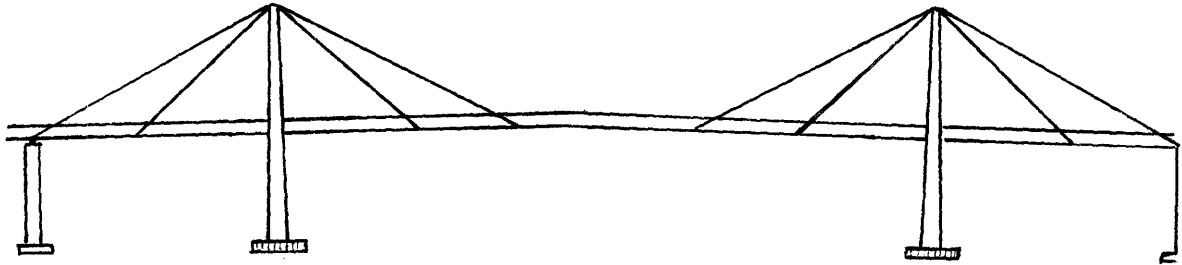deck, as it is called, is discussed in the following section.
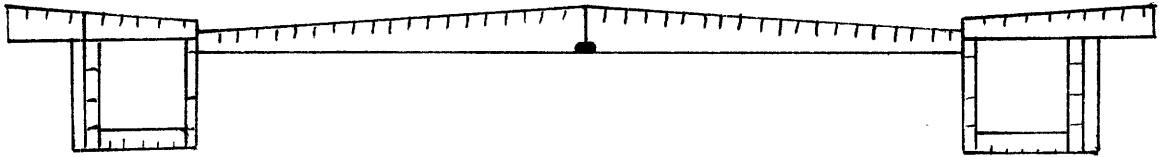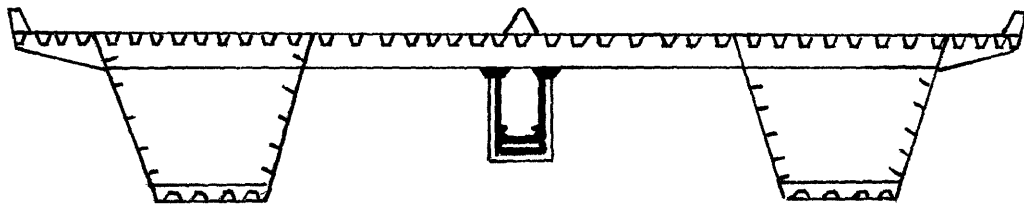
Severins Bridge (Deck Cross Section)
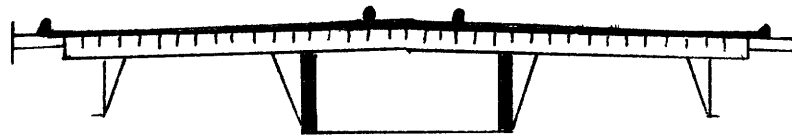
Norderelle Bridge (Deck Cross Section)

Figure 12. Box Sections

Southern Crossing Bridge

Deck Cross Section

Figure 13. Trapazoidal Section

The low weight, increased flexibility and lower damping inherent in these steel deck systems, increase the susceptibility to aerodynamic excitation. The varying lengths and stiffnesses of the cables serve as high dampers in the cable stayed bridge response and, as discussed previously, the stiffness of the entire structure may be increased by various methods. Recent aerodynamic tests show that the best stability under wind forces is obtained by streamlining the deck structure. This solution has been successfully used in many designs including the Kniebrucke-Dusseldorf and Duisburg Bridges (Fig. 14). Also, the long slender decks have a large separation of natural frequencies of the bending and torsional modes thereby preventing simultaneous excitation of these modes. Consequently, the overall dynamic response is greatly reduced.

Section 2.4   Deck Designs

A concrete deck may be used in a composite design and the cable stayed system offers the possibility of using the concrete more efficiently without excessive additional costs. In the design of a cable-stayed bridge it is essential to have a continuous main girder. By appropriately adjusting the stress in the cables, the deck supports may be positioned so as to utilize the composite section along the total bridge span. The horizontal thrust transmitted to the deck by the cables will raise the neutral axis in the areas of negative moment. This will also serve to minimize the amount of ineffective
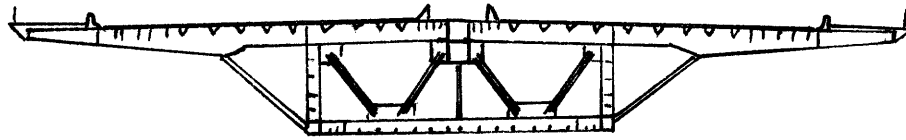
Kniebrucke-Dusseldorf Bridge
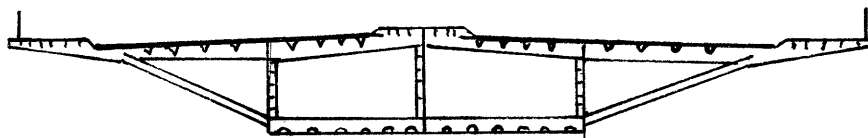
Duisburg Bridge

Figure 14.   Streamlined Sections

material. The additional weight of the concrete may be excessive under certain circumstances, and it may be preferable to use an orthotropic steel deck.

The orthotropic steel deck consists of a steel plate stiffened longitudinally and transversely which produces different rigidities in the two perpendicular directions. Used in the design of a main bridge girder the deck serves three main functions:

    a) A transverse beam to transmit deck loads to the main girder.

    b) The upper chord or flange main girder to transmit longitudinal bending moments.

    c) The web of a horizontal plate girder to resist transverse wind loadings.

As opposed to girder and suspension bridges, the weight of the cable-stayed bridge deck does not increase as a function of the main span. With the efficient use of the orthotropic plate system, the weight of the deck can be kept below seventy five pounds per square foot as seen in the following table.

Weight of Orthotropic Steel Decks.

Bridge	Main Span (ft)	Weight of Steel ($\#/ft^2$)
Bonn Nord	918	69
Duisburg-Neuenkamp	1150	62
Leverkusen	918	70
Ludwigshaven	900 (equivalent)	70
Rees	840	75

Due to the minimum thickness requirement of the ortho-
tropic steel deck acting as a transverse beam, no additional
material is needed to carry the horizontal thrust, transmitted
by the cables, for spans at least up to one thousand feet.
The low weight of the orthotropic deck and the ease of con-
struction are also major advantages of this system.

There are two basic methods used to analyze the ortho-
tropic deck;

      a)   The equivalent orthotropic slab.

      b)   The equivalent grid

The orthotropic slab method consists of distributing
the ridigities of the longitudinal and transverse stiffeners
uniformally over the deck. This procedure transforms the deck
into an equivalent orthotropic slab. The determination of the
stresses in the plate and stiffeners is very laborous due to
the complicated boundary conditions and therefore the proce-
dure has been transformed by Pelikan and Esslinger[8] into a
series of tables and charts that substantially reduce the
amount of work required for solution.

The equivalent grid method is similar in that the rigi-
dity of the orthotropic plate is transformed into a grid of
one-dimensional bars with the appropriate properties. The
standard slope deflection method is used in analysis and, com-
bined with the large number of grid components, the use of a
computer is necessary. This method is most suitable for

unsymmetric live loadings and transverse loading where there exists a significant stress variation across the section.

Chapter 3 - Towers & Cable Anchorages

Section 3.1   Introduction

The towers of a cable-stayed bridge must carry large axial loads and, depending on support conditions, large bending moments. Steel box sections have been used most often although the compression capacity of concrete makes reinforced concrete towers most efficient especially for long span bridge design.

The design of the cable anchorages to the towers and to the deck are a function of the forces in the cables, the location of the anchorages with respect to the centerline of the towers, and the distance out from the main girder. The anchorages may either fix the cables or, in the tower, they may be of the "saddle" type which allows horizontal movement of the cables. For fixed anchorages the cables are usually passed through a steel duct and encased in concrete. The saddle type usually consists of fabricated plates in the form of an arc over which the cables pass. The radius of the arc is dependent of the magnitude of the cable forces since there must be a suitable bearing area. Also, the radius must be large enough so that excessive bending moment stresses do not develop in the upper fibers of the cables.

Section 3.2   Tower Designs

The various types of possible tower arrangements are dependent on the cable configuration. As discussed earlier,

the towers may be single plane, double plane or A-frame. In
the case of the double plane it is usually not required to
incorporate a cross beam at the top of the towers. For most
tower support conditions the cables will act as restraints for
transverse deflection of the towers. If the towers are pin
connected at the deck level it is necessary to locate the cable
anchorages at the deck above the level of the tower hinge in
order to obtain transverse tower restraint from the cables.

Due to the large compression force and fluctuating bend-
ing moments a box section with a large width is desirable in
order to eliminate the need for excessive material to resist
buckling. Most designs to date have been rectangular boxes
constructed with thick steel plates. Since steel plates are
susceptable to local buckling under large compression forces,
additional stiffening may be required in the interior of the
towers. Reinforced concrete towers may be used instead of
steel to reduce the local buckling effect and to take advan-
tage of the material's economy in design for large compression
forces. The savings in the design of concrete towers is more
pronounced for long span bridges where the compression forces
are very large under the dead weight and the moments induced
in the towers under live loads will not produce tensile stresses.

For optimum design it is preferable to have the cable
forces applied down the centerline of the towers. In prelim-
inary design the forces acting on the towers should be resul-
tants of distributed dead loads plus live loads along the

length of the span. Under this loading condition the forces
should be distributed to the towers in a manner that produces
no bending moments.

The tower heights are dependent on the length of the
main span. Height to main span ratios of one to five or six
have proven most economical since the cable forces and hori-
zontal thrust transmitted to the main girder are also func-
tions of the height to span ratio. If concrete decks are used
it may be preferable to lower the height of the towers in
order to utilize the compressive strength in the deck. This
will also serve to increase the applied bending moment to the
towers and therefore may increase the width of the box section.
Whenever reinforced concrete sections are used in the design
of a cable-stayed system, the compression forces are very
large and the effects of shrinkage and especially creep can
be much larger than for conventional structures. The stati-
cally indeterminate forces add to the complexity of analysis
but must be considered in the final design. A method for
analyzing shrinkage and creep effects for the cable-stayed
system has been proposed by Akae, Murater and Kurite.[1]

Section 3.3   Tower Supports

The majority of the existing cable-stayed bridges have
been built with towers fixed at their base. In these designs,
large bending moments are produced in the towers due to un-
symetrical live loadings. The advantage of this system is

the increased rigidity of the structure which usually is more crucial than the additional material needed at the base of the tower. A method for designing these towers has been proposed by Kloffel, Esslinger, and Kollmeier.[14] This method is most suitable for fan-type arrangements where the cables converge at the top of the tower. The Harp-type arrangements are highly indeterminate and therefore a computer analysis is needed, taking into account the non-linear beam-column effects.

It is also possible to hinge the towers at the deck level. This alternative is most suitable for shorter spans since there is a large loss in rigidity and the deflections are substantially increased. Hinging the tower at the base or at the deck will greatly reduce the applied bending moments and therefore reduce the cross-section of the tower. The loss in rigidity is more pronounced for long-span bridges and therefore may not be suitable under certain conditions. Site conditions may not allow the tower foundations to resist bending moments and may require a hinged base.

Site conditions and design criterior are most important when the tower connections are being selected. The overall behavior of the bridge is highly dependent on the choice of tower connections. In the design of the proposed Southern Crossing Bridge in California, seismic design criterior necessitated innovation in the tower connection designs. The West Tower is supported by a rocker and buffer system which offers no restraint under slow temperature movement but locks

and acts as a hinge under sudden wind and seismic loadings.

Section 3.4   Deck Anchorages

As mentioned previously, it is desirable to have no applied bending moments to the towers under distributed loadings. In order to satisfy this condition cable planes must intersect along the centerline of the towers.

The cable anchorages usually require the most detailed analysis. Due to the large forces applied at points to the deck, there is a complicated stress flow in these areas. The most suitable method of analysis is the Finite Element Method for which there are many computer programs available.

The deck anchorages often consist of transverse inclined beams which span the width of the deck cross-section. This system is used to transfer the horizontal thrust to the girder more efficiently.

Since it is desirable to have the plane of the cables coincide with the towers, the cable anchorages will not always connect directly to the web of the main girder. In these instances, large shear forces and bending moments must be transmitted some distance to the main girder. Therefore, to minimize the cantilever moment, it is necessary to locate the cable planes and towers as close to the main girder web plates as possible.

As an alternative to the transverse, inclined beam anchorages,is the construction of longitudinal shear plates.

Individual cable strands may be symmetrically attached to the plates resulting in the need to transmit only shear forces.

The complexity of the cable anchorage design is greatly reduced by including a large number of cables in the structure. The magnitude of the forces transferred by the anchorages is mainly dependent on the number of cables.

Chapter 4 - Construction Techniques

Section 4.1   Introduction

Construction and fabrication costs are highly significant and are constantly becoming a higher percentage of the overall cost of the structure. Prefabrication of large sections of the structure can reduce the cost by minimizing the erection of small sections in exposed conditions which are costly and weather dependent. The most commonly used construction procedure for cable-stayed bridges, is the cantilever method. Using this method, the deck spans between cable supports can be set into position and may not require temporary erection guys. During construction, the bridge components may be subjected to higher stresses than under design loading conditions. In addition, the deflections during construction will be much larger, thereby producing substantial non-linear effects. Consequently, it is important to analyze the structure at every stage of construction.
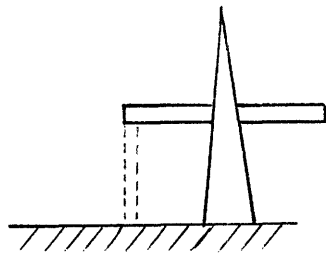
Section 4.2   Balanced Cantilever Method

During construction it is usually required or most desirable to leave the area under the main span free and clear of obstruction. Although it is possible to build temporary piers to support the deck on the side spans, temporary erection guys may be needed to support the main span. In general, falsework can be very expensive and therefore has motivated

engineers to study alternative methods. In cable-stayed
bridge design, the continuous deck may be sufficiently stiff
to cantilever the span between cable supports. In order not
to apply excessive unbalanced forces to the towers, it is de-
sirable to simultaneously cantilever the deck on both sides
of the tower. This method has been given the name "Balanced
Cantilever Method", and is shown in figure 15.

Stage one consists of cantilevering the first deck sec-
tions on either side of the towers. Temporary or permanent
piers are constructed on the side span. In stage two the
first cables are put into place and tensioned in order to de-
crease the applied moment to the tower at the deck connection.
The next deck sections are cantilevered out from the cable
supports in stage three and finally in stage four the next
cables are put into place and tensioned. This procedure is
continued until all cables are into place and the final stage
is the placement of the central section in the main span.

In the analysis of this procedure the non-linear ef-
fects of the cable sag and the beam-column behavior must be
taken into account. This involves an iterative process either
of modifying stiffnesses or applying imaginary loads until con-
vergence. These analyses can be very costly in computer time
therefore, economical techniques must be produced. The time
necessary to converge on the solution is highly dependent on
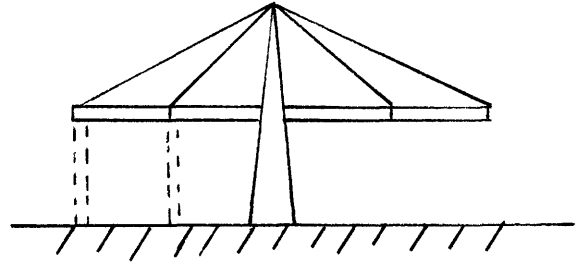the initial assumptions of the analysis. In order to obtain
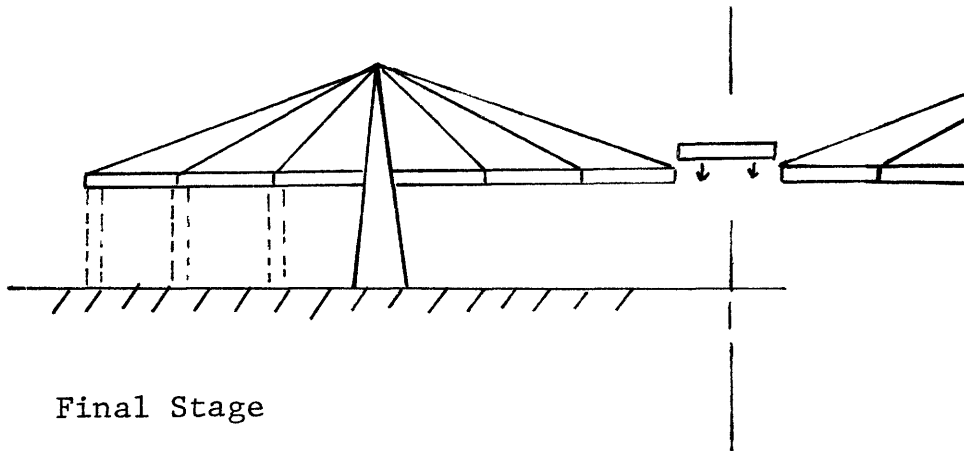
Stage 1
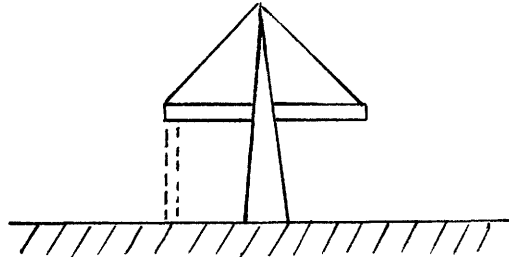
Stage 2
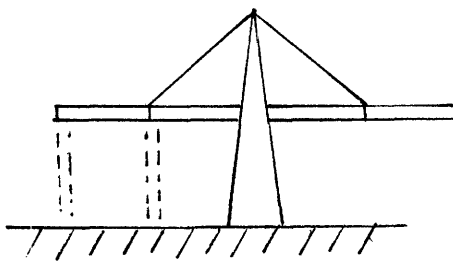
Stage 3

Stage 4

Final Stage

Figure 15. Balanced Cantilever Method

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accurate approximations for the displacements or stiffnesses, as required by the particular solution method, the results of a previous analysis may be used. As an example, suppose that the analysis for stage two in figure 15 has been completed and now the analysis of stage three is needed. If the particular solution technique requires the convergence of the cable, deck and tower stiffnesses, the quantities obtained from the analysis for stage two with appropriate alterations for the newly applied loads, may be used as input for the required analysis. The method can be repeated at each stage and will significantly reduce the computation time.

Section 4.3   Erection Procedure to Reduce Beam-Column Effect

Since the deck in a cable-stayed system is designed as a continuous girder on elastic supports, the stresses induced by cantilevering the deck during construction may be higher than the stresses in the final structure. During erection the forces due to dead load will be substantially larger than the dead load of the entire structure. In some instances, these forces may be larger than the dead load plus live load forces anticipated for the life of the structure. It would obviously be uneconomical if a large part of the deck had to be designed for stresses occuring only during constructions, therefore, the method shown in figure 16 has been proposed. In this method, the cantilevered deck sections will still have to resist the applied bending moments. The important fact is
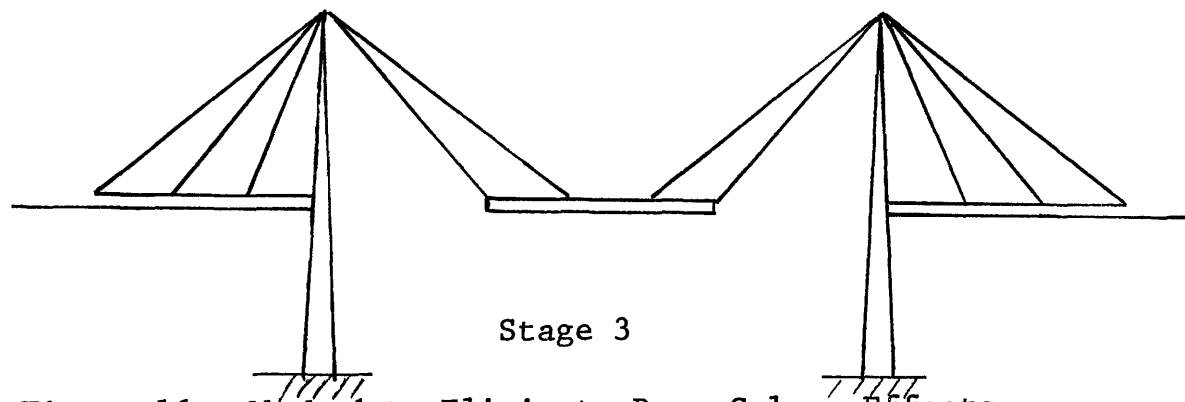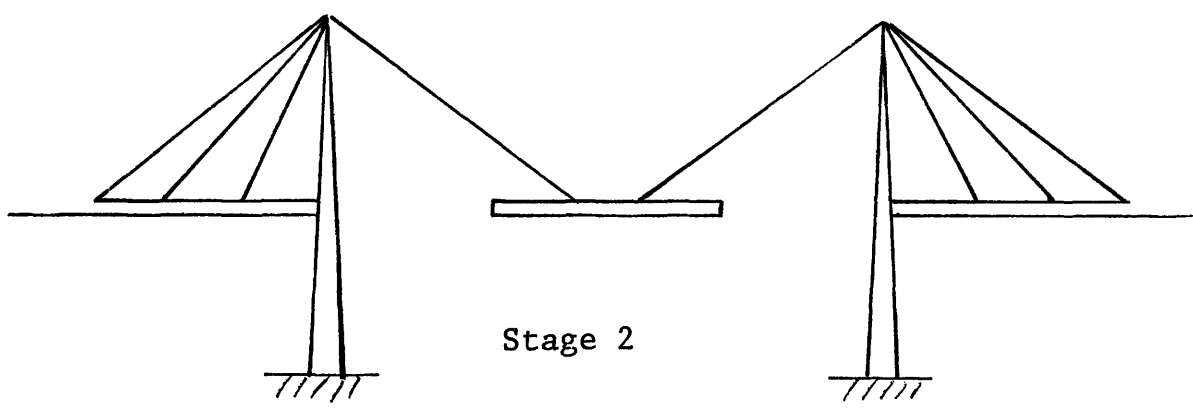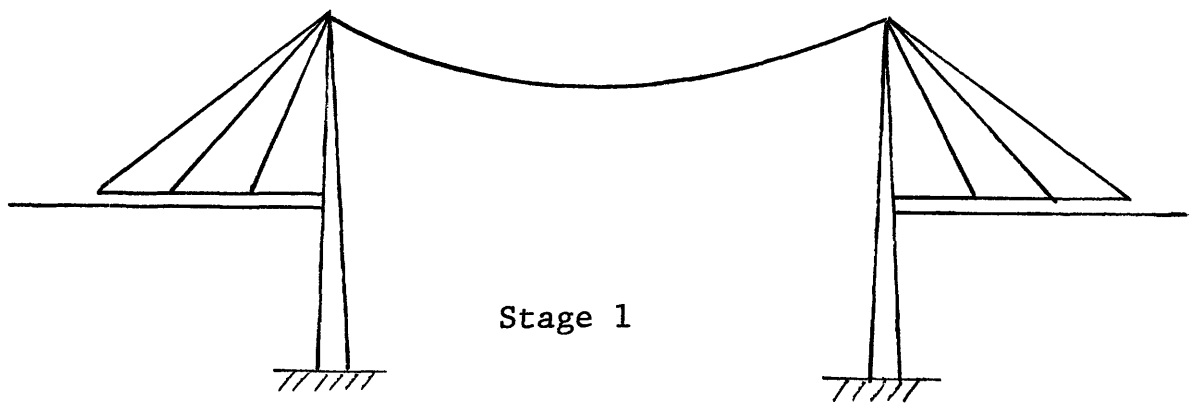
Stage 1

Stage 2

Stage 3

Figure 16. Method to Eliminate Beam-Column Effects

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that the constructed deck sections will have a tensile axial force. This eliminates the need for beam-column consideration and therefore there will be no additional moments due to this effect. Consequently, the constructed deck sections will be resisting only moments induced by the construction apparatus, which will be less than the live load, and the tensile forces transmitted from the cables. This procedure would be especially suited to the construction of an anchored bridge, discussed in Chapter Two, as the applied axial forces during erection are the same sign as applied to the final structure.

Section 4.4   Examples

The majority of the cable-stayed bridges have been built by the free cantilever method. This method usually requires temporary or permanent piers to be constructed on the side spans before the erection of the main span can begin. Bridges that are designed with many stays such as the Rees Bridge and the Bonn-Nord Bridge (Figure 17) do not usually require many temporary stays during erection. In fact, there were no temporary stays needed in the construction of the Bonn-Nord Bridge and only one temporary stay was needed, between the tower and the first cable support, during the construction of the Rees Bridge. Specific site requirements may not allow for the construction of temporary piers on the side spans. This was the case in the construction of the Julicher Strasse Bridge in Dusseldorf.

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Rees Bridge
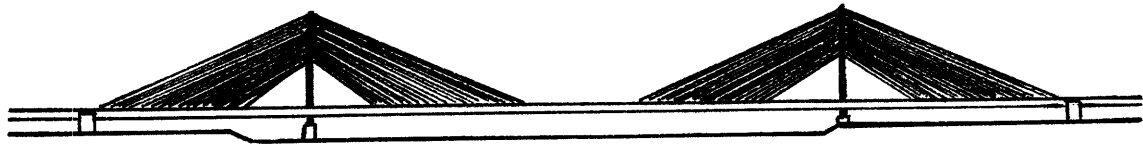
Bonn-Nord Bridge

Figure 17. Multiple Cable Arrangements

The Julicher Strasse Bridge spans approximately 325 feet over railroad tracks. Particular care had to be taken during construction so as not to interfere with the railway traffic. The solution was to assemble the entire superstructure on the side of the site and to jack it into place. The construction sequence is shown in figure 18 where it can be seen that the use of temporary piers under the main span was required. Additional savings were made by accomplishing most of the painting and welding in the shop.

One of the most interesting construction techniques was used in the erection of the Buchenauer Bridge, which was the first cable-stayed bridge built in Germany. This bridge was to replace an existing girder bridge spanning railroad tracks. Again, no temporary piers could be used due to the obstruction of railway traffic. The solution was to make use of the existing girder bridge during construction. In the first stage the cross girders and main girders were suspended from the old bridge. Next, the superstructure was hoisted into place, then the cables were attacked and tensioned. Now the cables are supporting the deck so the old bridge was lowered to the new deck and disassembled. Finally, the concrete deck was poured.
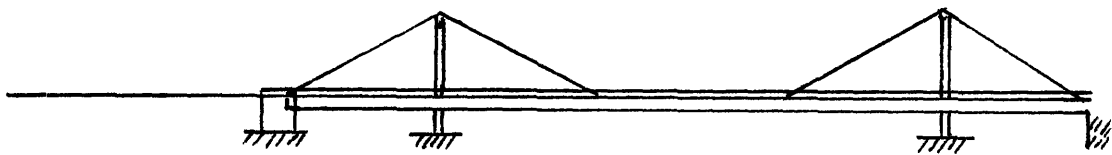
Stage 1
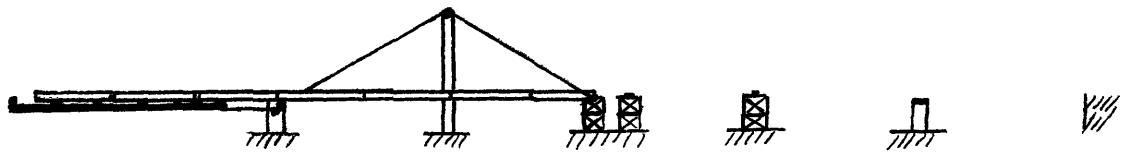
Stage 2
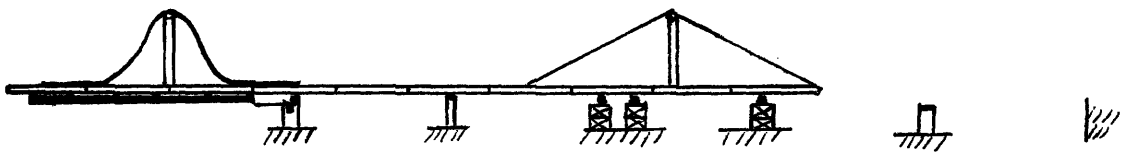
Stage 3

Stage 4

Figure 18. Erection of Julicher Strasse Bridge

Conclusions

The application of the cable-stayed system as a bridge design has come into prominence only in recent years. Many of the problems that arise in the design are associated with the major advantages of the system. The multiple cable supports and the continuous deck constitute a high degree of indetermanency which can lead to a very laborous analysis. With slight modifications, to include the non-linear cable stiffnesses and beam-column effects, existing computer programs for structural analysis may be used. The material and construction savings far outweigh the disadvantages. Also, the aerodynamic problems associated with long span bridge design are minimized by the use of the shallow, streamlined deck.

It is essential that the engineer recognizes the interdependence of the structural components and the behavior of the entire system. The behavior of the overall structure is dependent on the stiffnesses of the individual components. A change in design of the towers, cables or deck may significantly effect the response of all three of the major bridge components. Therefore, it is most important that all options are considered in the design.

The study of cable-stayed bridges is an excellent field for the application of innovative techniques in design, detailing and construction. The practical bridge designer must

first satisfy the functional requirements and, on this basis, produce an aesthetically pleasing structure.

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List of Figures

Appendix A

List of Terms

A . cross-sectional area of the cable

E . Young's Modulus

E_{eff} . Effective Modulus

h . height of cable

h_m . maximum sag in the cable

H . horizontal force component in the cable

k . cable stiffness

k_y . vertical component of cable stiffness

L . chord length of the cable

L_p . projected horizontal length of the cable

S . actual length of the cable

S_o . unstressed length of the cable

T . force in the cable

V . vertical force component in the cable

δ_x . horizontal displacement of the cable

δ_t . displacement in the direction of the cable

θ . angle of cable inclination

σ . stress in the cable

Derivation of Cable Stiffness.

For a uniformly loaded cable we have:
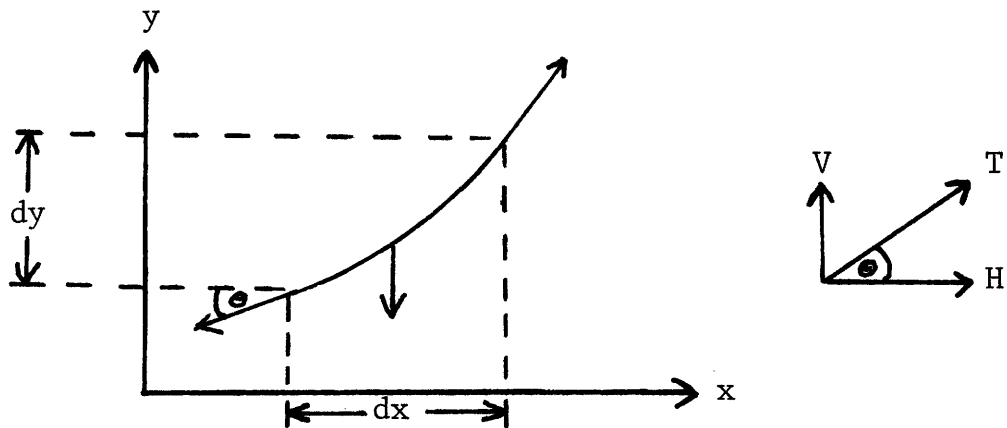
Slope = $dy/dx = \tan \theta$

From equilibrium:

$$T \cos \theta = H = \text{constant} \tag{1A}$$

$$d/dx \ (T \sin \theta) = W \tag{1B}$$
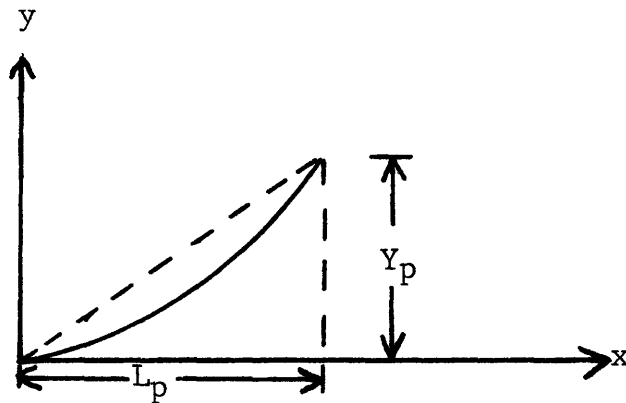
Now,

$$T \sin \theta = V = H \tan \theta = H \ ^{dy}/dx$$

Therefore (1B) reduces to:

$$d^2y/dx^2 = \frac{W}{H} = \text{constant} \tag{2}$$

Integrating,

$$Y = \frac{W}{H} \ (x^2/2 + C_1 x + C_2) \tag{3}$$

Locating the left end of the cable at the origin:

at $x = 0$, $y = 0$ therefore $C_2 = 0$

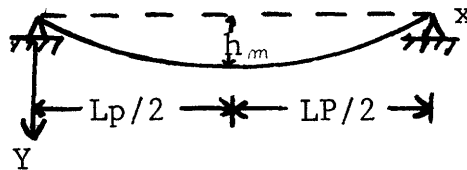at $x = L_p$, $Y = Y_p$ therefore $C_1 = \dfrac{Y_p H}{W L_p} - \dfrac{L_p}{2}$

Substituting into (3),

$$Y = \frac{W}{2H} (x^2 - x L_p) + x\left(\frac{Y_p}{L_p}\right)$$

Finally, using $\text{Tan}\ \theta = \dfrac{Y_p}{L_p}$

$$Y = \frac{W}{2H} (x^2 - x L_p) + x \tan \theta \qquad (4)$$

In order to simplify calculations, the following expressions will be derived for a shallow, horizontal cable and later will be transformed for a shallow inclined cable.

Using (4) with $\tan \theta = 0$,

$$Y = \frac{W}{2H} (x L_p - x^2) \qquad (5)$$

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Now,

$$ds = (1 + (\frac{dy}{dx})^2)^{\frac{1}{2}} dx$$

Expanding the square root and dropping insignificant terms,

$$ds \cong dx(1 + \tfrac{1}{2}(\frac{dy}{dx})^2) \tag{6}$$

From (5),

$$\frac{dy}{dx} = \frac{W}{2H} (Lp - 2x)$$

Substituting into (6),

$$ds \cong dx(1 + \frac{W^2}{8H^2}(Lp^2 - 4xLp + 4x^2))$$

Integrating,

$$S \cong \int_0^{Lp} ds = L(1 + \frac{W^2 Lp^2}{24H^2}) \tag{7}$$

The term $\dfrac{W^2 Lp^2}{24H^2}$ is actually $\dfrac{8}{3}(\dfrac{hm}{L})^2$ and therefore is small with respect to unity since the cable is shallow.

Define: S_o = unstressed length of the cable.

E = Young's Modulus of the cable

A = cross-sectional area of the cable.

Therefore, the strain is,

$$\bar{e} = \frac{ds - dS_o}{dS_o} = \frac{I}{AE} \tag{8}$$

For a horizontal cable, $T \approx H$

Therefore, integrating (8),

$$S - S_o = \frac{H}{AE} S_o$$

or $S = S_o(1 + H/AE)$ $\tag{9}$

Equating (7) and (9),

$$S_O(1 + \frac{H}{AE}) = L_p(1 + \frac{W^2 L_p^2}{24H^2}) \tag{10}$$

To obtain the stiffness, induce an elongation to the cable. Using i to denote initial state, equation (10) becomes,

$$S_O(1 + \frac{H_i}{AE}) = L_p(1 + \frac{W^2 L_p^2}{24H^2}) \tag{11}$$

With the elongation δ_x, equation (10) becomes,

$$S_O(1 + \frac{H}{AE}) = (L_p + \delta_x)(1 + \frac{W^2(L_p + \delta_x)^2}{24H^2}$$

or $\quad S_O(1 + \frac{H}{AE}) = L_p(1 + \frac{\delta_x}{L_p})(1 + \frac{W^2 L_p^2}{24H^2}(1 + \frac{\delta_x}{L_p})^2) \tag{12}$

The terms H/AE, $\frac{\delta_x}{L_p}$ and $\frac{W^2 L_p^2}{24H^2}$ are small with respect to unity.

From (11),

$$S_O = L_p(\frac{1 + \frac{W^2 L_p^2}{24H_i^2}}{1 + H_i/AE}) \tag{13}$$

Substituting (13) in (12),

$$\frac{1 + \frac{H}{AE}}{1 + \frac{H_i}{AE}}(1 + \frac{W^2 L_p^2}{24H_i^2}) = (1 + \frac{\delta_x}{L_p})(1 + \frac{W^2 L_p^2}{24H^2}(1 + \frac{\delta_x}{L_p})^2) \tag{14}$$

Neglecting products of small terms, (14) becomes:

$$\frac{H}{AE} + \frac{W^2 L_p^2}{24H_i^2} = \frac{\delta_x}{L_p} + \frac{W^2 L_p^2}{24H^2} + \frac{H_i}{AE} \tag{15}$$

Solving for $\frac{\delta_\chi}{Lp}$, we have

$$\frac{\delta_\chi}{Lp} = \frac{H - H_i}{AE} + \frac{W^2 Lp^2}{24}\left(\frac{1}{H_i^2} - \frac{1}{H^2}\right)$$

or $\qquad \frac{\delta_\chi}{Lp} = \frac{H - H_i}{AE} + \frac{W^2 Lp^2}{24 H_i^2}\left(1 - \frac{1}{(H/H_i)^2}\right) \qquad\qquad (16)$
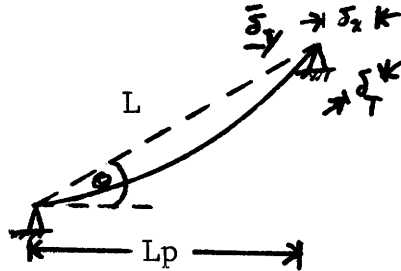
differentiating,

$$\frac{d\delta_\chi}{dH} = \frac{Lp}{AE}\left(1 + \frac{AE}{H^3}\left(\frac{W^2 Lp^2}{12}\right)\right)$$

Therefore,

$$\frac{dH}{d\delta_\chi} = \text{stiffness} = \frac{AE}{Lp}\left(\frac{1}{1 + \frac{AE}{H^3}\left(\frac{W^2 Lp^2}{12}\right)}\right) \qquad\qquad (17)$$

For our case of a shallow inclined cable,

Lp goes to L where L = Lp sec θ

T goes to H

W goes to γ A cos θ

δ_χ goes to δ_T

Where γ is the specific weight

Therefore, substituting into (17)

$$\frac{dT}{d\delta_T} = \frac{AE}{L}\left(\frac{1}{1 + \frac{AE}{T^3}\left(\frac{\gamma^2 A^2 \cos^2\theta\, Lp^2 \sec^2\theta}{12}\right)}\right)$$
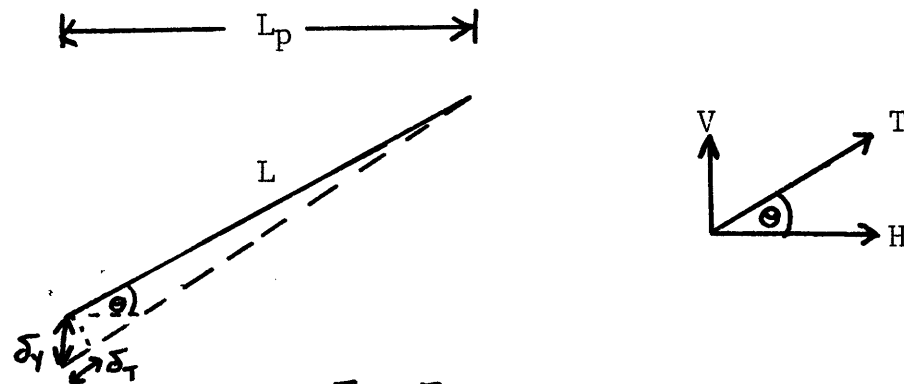
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or $\dfrac{dT}{d\delta_T} = \dfrac{AE}{L} \left(\dfrac{1}{1 + \dfrac{\gamma^2 Lp^2 E}{12\sigma^3}} \right)$ (18)

Where $\sigma^3 = \dfrac{T^3}{A^3}$ = tensile stress in cable.

In order to obtain the vertical support that the cables provide to the deck, define the following parameters:

k = stiffness of the cable

k_y = vertical component of the cable stiffness.

$T = k\delta_T \qquad\qquad \delta_T = \delta_y \sin\theta$

$V = T\sin\theta = k\,\delta_y \sin^2\theta$

Therefore,

$k_y = \sin^2\theta (k)$ (19)

From (18)

$k = (\cos\theta)\dfrac{AE}{Lp}\left(\dfrac{1}{1 + \dfrac{\gamma^2 Lp^2 E}{12\sigma^3}} \right)$ (20)

Substituting (20) into (19),

$$k_y = \cos \Theta \, \sin^2 \Theta \, (\frac{AE}{LP})(\, \frac{1}{1 + \frac{\gamma^2 L_p{}^2 E}{12\sigma^3}} \,) \qquad (21)$$

Equation (21) is the stiffness used for the elastic supports in the preliminary design of the main girder.

The axial load (H) transmitted to the girder is,

$$H = \tan \Theta$$