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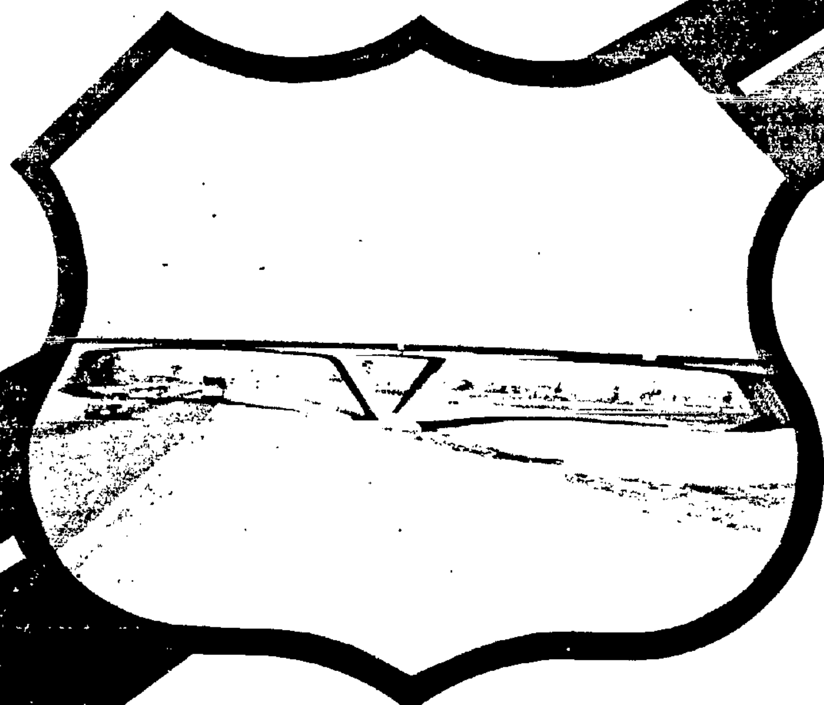
OPTIMIZED SECTIONS FOR MAJOR PRESTRESSED CONCRETE BRIDGE GIRDERS

February 1982
Final Report



Prepared for

U.S. Department of Transportation
Federal Highway Administration

Offices of Research & Development
Structures and Applied Mechanics Division
Washington, D.C. 20590

FOREWORD

This report presents the results of an analytical investigation of standard prestressed concrete bridge girder designs and makes recommendations regarding new national standards. The report will be of interest to engineers involved in the design, analysis and construction of highway bridges.

The research documented in this report was done as part of the Federal Highway Administration FCP Program. Results are being integrated into FCP Project 5K entitled "New Bridge Design Concepts." Mr. Craig A. Ballinger is the project manager and Mr. Thomas Krylowski is the task manager.

Sufficient copies of the report are being distributed to provide a minimum of one copy to each regional office, one copy to each division office, and two copies to each State highway department. Direct distribution is being made to the division offices.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

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16. Abstract

The objectives of this investigation were to evaluate the latest prestressed concrete bridge girder designs being used in the United States and to determine which represent optimum designs that could be promoted as national or regional standards. Bridges built with pretensioned I- and T-sections for spans in excess of 80 ft (24.4 m) and concrete compressive strengths up to 7000 psi (48.3 MPa) were considered.

Information on current designs was collected from selected highway agencies and producers throughout the United States. In all states surveyed except California, the most economical bridges for spans of 70 to 130 ft (21.3 to 39.6 m) were constructed with pretensioned bridge girders. Precast prestressed bridge girder sections inventoried were analyzed on three efficiency scales. Bulb-T's, Colorado, and Washington girders were more structurally efficient than AASHTO-PCI girders.

A computer program called "BRIDGE" was developed to perform cost analyses. Parameters included girder span, girder spacing, deck thickness and concrete compressive strength. Based on relative unit costs for in-place materials and labor, cost charts were prepared for existing Bulb-T's, Colorado, Washington and AASHTO girders, and for their modified counterparts with 6-in. (152 mm) thick webs. All girders were compared using optimum cost curves. Bulb-T's were found most cost-effective with estimated cost savings of 17% on the in-place cost of girders and deck compared to the AASHTO girders. Next most cost-effective sections were the Washington Series girders.

Modified Bulb-T's are recommended for use as national standards.

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INTRODUCTION

Background

The concept of prestressed concrete can be traced back to early 19th Century.[1]* Modern development of prestressed concrete is credited to E. Freyssinet of France, who, in 1928, started using high strength steel wires for prestressing. Early application in the United States tended to be concentrated in the area of circular prestressed structures, especially as applied to storage tanks.

Linear prestressing did not start in the United States until 1949 when the famous Walnut Lane Bridge was constructed in Philadelphia. Following this initial effort, application of prestressing in the transportation industry grew rapidly. Annual reports of Bureau of Public Roads for each of fiscal years 1954 through 1957 reported that "Use of prestressed concrete in bridge construction continued to grow in favor because in many situations it permits large savings in materials and cost."[2]

In early applications of prestressed concrete to bridges, designers developed their own ideas of the "best" girder sections. The result was that each contractor used a slightly different girder shape. Consequently, it was not possible to re-use girder forms on subsequent contracts. It soon became apparent that producers could not afford to have a variety of expensive steel forms. Moreover, it was too expensive to design "custom" girders for each bridge.

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* Numbers in parenthesis denote references listed at the end of the report.

As a result, representatives of the Bureau of Public Roads, the American Association of State Highway Officials (AASHO)*, and the Prestressed Concrete Institute (PCI) began working on what has since become a series of standard AASHO (now AASHTO) sections for bridge girders. Standard girders Types I through IV were developed in the late 1950's, while Types V and VI were developed in the early 1960's.

Adoption of the AASHO-PCI standard bridge girders simplified design practice and led to wider utilization of prestressed concrete for bridges. A Bureau of Public Roads survey showed that for the years 1957-1960, 2,052 prestressed concrete bridges were authorized for construction with an aggregate cost of $290 million.(1)

There is no doubt that standardization of girders has led to considerable savings in the construction of bridges.(3-5) While standardization may be good, it should also be recognized that it has some drawbacks. Standardization may not only retard further development but also may result in a decrease in economy as the basis for the original selection becomes obsolete.

Since adoption of the standard AASHO girder shapes, there have been significant advancements in the technology of prestressed concrete design and construction. Numerous research studies have provided increased knowledge of structural behavior of prestressed concrete members. This, in turn, has led to refinements in criteria for designing such members. Also, safety standards for interstate and other high speed highways require stricter clearance requirements. These have necessitated longer span bridges.

In recent years, there have been several indications of a need to update design of the standard AASHTO-PCI girders or possibly develop entirely new designs for major prestressed

* The American Association of State Highway and Transportation Officials (AASHTO) was formerly called American Association of State Highway Officials (AASHO).

concrete girders. For example, some state highway departments have developed "improved" girder shapes and others have begun using other girder types or have ceased using prestressed concrete girders. Therefore, the question is, "How efficient are Standard AASHTO-PCI girder sections, especially for spans in excess of 80 ft?"

Objectives

The objectives of this investigation are:

"To evaluate the latest prestressed concrete bridge girder designs being used in this country and to determine which represent optimum designs that could be promoted as national or regional standards."

This investigation was limited to bridges built with pretensioned I- and T-sections, for spans in excess of 80 ft (24.4 m), and with concrete compressive strengths up to 7000 psi (48.3 MPa).

Scope

The above objectives were accomplished within the following scope:

1. Current prestressed concrete girder designs being employed in the United States were summarized.
2. Creative, new concepts becoming available through research were reviewed.
3. Girders representing optimum designs and exhibiting strong potential for standardization were determined.
4. Recommendations for standardization of most practical and cost-effective designs were made.

Findings

The cost-effectiveness of existing "improved" girders was compared with that of the AASHTO girders. Existing improved girders included Colorado and Washington state girders, and Bulb-T's. Most cost-effective girders were the Bulb-T's. Next most cost-effective girders were the Washington Series followed by the Colorado girders.

Except for one section, the above improved girder shapes have 5-in (127 mm) thick webs. As a result, to satisfy the minimum clear concrete cover requirements, the strands are bundled in the center portion of the girder. Moreover, these improved girders have end blocks.

To avoid bundling of strands, and for ease of consolidating the concrete in the girders during manufacture, cost-effective sections with 5-in (127 mm) thick webs were analyzed with modifications. The web thickness was increased to 6 in (152 mm). Cost-effectiveness of the modified sections was also compared with that of the AASHTO girders. Among the modified sections, Bulb-T's were found to be the most cost-effective.

Based on survey results and cost analyses made in this report, Modified Bulb-T's are recommended for use as national standards. These girders lead to savings up to 17% on the in-place cost of deck and girder compared to the AASHTO girders.

RESEARCH APPROACH

The project was divided into two phases.

Phase I - Evaluation of Current Designs

The project was limited to "Solid-Form" prestressed concrete girders for spans in excess of 80 ft (24.4 m). Information was collected from selected users and suppliers throughout the United States. Selected highway agencies and producers were surveyed either through telephone conversations or through site visits. The purpose of collecting the information was to permit effective evaluation of girder design concepts used in different states.

Advantages and disadvantages of concepts inventoried were assessed. Design, fabrication, transportation, erection, and performance of the different sections were evaluated.

Information inventoried and assessed during Phase I was summarized in an Interim Report.[6] This summary is repeated in this report. The Interim Report also contained a work plan for Phase II of the project.

Phase II - Structural Efficiency and Cost Effectiveness

In Phase II, structural efficiency and cost effectiveness of the "best" existing designs, as well as some modified ones, were evaluated relative to the efficiency of AASHTO sections. This included evaluation of structural parameters such as girder spacing, span length, concrete strength, and deck thickness.

A computer program was developed for use in the parametric studies. A relative unit cost index was assigned to girder and deck slab concretes, prestressing strands, and reinforcing steel. These units reflected in-place relative costs for finished girder and deck. Costs of materials and labor were included. Parameters considered in the cost-effectiveness analysis pertained to the superstructure only. Data generated by the computer program was used to determine the most cost-effective girders.

PHASE I - SURVEY RESULTS

During Task A of Phase I, information was collected on "solid-form" prestressed concrete bridge girders with spans in excess of 80 ft (24.4 m) and concrete compressive strengths up to 7000 psi (48.3 MPa). This information was collected from selected highway agencies and producers located in different regions of the United States.

Highway agencies and other organizations were selected to reflect practices of different regions of the United States. Some States adopted the AASHTO-PCI standard bridge girders. However, a broad range of States used modified and improved girders as discussed later. States with known innovative concepts were surveyed.

Agencies and producers participating in the survey are listed in Appendix A. The survey started with telephone conversations with each participant. It was then followed by site visits to some of the agencies and producers. An outline of items discussed during site visits is listed in Appendix B.

Following the site surveys, collected information was compiled. Findings are summarized in the following sections.

Design, Fabrication and Construction Details

Girder Sections

Girder sections used by each of the surveyed highway agencies are listed in Appendix C. Included are sectional dimensions and properties of the AASHTO-PCI standard bridge girders and other girders used by the survey participants. Approximate span ranges of each section, are also given. Sections used for spans longer than 80 ft (24.4 m) are shown in Figures 1.a through 1.e. Main details differentiating sections are web thickness, size of flanges, and slope of haunch between web and flanges.

It is important to keep evolution of the different sections in perspective. Within the highway agencies surveyed, Pennsylvania's sections were the first to be developed. They were developed in the early 1950's. The AASHTO-PCI standard bridge girders Types I through IV were developed around 1957. They were adopted by several states. Around the same time, several states such as California, Illinois, Texas, and Washington started to produce their own sections.

Recognizing the need for long span girders, AASHTO-PCI standard bridge girders Types V and VI were developed around 1963. By 1971, the State of Colorado stopped using the AASHTO girders and adopted their present sections. At the same time the State of Wisconsin adopted the 70-in (1,778 mm) section.

In the Fall of 1979, none of the agencies and producers surveyed were spending any effort to modify or improve sections for long span girders. Virginia's Department of Highways and Transportation, in cooperation with the local producers was working on developing a single and a double-T girder for short span bridges.

More recently, it was learned that the State of Washington revised their Series 120 prestressed girder. The new section, labeled Series 14 prestressed girder, has a modified top flange that provides additional lateral stiffness. The dimensions of the new section are shown in Figure 1.e. The Series 14 girder will eventually replace the present Series 120 girder.[7]

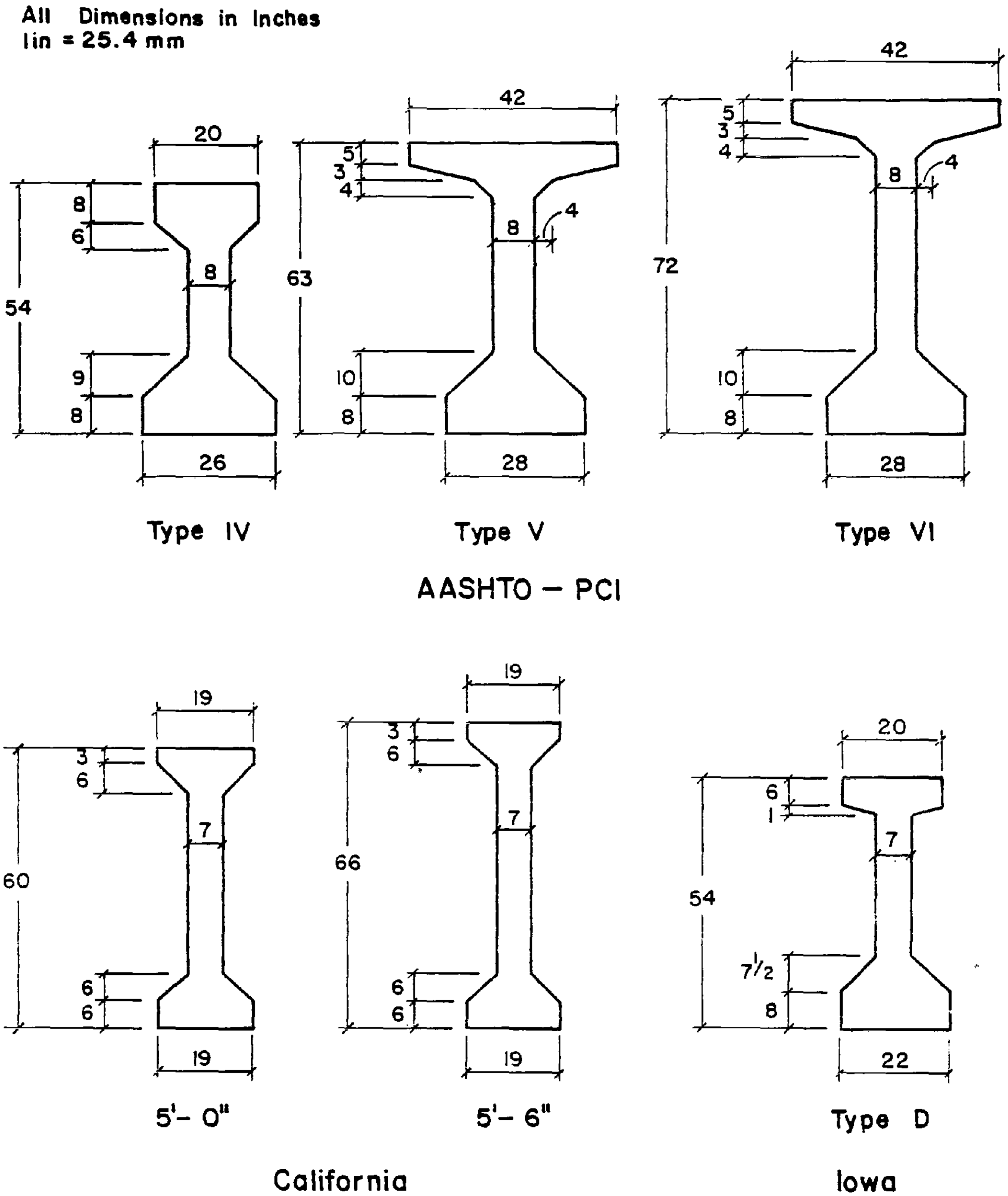


Figure 1.a Girder Cross Sections

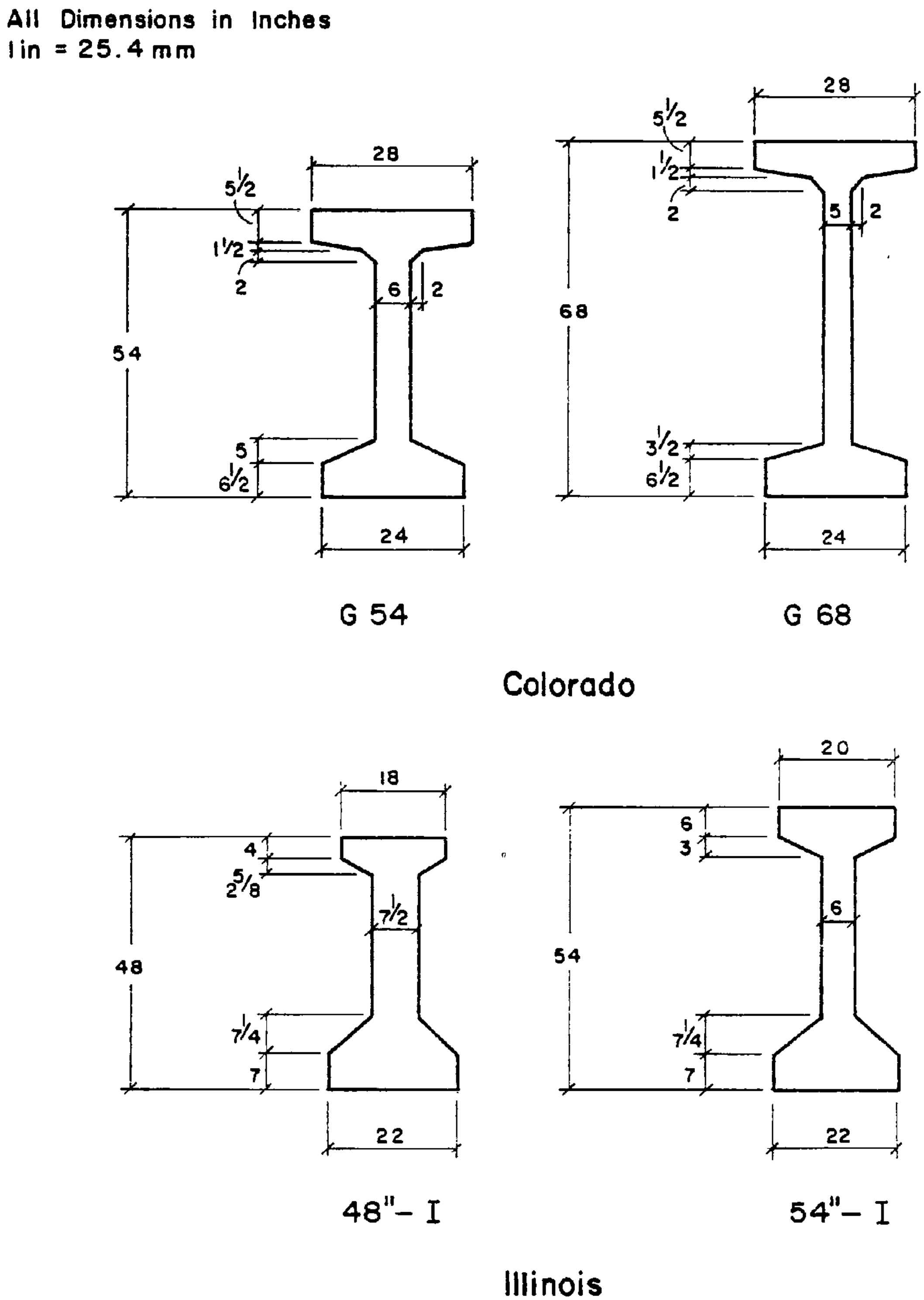


Figure 1.b Girder Cross Sections

All Dimensions in inches
1in = 25.4 mm

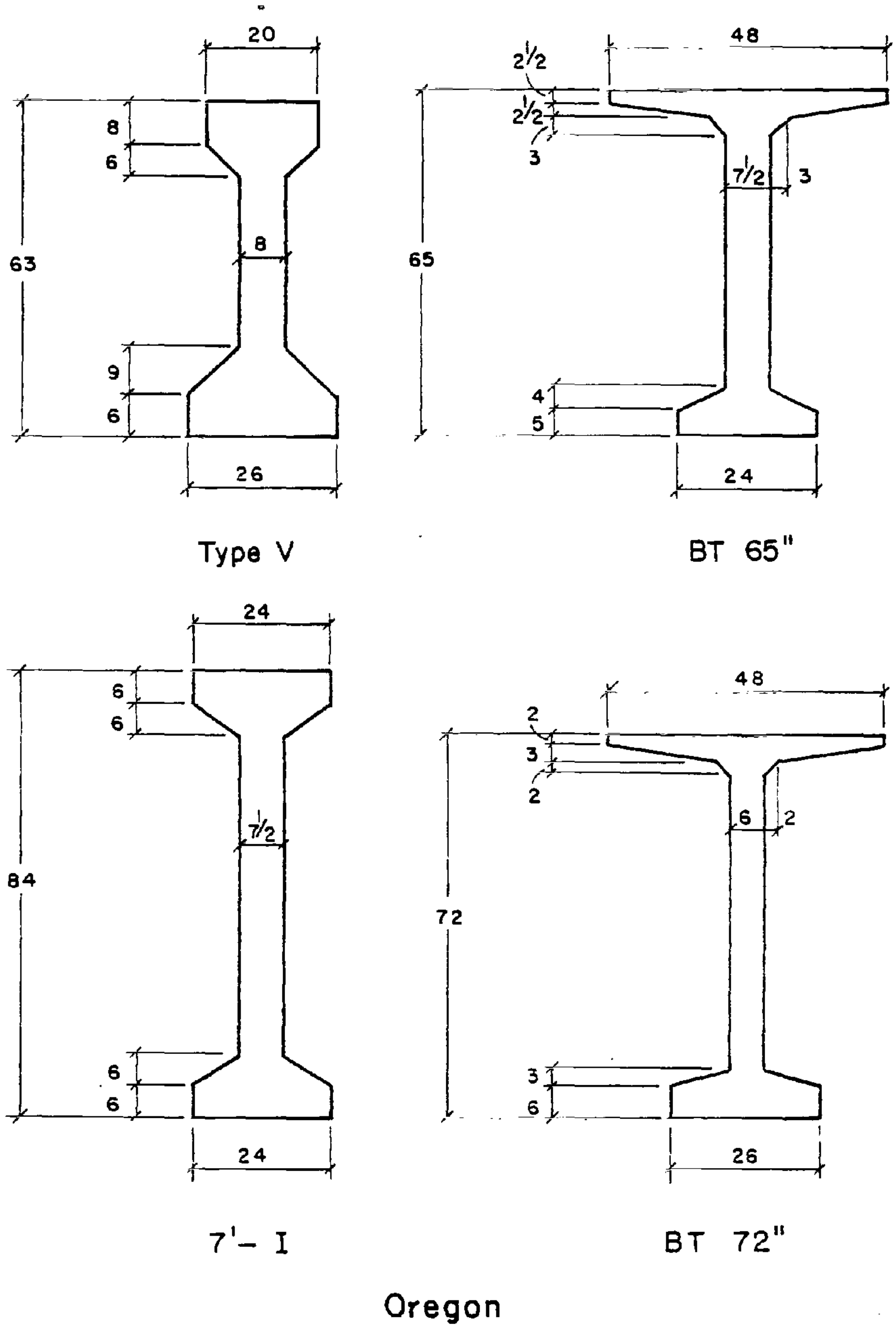


Figure 1.c Girder Cross Sections

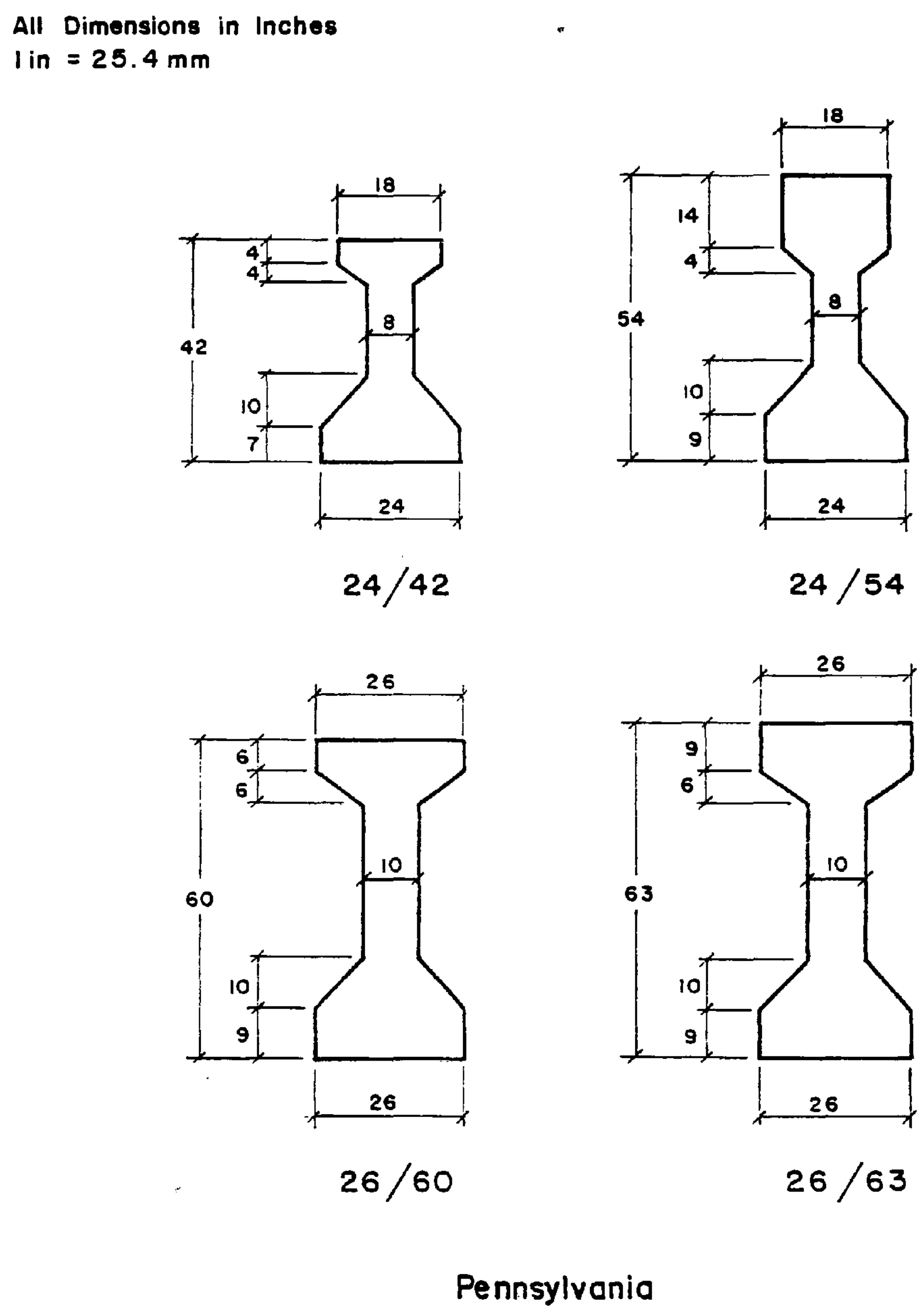


Figure 1.d Girder Cross Sections

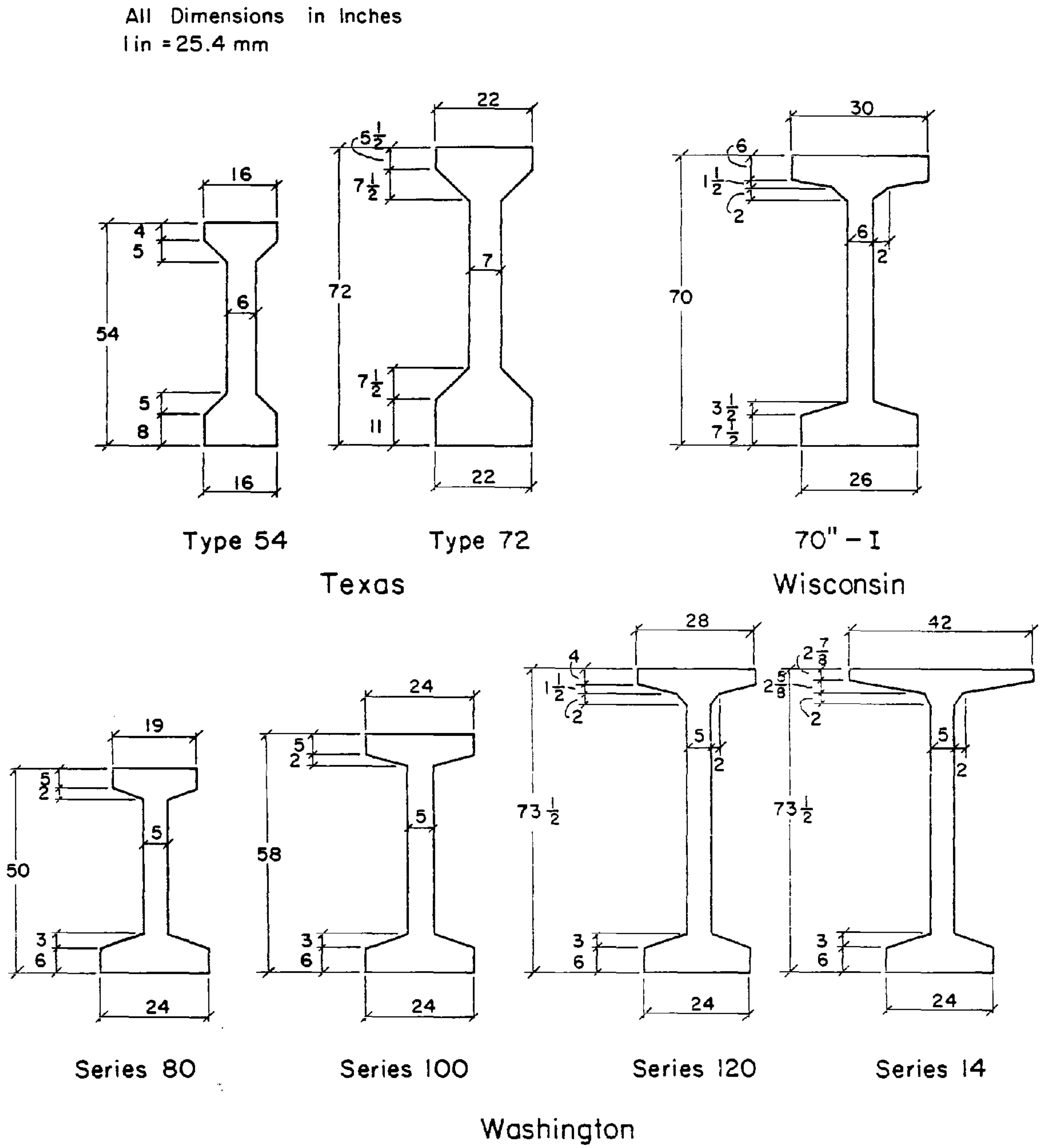


Figure 1.e Girder Cross Sections

Participants were asked if they would consider improved sections if developed. Several agencies are satisfied with their present sections. However, they said that they would consider new sections if these resulted in significant advantages and were cost effective. Some agencies are reluctant to change the AASHTO sections they use. The basic reason is that their producers are successful in producing these sections. Moreover, the forms represent a large investment. New forms would increase the unit cost of girders.

Longest spans built from standard, single unit girders are as follows: 105 ft (32 m) in Illinois, 112 ft (34.1 m) in Louisiana, 118 ft (36 m) in Pennsylvania, 120 ft (36.6 m) in Virginia and California, 125 ft (38.1 m) in Tennessee, 130 ft (39.6 m) in Colorado and Wisconsin, 140 ft (42.7 m) in Oregon, 145 ft (44.2 m) in Washington, and 150 ft (45.7 m) in Texas. Spans longer than 150 ft (45.7 m) have been produced by increasing the depth of the top flange or through splicing of long precast segments. Splicing of I-girders is discussed separately under the heading "Special Concepts".

End Blocks

End blocks serve two functions. First they provide a larger anchorage zone at the ends of the girder. Large bursting stresses occur in these areas due to stress transfer from the pretensioned strands to the surrounding concrete. Vertical stirrups are required to resist the bursting stresses and thereby prevent splitting (horizontal) cracks at the ends of the girder.

The second function of end blocks is that they provide greater web thickness at the ends of girders where shear forces are highest. Where adjacent spans are made continuous for live load, higher shear forces occur near the supports.

The most recent edition of "AASHTO Standard Specifications for Highway Bridges"[8] does not require use of end blocks in pretensioned beams. End blocks are utilized in very few states. Shear design dictates the need for end blocks in sections with thin webs and long spans. Included in this category are the

Washington and Colorado State standard sections as well as the Wisconsin 70-in (1,778 mm) girder. These girders have webs 5- or 6-in (127 or 152 mm) thick.

In Oregon, all bridge girders have end blocks. Girder sections include AASHTO standard girders Types II, III, and IV, Oregon Type V, bulb-T's and a 7-ft (2.13 m) deep I-Girder. Oregon sections having 7.5- and 8-in (190 and 203 mm) thick webs have three side-by-side strands deflected from each row. The end blocks are utilized to prevent cracking and spalling in end regions.

In Pennsylvania, some sections have webs up to 14 in (356 mm) thick. In these sections, five side-by-side strands are deflected from each row of strands. The State of Texas has one producer equipped to deflect three side-by-side strands.

Continuity for Live Load

Most survey participants design adjacent spans to be continuous for live load. Some started this practice recently and consider it occasionally for long spans. It has been estimated that continuity will provide about 10% more span capability than if the beams are simply supported. The AASHTO Specifications provide guidelines for design of "Bridges Composed of Simple-Span Precast Prestressed Girders Made Continuous".[8]

Diaphragms

In all states surveyed, diaphragms are used at span ends as recommended by the AASHTO Specifications.[8] It was reported that end diaphragms ensure distribution of reactions at span ends and provide smoother riding as vehicles cross over supports.

Intermediate diaphragms are not used in the States of Illinois and Tennessee. In all other states, intermediate diaphragms are used for spans in excess of 40 ft (12.2 m) as recommended by the AASHTO Specifications.[8] For spans from 40 to 80 ft (12.2 to 24.4 m), one diaphragm is recommended at midspan. For spans in excess of 80 ft (24.4 m), diaphragms are recommended at third points of the span.

In several states, the feeling was that intermediate diaphragms were useful in cases of collisions by overheight vehicles. Research has indicated that the structural usefulness of diaphragms is minimal, and they are harmful in most cases.(9-10)

Level of Tension in Concrete

Most states surveyed design for 6 $\sqrt{f'_c}$ psi (0.5 $\sqrt{f'_c}$ MPa) tensile stress in the concrete under service conditions. This is the upper level currently permitted by the AASHTO Specifications.(8) In one state, allowable concrete tension under service condition is limited to 3 $\sqrt{f'_c}$ psi (0.25 $\sqrt{f'_c}$ MPa). Several states do not allow tension in the concrete after all losses have occurred. In one case, factored dead and live loads correspond to cracking moment.

Concrete Properties

In most states, pretensioned bridge girders are manufactured from normal weight concrete (150 lb/cu ft, 2,403 kg/m^3). In the Pacific Northwest Region, concrete density is 155 lb/cu ft (2,483 kg/m^3).

Generally, girders are designed for concrete compressive strength, f'_c, of 5000 to 6000 psi (34.5 to 41.4 MPa) at 28 days. However, concrete design strength is as low as 4000 psi (27.6 MPa) in the Pacific Southwest Region and as high as 7000 psi (48.3 MPa) in the Pacific Northwest Region. Compressive strength of concrete at time of initial prestress, f'_{ci}, ranges from 4000 to 6000 psi (27.6 to 41.4 MPa).

Among survey participants, girders of only one bridge in Washington were built from lightweight concrete. Decks made of lightweight concrete have been used only on a few bridges having steel girders. Survey participants believe that the behavior of structural lightweight concrete is not adequately understood, and creep characteristics are not sufficiently documented. Consequently, lightweight concrete is not considered for pretensioned bridge girders.

Strand Properties

The most common size strand for pretensioned girders is 1/2-in (12.7 mm) diameter. It is the only size used in several states. A few states also design with 7/16-in (11.1 mm) diameter strands. To allow for competitive bids, alternate designs with both sizes of strands are prepared. In all cases, Grade 270 (1,860 MPa) strands are used.

Requirements for surface condition of strands vary with states. The State of Washington specifies bright strands. In California, shiny strands are required for post-tensioned girders. "Rust free" steel is specified for use in pretensioned girders. Most states permit a light coating of surface rust. In no case is loose rust or pitting allowed.

All states specify stress-relieved strands, while some also allow low-relaxation strands.

Girder Spacing

Spacing used for pretensioned girders varies between 4.5 and 10 ft (1.37 and 305 m) in most states. In exceptional cases, the spacing is larger. Maximum spacing reported is 12 ft (3.66 m). To reduce the cost of prestressed concrete bridges, Scott[11] has suggested using as few girders as possible in each span.

Decks

Three types of decks are used in conjunction with pretensioned bridge girders. Most decks are cast-in-place and supported on temporary wood forms. In Pennsylvania and Texas, some decks are cast on permanent steel forms.

A technique gaining popularity in several states is the use of precast prestressed concrete deck panels. These panels act as permanent forms for the cast-in-place concrete deck. They become an integral part of the finished deck. Recent AASHTO Specifications provide guidance for design of the precast deck panels.[12]

Design Loads

All survey participants design their highway bridges to carry the HS 20-44 loading. It is recognized that overloads occur frequently. In the State of Oregon, bridges subjected to heavy traffic are designed for HS-25 loading.

All Interstate highway bridges are also checked for military loading. This loading governs the design of spans less than approximately 37 ft (11.3 m) when simply supported.

Design Aids

Preliminary designs are based on experience and/or design charts or tables. In the majority of agencies, final design calculations are performed by electronic computers. In one state, the only design aids used are detailed design charts. This elaborate set of charts was developed at an appreciable initial cost.

Influence of Producers and Contractors on Type of Bridge

During the survey, questions were asked regarding adequacy of girder producers and construction contractors and whether they influenced the engineer's decision regarding type of bridge. Generally, highway agencies are satisfied with performance of both manufacturers and contractors. Occasionally, problems are encountered in some states. Usually, decisions regarding type of bridge rest on least initial cost and contractor's experience.

Alternate Designs

In many states, alternate designs are provided to stimulate competition and obtain least cost bridges. In some agencies, options are prepared for straight or draped strands and different compressive strength of concrete at initial prestress. Alternate designs and options are prepared by the highway agencies. Preliminary designs undergo value engineering, i.e., overall improvement in the cost of bridges is realized by carefully investigating availability of materials, construction methods, shipping costs, and similar cost influencing items.

Fabrication Details

Procedures for pretensioning strands, and deflecting draped strands vary with manufacturers. Some highway agencies require bundling of strands at hold down points. Usually strands are deflected at two locations, each at about 0.4 point of span. In long span girders, a larger number of strands are draped. In such cases, some plants choose to deflect the strands at four hold down points in each girder. Acceptable surface condition of strands was described earlier under "Strand Properties".

The AASHTO Specifications[8] contain requirements for minimum concrete cover for reinforcement and minimum spacing between strands. The minimum concrete cover is 1-1/2 in (38.1 mm) for prestressing steel and 1 in (25.4 mm) for web reinforcement. For pretensioning steel, minimum clear spacing of strands at the ends of beams is three times the diameter of the steel or 1-1/3 times the maximum size of the concrete aggregate, whichever is greater. For 1/2-in (12.7 mm) diameter strands, the minimum distance between center of strands is 2 in (50.8 mm) provided maximum aggregate size does not exceed 1-1/8 in (28.6 mm).

Almost all survey participants follow the above requirements. There are two exceptions. In Washington, minimum vertical and horizontal spacing between center of strands is 1-3/4 in (44.5 mm). However, maximum aggregate size is 1/2 in (12.7 mm) and the concrete is denser than in other states. In Colorado, horizontal spacing between center of strands is 1-3/4 in (44.5 mm) while vertical spacing is 2 in (50.8 mm). In Pennsylvania, Virginia, and Wisconsin, minimum clear concrete cover for the web reinforcement is increased to 1-1/2 in (38.1 mm).

Detailing of web reinforcement greatly affects fabrication costs of pretensioned girders. Figures 2.a and 2.b show the types of web reinforcing schemes used in different states. It can be seen that there are as many schemes as survey participants. Scott[11] has suggested stirrups should be detailed such that reinforcement cages can be prefabricated. Alternatively,

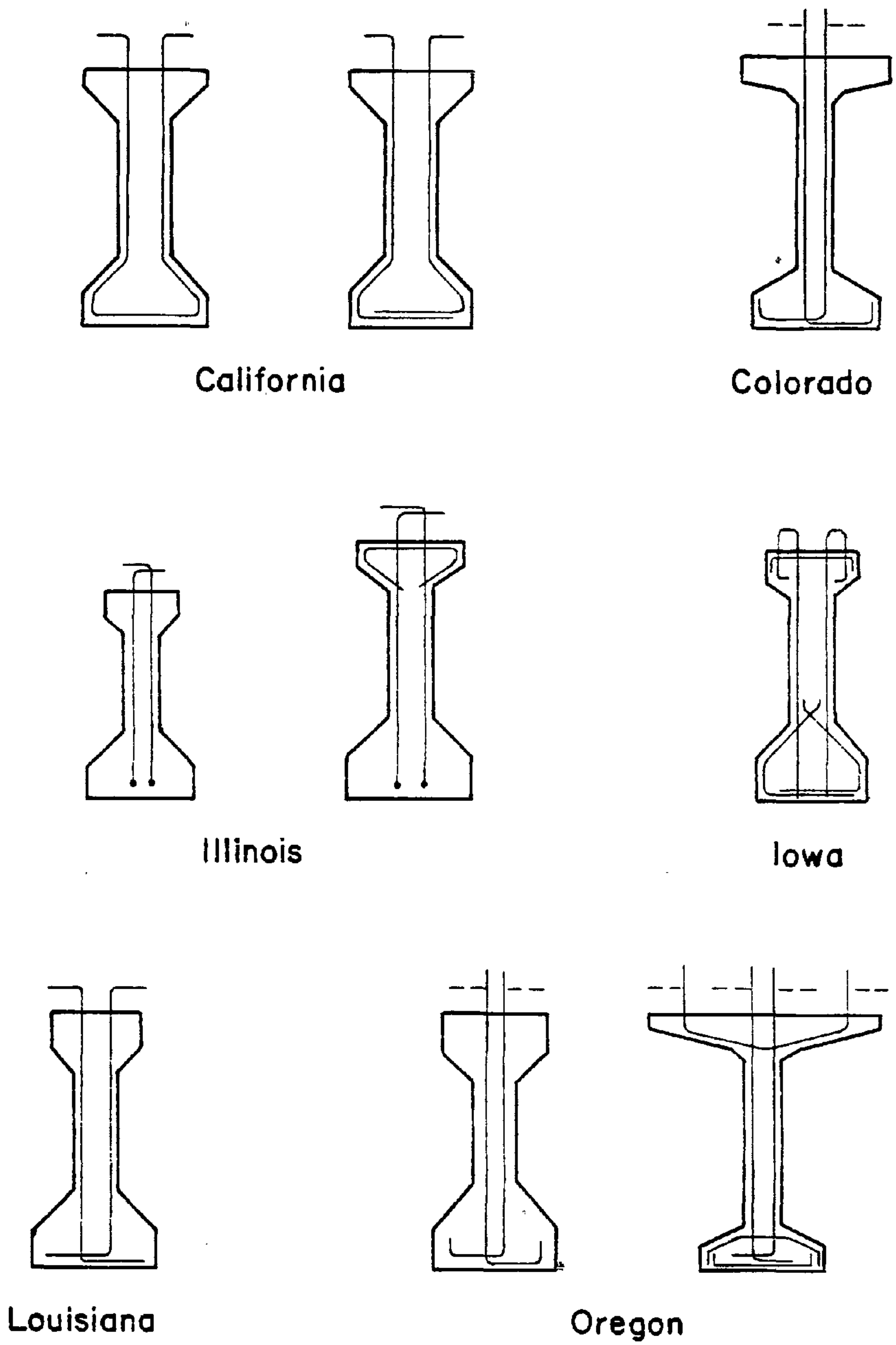


Figure 2.a Stirrup Schemes

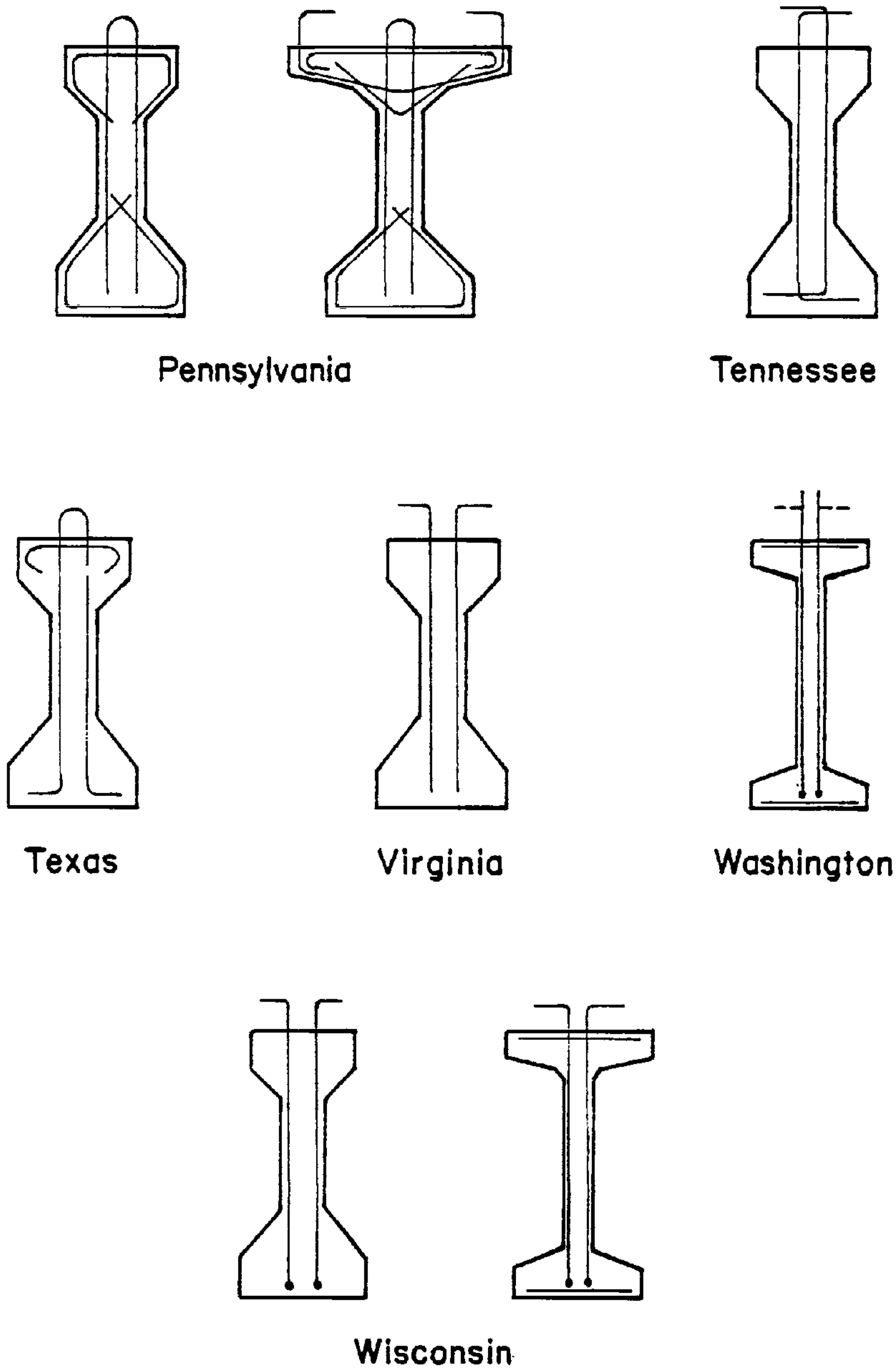


Figure 2.b Stirrup Schemes

stirrups should be easy to tie into place after the strands are tensioned. Moreover, it is not necessary to have the stirrups surrounding the strands.

Construction Details

Lateral stability of long span slender sections, during hauling and erection, presents problems in several states. External lateral stiffening devices made up of preassembled steel space trusses or external adjustable devices (hog rods) are provided. These devices are attached to the girder in the plant. They are only removed after setting and bracing the girders in their final position. The problem of lateral stability is discussed later under the heading "STRUCTURAL EFFICIENCY".

Minimum erection and bracing requirements are provided on some state drawings. Girders set on neoprene pads are found to be wobbly. Additional diagonal bracing is required. Guidelines for construction procedures are available in AASHTO Specifications[8] and state construction specification manuals.

Construction manuals provide a set of fabrication and construction tolerances. Camber differential of girders is of particular interest. In Pennsylvania and Washington, allowable camber differential between adjacent beams is 1/8 in per 10 ft (3.2 mm per 3.05 m) of span, up to a specified maximum. Maximum camber differential is 1-1/4 in (31.8 mm) in Pennsylvania, and 1 in (25.4 mm) in Washington.

Transportation Requirements and Restrictions

Regulations for hauling girders on highways and roads vary from state to state. Maximum overall length hauled without permit varies between 55 and 80 ft (16.8 and 24.4 m) for states surveyed. Lengths beyond these limits require special permit. Maximum permitted length depends on the route traveled. In some states, there is no limit to overlength, while others have an upper limit. In Pennsylvania, a length in excess of 160 ft (48.8 m) is not normally permitted. For spans discussed in this report, special overlength permits would be needed.

Maximum legal gross weight hauled without permit varies between 72 and 96 kips (320 and 427 kN), depending on the state. Beyond this limit, a special permit is required. Maximum legal overloads depend on the number of axles and distance between consecutive axles. Using appropriate number of axles, maximum girder weights discussed in this report may be hauled with special permits.

Overwidth and overheight regulations do not present limitations for the type of girders considered in this report.

Availability and Costs

Cost data obtained during the survey are summarized in Appendix D. Costs of material and labor vary from region to region within the United States. They also vary between states of a region, between districts of a state, and within a district according to bridge location.

Main factors affecting materials costs within a state are accessibility of the construction site, distance between girder producers and site, and distance between ready mix plants and site. Size of a bridge, and consequently number of girders ordered for a project, affect the unit cost of girders. This cost is higher for smaller quantities.

In remote areas, availability of labor experienced with bridge construction is often limited. This necessitates relocating skilled labor. As a result, contract cost increases.

Combining all the above factors presents difficulties in assessing cost factors in the different regions of the United States. The effect of these factors is reflected in the discrepancies in unit costs within a given state as shown in Appendix D. In addition, each highway agency uses a different approach to itemize unit costs. Costs for new construction, rehabilitation, and bridge widening are all averaged in several states.

Suggestions for Bridge Cost Reduction

During the survey, the question was asked about what could be done to reduce the cost of bridge construction. Suggestions to reduce the cost of pretensioned concrete bridge girder construction included:

1. Eliminate intermediate diaphragms
2. Use superplasticizers
3. Reduce web thickness of AASHTO standard bridge girders
4. Use a more efficient section to replace AASHTO Type IV girder
5. Use higher concrete compressive strength
6. Use maximum girder spacing

Structural Durability

Prestressed concrete bridge construction started in the United States 30 years ago. In recent years, the first post-tensioned bridge, the Walnut Lane Bridge, has shown signs of distress.(13) Generally, prestressed concrete bridges have displayed excellent performance. In all states surveyed, durability and performance of pretensioned bridge girders were reported to be very good.

Problems with pretensioned girders occur due to collisions by overheight vehicles. Where de-icing salts are used, deterioration of concrete decks presents problems. However, pretensioned concrete girders require virtually no maintenance.

Expected life span of pretensioned bridges varies between 40 and 100 years. Inspection frequency varies with age and condition of bridges. In several states, inspection is performed on a yearly basis. In some states, a routine inspection is made every six months, and an in-depth inspection is made every two years.

In-service pretensioned girders are reported to be free of cracks. The only cracks reported have occurred in the plant at release of prestress.

Aesthetics and Safety

No strict aesthetic requirements are specified by any of the states surveyed. However, they all plan and build their bridges to be aesthetically pleasing. It has been suggested that the most structurally efficient bridge would be the most aesthetically pleasing.[14]

In California, box girder bridges are found to be more appealing and economical. In all other states, pretensioned bridge girders are found more economical for spans of 70 to 130 ft (21.3 to 39.6 m).

In Oregon and Virginia, exposed girders and supports are painted to improve their appearance. In Washington, architects are consulted in the preliminary design stages. In the City of Philadelphia, an art commission approves every bridge.

Girders having end blocks were discussed earlier. For aesthetic reasons, all end blocks are gradually tapered over several feet.

Safety requirements specified by the Federal Highway Administration are incorporated in the section on General Features of Design in the AASHTO Specifications for Highway Bridges.[8]

Opinions and Policies

In all states surveyed, opinions and policies regarding type of bridge have a common ground. The type of bridge selected is based on economy. For short spans up to 30 ft (9.1 m), cast-in-place reinforced concrete slabs are the most economical. For spans from 30 to 75 ft (9.1 to 22.9 m), precast spread boxes or double-T's are considered. For longer spans between 70 and 150 ft (21.3 and 45.7 m), depending on the state, pretensioned I and T girders are selected. Beyond this range, steel bridges are considered.

In recent years, post-tensioned segmental construction has gained acceptance. This has proven to be the most economical for spans from roughly one hundred to several hundred feet long.

In California, 85% of the bridges are cast-in-place post-tensioned box girders. This type of bridge is found to be more attractive and economical than pretensioned I-girder construction.

Special Concepts

The following special concepts used by some states deserve mention:

Splicing of Girders

Transportation of girders for spans in excess of about 120 ft (36.6 m) presents overlength and overload problems in some areas. To achieve longer spans, splicing of precast segments has been used successfully in several states and in Ontario, Canada. Several splicing techniques are available. Guidelines to design the splice have been published.[15-16]

Among participants, four states have built bridges from long precast segments. Three other states have considered the technique. In California, spans of 139 ft (42.4 m) have been achieved. In Illinois, two bridges, each having two 125 ft (38.1 m) spans, have been built by splicing three girders segments.[17] In Oregon, 190 ft (57.9 m) spans have been achieved by splicing 7-ft (2.1 m) deep I-Sections or bulb-T sections. In 1975, Pennsylvania built a 140 ft (42.7 m) span from spliced girders.

Drop-In Spans

Another method to achieve long spans uses dapped girders dropped onto cast-in-place members cantilevering from the piers. This technique has been used in Oregon, Virginia, and Washington.

Precasting Deck Panels

For multiple span bridges built with precast I-girders, it is often cost effective to use precast prestressed deck panels. These pretensioned deck panels serve as permanent forms to place the cast-in-place concrete deck. They become an integral part of the finished deck. Recent AASHTO Specifications provide guidance for design of the precast deck panels.[12]

Precast deck panels have been successfully used in Illinois, Louisiana, Texas, and Virginia. This technique has also been used experimentally in Pennsylvania. California, Oregon, and

Washington are considering precast deck panels as alternate designs.

Bridge deck construction is accelerated where precast deck panels are used. This technique has been found economical on multiple span bridges. The large volume of required panels justifies the initial investment for set-up of a precasting bed.

Blanketing Strands

Blanketing is a means of eliminating the need for draped strands.[18] With this method, all strands are kept straight. Some are debonded at the ends of the girders to control the concrete stresses. Advantages of this technique are that stressing is done in one operation. There is better control over the level of prestress along the length of the girder. No hold-up or hold-down devices are needed.

Blanketing has been used for several years in Tennessee for the manufacture of pretensioned box girders. Several plants blanket some strands at the end of the member to avoid sudden transfer of prestress from all strands at the same region. Bursting stresses are spread over a longer portion of the member. Recently, the State of Louisiana has utilized this concept on some bridges. The PCI Committee on Bridges is preparing a report on "Use of Debonded Strands in Pretensioned Bridge Members".[19]

Pre Post-Tensioning

In pre post-tensioning, girders are pretensioned in the plant to a level that permits handling. They are then post-tensioned either in the plant at a later date or on site. Advantages of this procedure[20] are early stress transfer and control of deflection. Higher prestress eccentricities are achieved.

Pre post-tensioning has been used in the past in Washington. It has also been experimented with in Pennsylvania. It has been considered as an alternate design by other states.

PHASE II - STRUCTURAL EFFICIENCY

Currently, there is no well established procedure to measure the efficiency of a structural section. In a pretensioned bridge member, predominant stresses are flexural. Therefore, the designer's goal should be to use a section that has the highest section modulus with least area. Conversely, for a given sectional area, the highest section modulus is desirable. The final decision is based on economy. Usually, efficiency and economy go hand-in-hand.

One way of measuring the relative performance of bridge girder sections has been suggested by Anderson.[20] He suggests that the relationship between cross sectional area and section modulus for the bottom fibers be compared. Such a relationship is shown in Figure 3 for standard state sections of survey participants. Although the AASHTO-PCI Type VI section has the highest modulus, it also has the biggest area, i.e., it is the heaviest. It is interesting to note that Colorado's G68 section, the Washington Series 120 and 14, and AASHTO Type VI have about the same span capabilities. This is shown in Appendix C. However, G68 and Series 120 and 14 are about 40 percent lighter than Type VI.

An efficiency factor for prestressed sections has been derived by Guyon.[21] It is based on minimizing the area of the section for a given section modulus. This efficiency factor, ρ, is defined as:

$$\rho = \frac{r^2}{y_t y_b}$$

where r = radius of gyration of section $= \sqrt{I/A}$

y_t, y_b = distance from center of gravity to top and bottom fibers, respectively

I = moment of inertia

A = cross sectional area

The efficiency factor for various sections is listed in Tables 5 and 6 of Appendix C. Variation of the efficiency

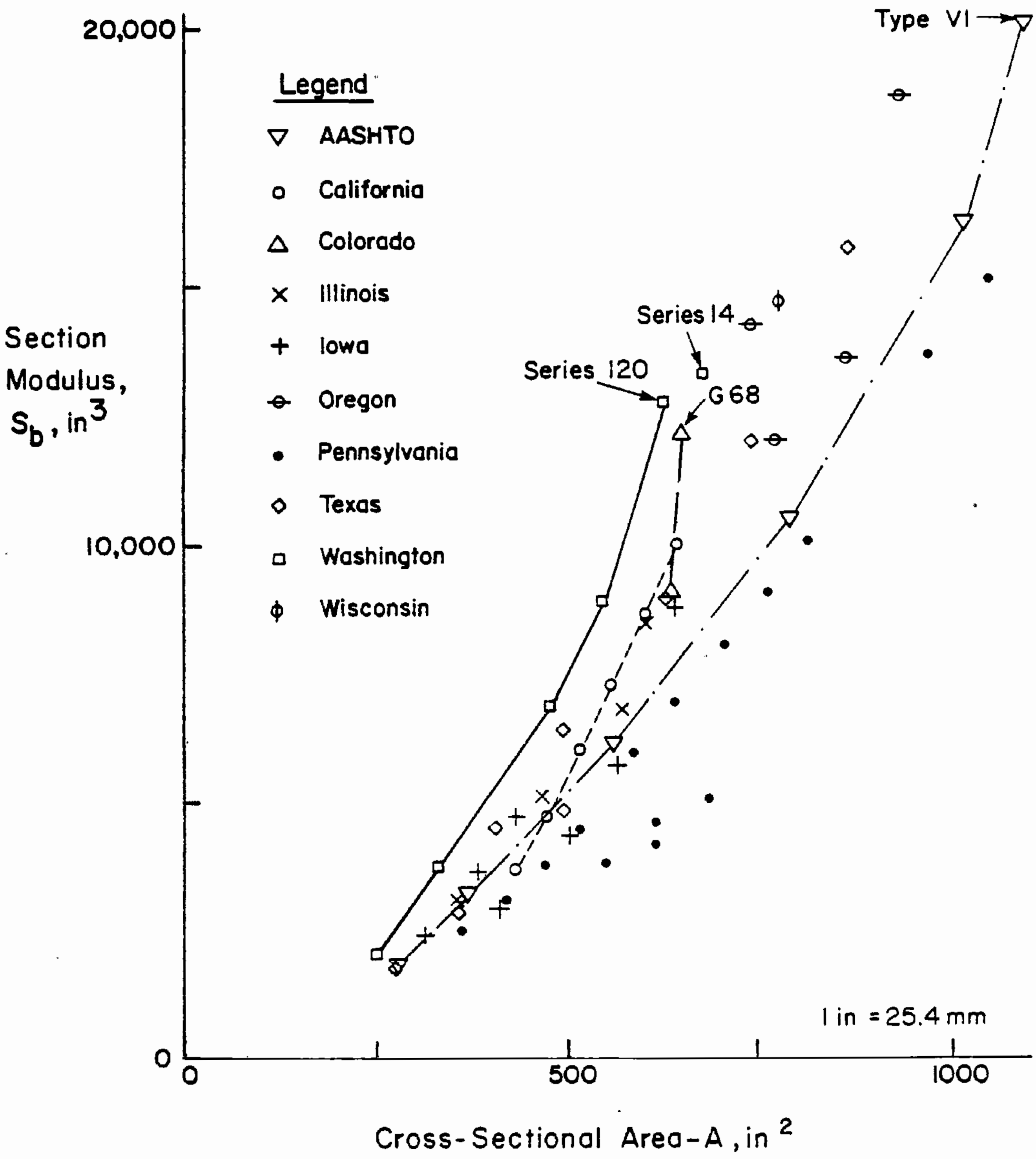


Figure 3 Relationship Between Modulus of Section for Bottom Fibers and Cross-Sectional Area

factor, ρ , with respect to depth of section is plotted in Figure 4. This figure indicates that the Colorado sections are comparable in efficiency to the Washington sections. Both are superior to the AASHTO Sections.

More recently, Aswad[22] has suggested another way of judging efficiency of I-Sections used in bridge superstructures. He proposed an efficiency ratio, α, defined as

$$\alpha = \frac{3.46\ S_b}{A\ h}$$

where S_b = section modulus for bottom fibers
A = cross sectional area
h = depth of section

Through analysis based on Colorado regional costs, Aswad found that the girders with the highest efficiency factor had the lowest cost per square foot of superstructure.[22]

Efficiency ratios of various sections are listed in Tables 5 and 6 of Appendix C. Figure 5 shows variation of efficiency ratio for different section depths. Data plotted are for all standard sections inventoried. According to Figures 3 to 5, Washington Series and Colorado girders are the most structurally efficient sections.

The three approaches described above confirm that the designer's goal should be to use sections that have the highest modulus with the least area within practical limitations. To achieve these goals in I-Sections, as much of the area as possible should be concentrated in the flanges. Therefore, the web should be as thin as practicable. Moreover, the haunch between web and flanges should be as flat as possible but still permit placing the concrete and stripping the forms.

Minimum web thickness is controlled by the present AASHTO Specifications.[8] For two 1/2-in (12.7 mm) diameter strands deflected side-by-side and No. 4 web reinforcement, the minimum web thickness is 5-1/2 in (140 mm). If the strands are bundled, minimum web thickness is 5 in (127 mm). All Washington

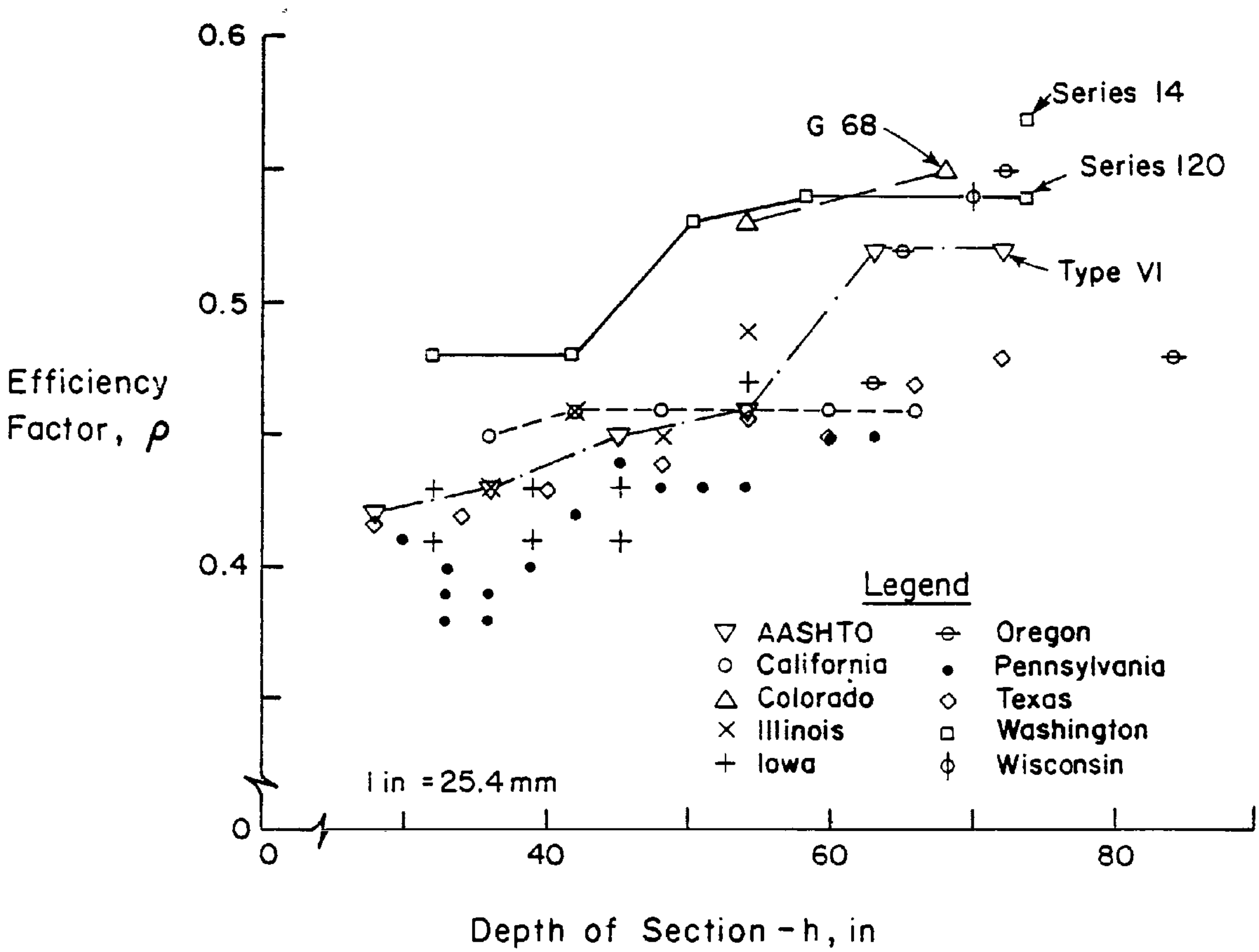


Figure 4 Variation of Efficiency Factor with Depth of Section

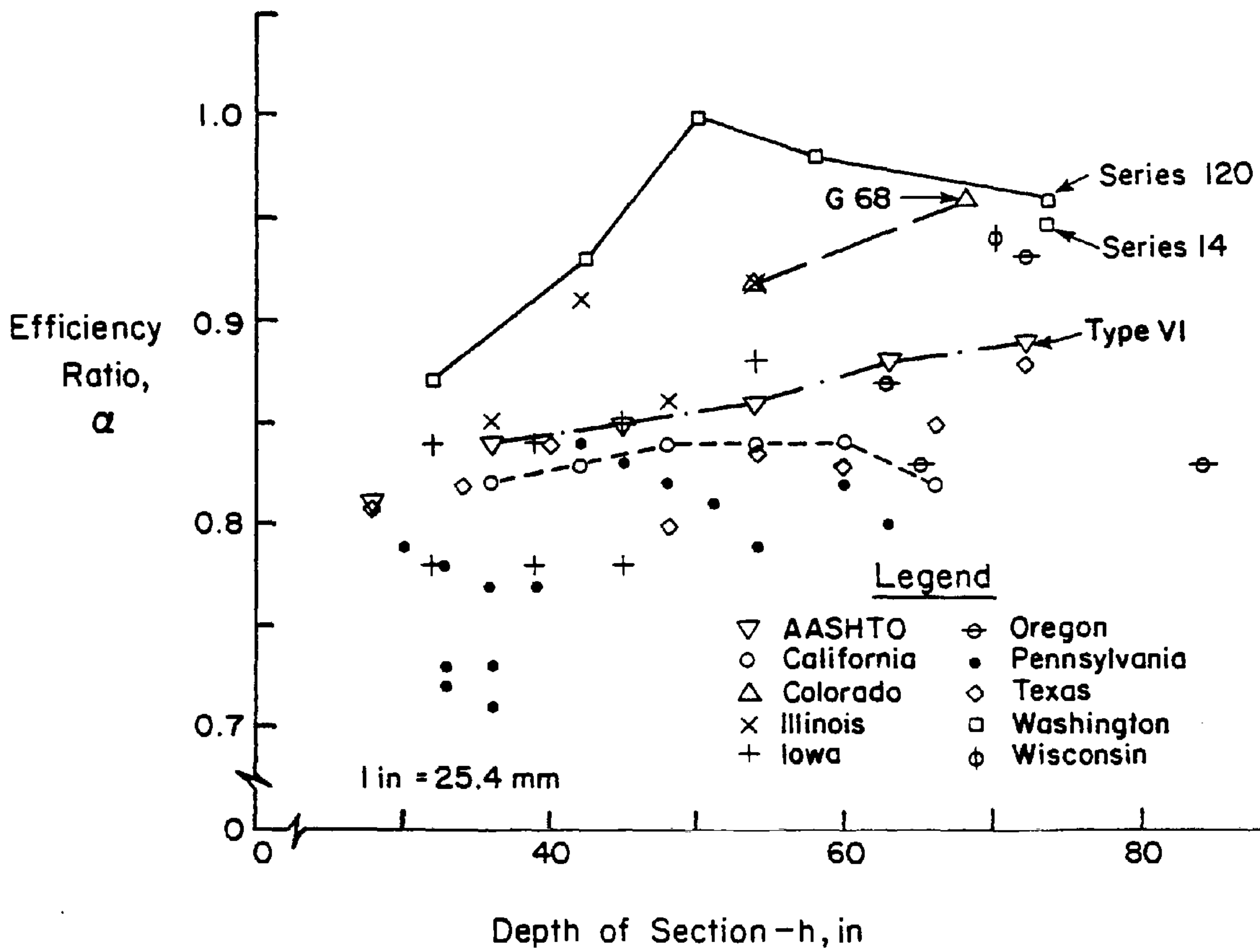


Figure 5 Variation of Efficiency Ratio with Depth of Section

State I-Sections and Colorado's G68 have 5-in (127 mm) thick webs.

It should be recognized that I-Sections with very thin webs are slender sections. As a result, stability during handling becomes a problem. Girders having 5-in (127 mm) thick webs and spans in excess of about 100 ft (30.5 m) require special attention during handling. On the other hand, I-Section girders having 7-in (178 mm) thick webs and narrow flanges also present stability problems during handling. Therefore, the overall lateral stiffness should be considered.[(23-24)]

Concern about the lateral stiffness of long spans prompted the State of Washington to develop Series 14 prestressed girder. This new section will eventually replace Series 120 girder. Lateral stiffness of Series 14 is about 1.9 times that of Series 120.

Bulb-T's and decked Bulb-T's are utilized in the Pacific Northwest. They provide better lateral stiffness compared to I-Sections. A Bulb-T is an I-Section with a wide (4 to 6 ft, 1.2 to 1.8 m) and thin top flange. A concrete deck is cast-in-place on top of the Bulb-T. As the top flange of Bulb-T's is wider than that of I-girders, less formwork is needed to support the deck concrete.

A decked Bulb-T has a full depth (5 to 6 in, 127 to 152 mm) and full width (4 to 10 ft, 1.22 to 3.05 m) top flange. The top flanges constitute the bridge deck. Adjacent units are joined through lateral post-tensioning or welded flange connectors where potential problems are currently being studied. The shear keys are grouted. Anderson has suggested use of decked Bulb-T's for simple spans up to 190 ft[(20)] (57.9 m). Because of their weight and length, such girders usually can be transported only on barges.

Bulb-T sections have been utilized in the State of Oregon for construction of bridges on the primary highway system during the last 15 years. Spans of 190 ft (57.9 m) have been achieved by splicing segments. At splice locations, the segments are supported on temporary supports. After casting the concrete deck, the bridge is post-tensioned. Continuity at permanent supports is obtained. Temporary supports are then removed. Over half a dozen bridges have been built using this technique.

Some 15 bridges have also been built in the State of Oregon from single unit Bulb-T's. Maximum spans obtained are 140 ft (42.7 m). Longer spans have been achieved through drop-in methods. Cast-in-place concrete members cantilever from the supports. Dapped end Bulb-T's are then dropped-in.

Decked Bulb-T's are also called integral deck Bulb-T's. They have been used for bridge construction only on secondary routes in the State of Oregon. First applications started three years ago. A dozen bridges, with spans up to 130 ft (39.6 m) have been built.

At the time of the survey, the State of Washington Department of Transportation had not built any bridges with decked Bulb-T's. However, they had been considered as alternative designs. Bulb-T's have been utilized during the last three years on about a dozen bridges in Washington. These sections have 5-in (127 mm) thick webs.

With the need for energy efficiency, the designer's goal should be to use the lightest section. This is accomplished by using the least amount of material possible. The section is lighter to transport, lighter to erect, and requires fewer prestressing strands.

In the previous section, different girder sections were compared with the AASHTO sections on three structural efficiency scales. In the next section cost effectiveness comparisons will be made for selected girder sections. Additional observations will be reported on the structural efficiency of selected girders.

PHASE II - COST EFFECTIVENESS

In this section, girder cross sections are selected for cost-effectiveness analysis. Assumptions made for the cost-effectiveness analysis are stated. The number of parameters involved in the analysis necessitated development of a computer program. This program is briefly described. Cost-effectiveness comparisons are made to reflect the effect of the parameters considered.

Cross Sections Analyzed

Earlier discussions under the heading "STRUCTURAL EFFICIENCY" indicated that the most efficient sections were the Washington state series and Colorado's G54 and G68 sections. It was also indicated that Bulb-T's had been used successfully in the Pacific Northwest. A set of Bulb-T sections was developed in 1959 by Anderson.[20] These sections, as well as the Washington series and Colorado's G68 have 5-in (127 mm) thick webs. To satisfy the AASHTO specifications,[8,12] web width should be not less than 5-1/2 in (140 mm) if strands are not bundled at mid-span. If strands are bundled, web width of 5 in (127 mm) would satisfy requirements for concrete cover and clear spacing between strands.

Several survey participants expressed concern about possible difficulties in manufacturing and transporting girders having 5-in (127 mm) thick webs. Main concerns were consolidation of the concrete in thin and deep members, and stability of such slender members during transportation. On the other hand some survey participants felt that present AASHTO girders can be improved by reducing their web thickness.

At a meeting[25] held in April 1980, members of the PCI Bridge Committee were asked about minimum practical web width to place and consolidate the concrete in precast prestressed I-sections. All committee members were in favor of a minimum web width of 6 in (152 mm).

AASHTO Standard Bridge Girders Types I and II have 6-in (152 mm) thick webs. In all regions of the United States, concrete has been placed and consolidated in these sections without difficulty. Therefore, in Phase II, existing sections having 5-in (127 mm) thick webs were evaluated and compared with similar sections having 6-in (152 mm) thick webs. Sections with 6-in (152 mm) thick webs should be easier to manufacture and transport than sections with 5-in (127 mm) thick webs.

Based on the above discussion, the following sections were evaluated in Phase II of the project:

1. Colorado's G54 and G68 girders. Girder G68 has a 5-in (127 mm) thick web.
2. Washington Series 80, 100, 120, and 14 girders. These girders have 5-in (127 mm) thick webs.
3. Anderson's Bulb-T's[20] BT48, BT60, and BT72. These girders have 5-in (127 mm) thick webs. The tips of the top flanges are 1 in (25.4 mm) thick.
4. Girder similar to Colorado's G68 but with 6-in (152 mm) thick web. This girder is designated Modified Colorado G68/6 in this report.
5. Girders similar to Washington series but with 6-in (152 mm) thick webs. These sections are designated Modified Washington Series 80/6, 100/6, 120/6, and 14/6.
6. Girders similar to Anderson's Bulb-T's, but with 6-in (152 mm) thick webs and 2-in (50.8 mm) thick top flange tips. These sections are designated Modified Bulb-T's BT48/6, BT60/6 and BT72/6.
7. AASHTO standard bridge girders Types IV, V, and VI.
8. Modified AASHTO girders where web thickness, and top and bottom flange widths are reduced by 2 in (50.8 mm). These were considered with the idea that existing forms could be used with reduced space between them. These sections are designated Modified Types IV, V, and VI.

Dimensions of existing and modified sections considered are shown in Figures 6 and 7, respectively. Both the Washington series and Anderson's bulb-T's with 5-in (127 mm) thick webs and the modified sections with 6-in (152 mm) thick webs are compared on the three efficiency scales in Figures 8 to 10. Sections with 6 in (152 mm) webs are slightly less efficient than similar sections with 5 in (127 mm) webs. However, according to Figures 9 and 10, Bulb-T's are more efficient than Washington Series. Further comparisons between sections are made below based on cost-effectiveness charts.

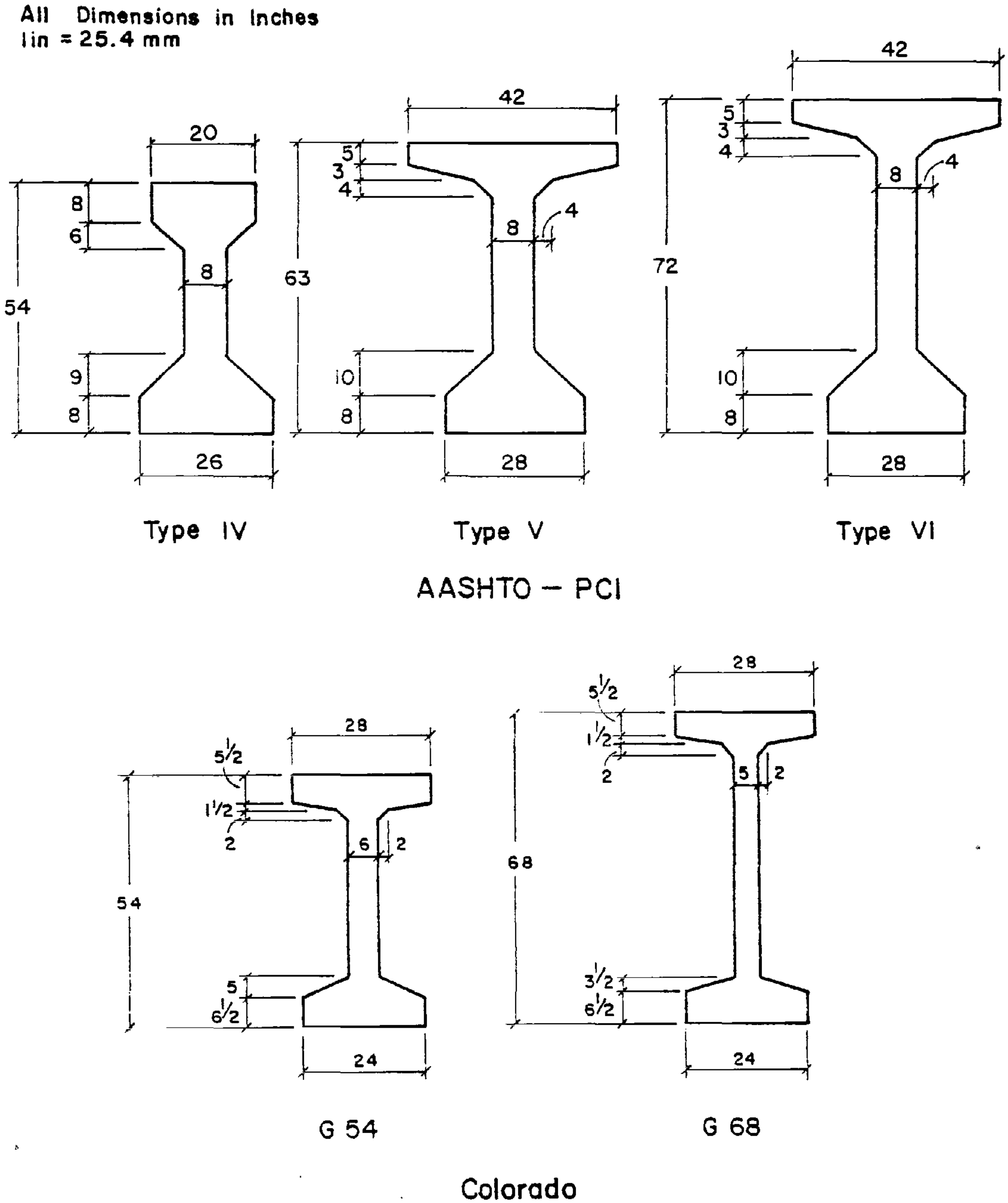
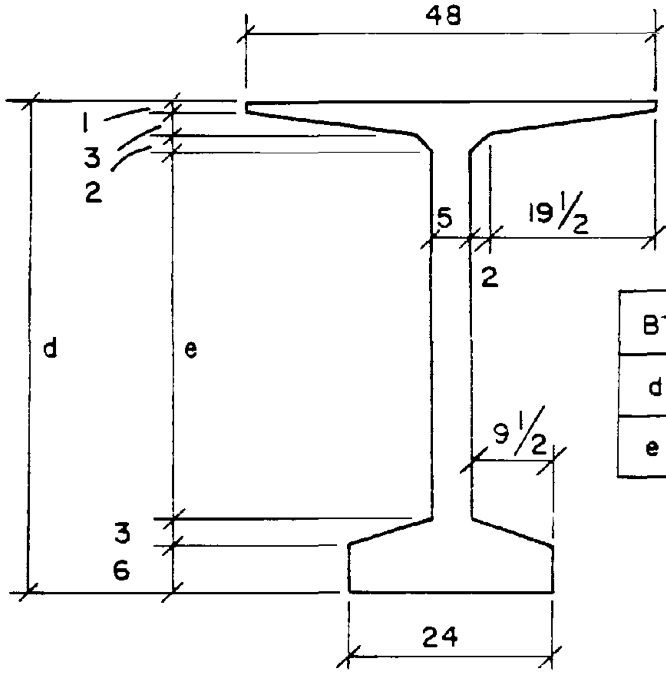


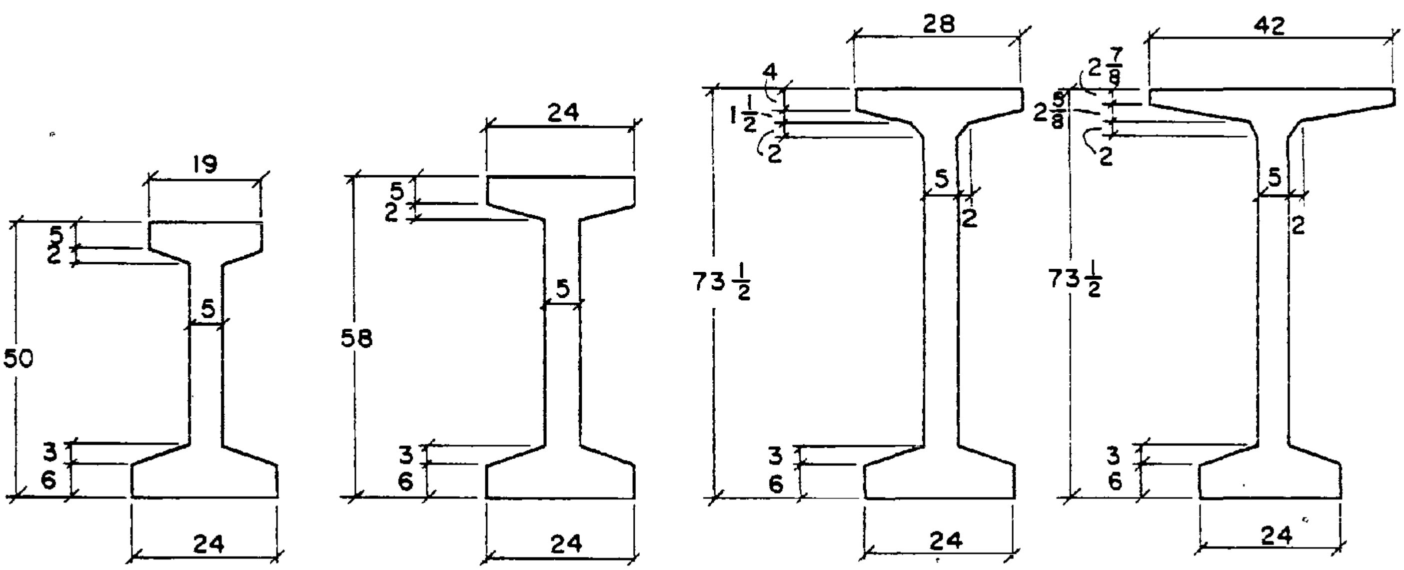
Figure 6.a Existing Girders Analyzed

All Dimensions in Inches
1 in = 25.4 mm



BT	48	60	72
d	48	60	72
e	33	45	57

Bulb-T



Series 80 Series 100 Series 120 Series 14

Washington

Figure 6.b Existing Girders Analyzed

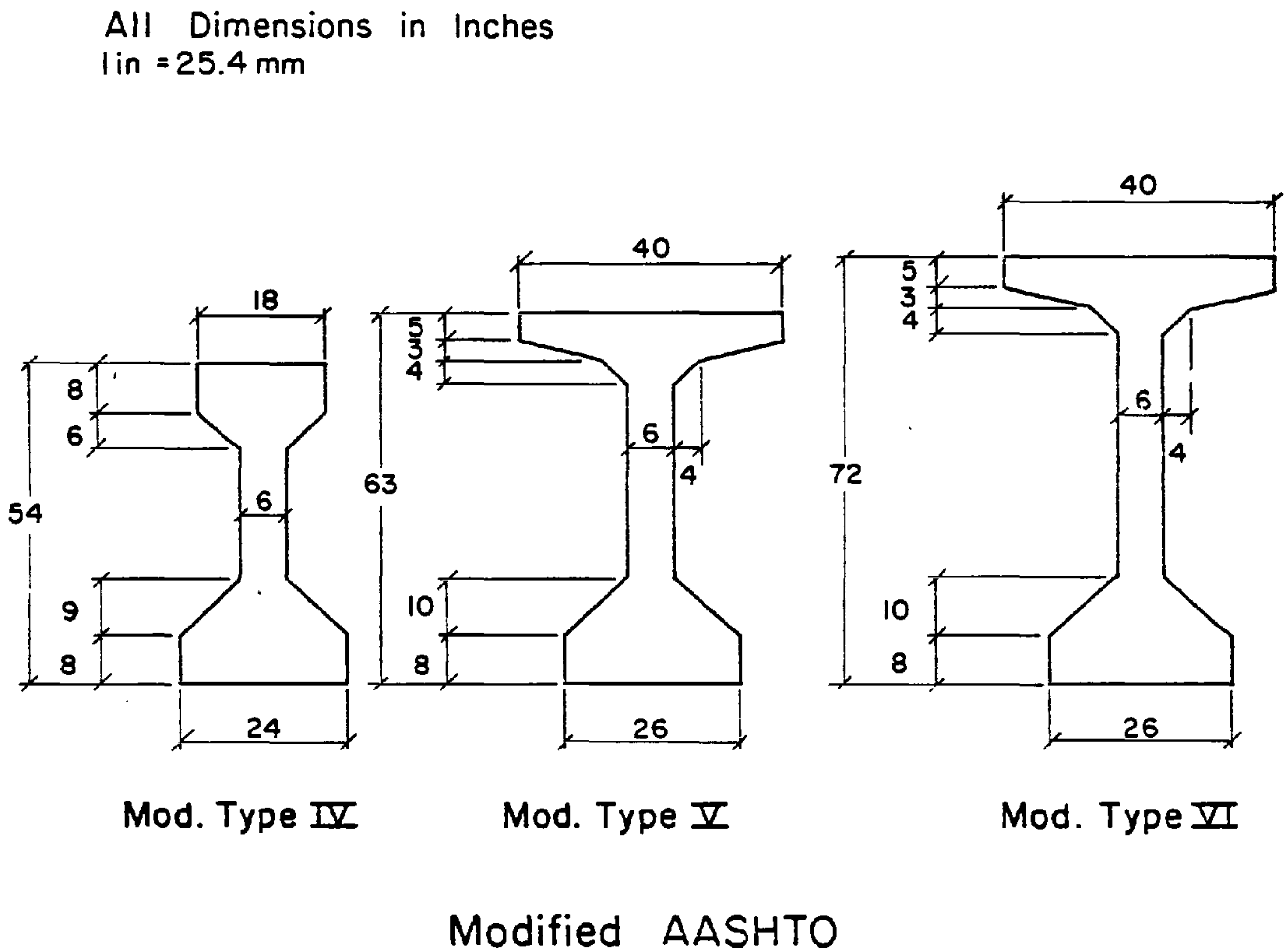
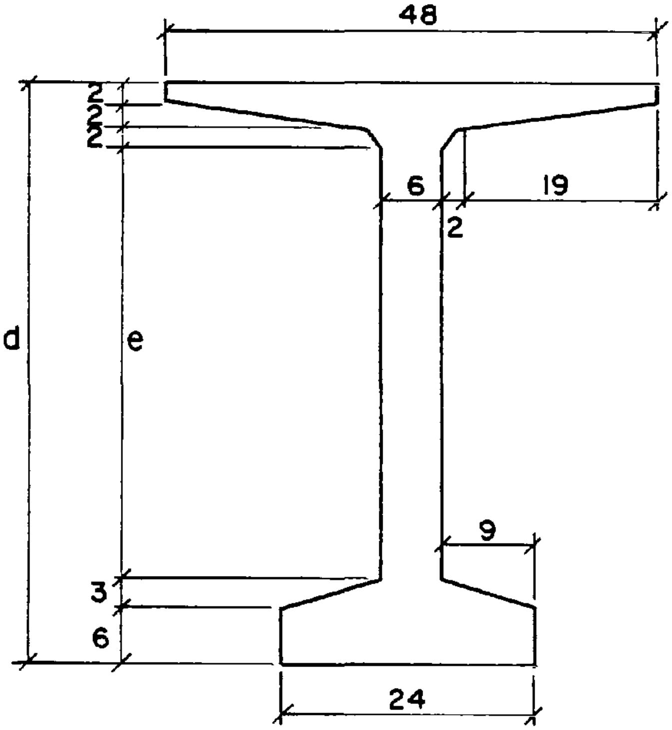


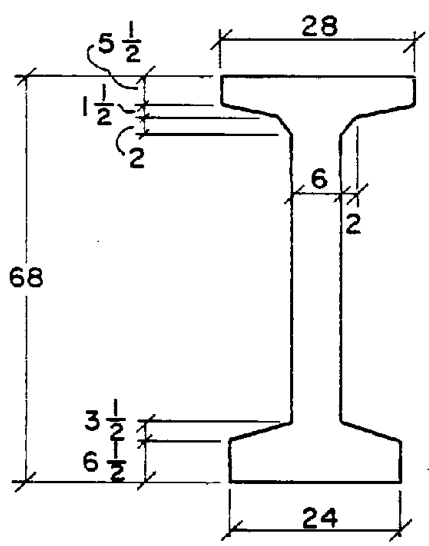
Figure 7.a Modified Girders Analyzed

All Dimensions in Inches
1 in = 25.4 mm



BT	48/6	60/6	72/6
d	48	60	72
e	33	45	57

Modified Bulb-T



Modified Colorado G 68/6

Figure 7.b Modified Girders Analyzed

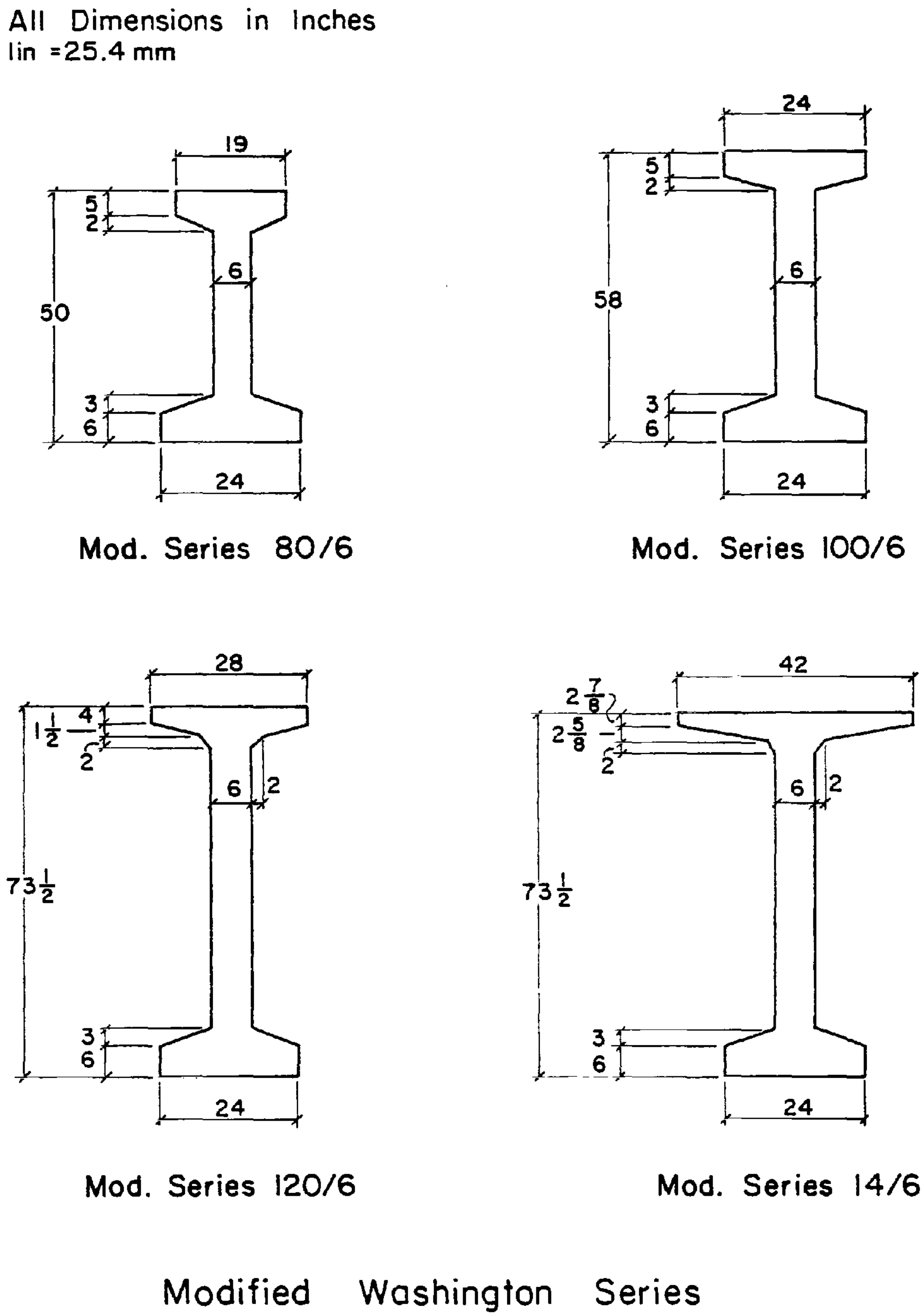


Figure 7.c Modified Girders Analyzed

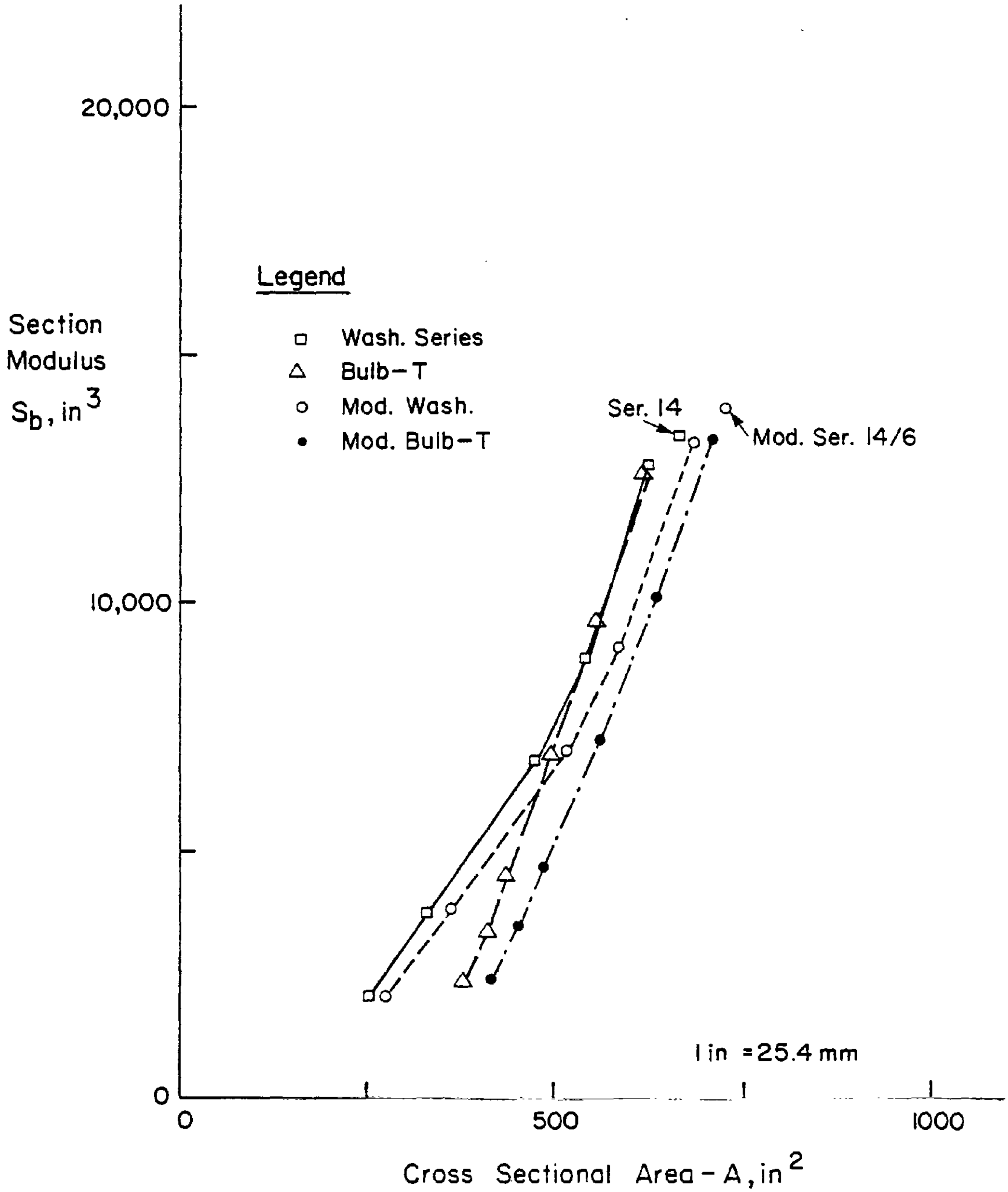


Figure 8 Relationship Between Modulus of Section for Bottom Fibers and Cross-Sectional Area for Washington Series and Bulb-T's

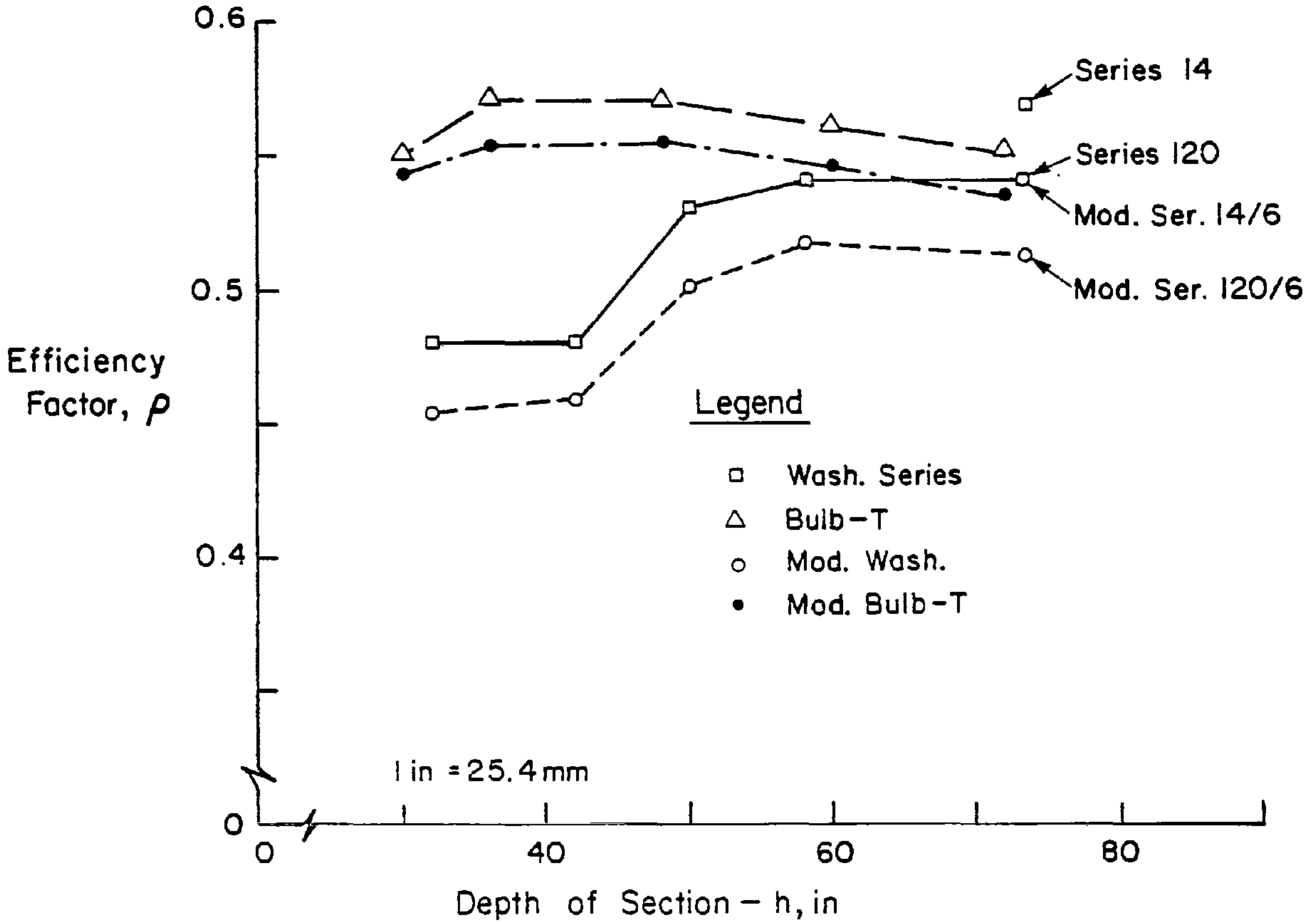


Figure 9 Variation of Efficiency Factor with Depth of Section for Washington Series and Bulb T's

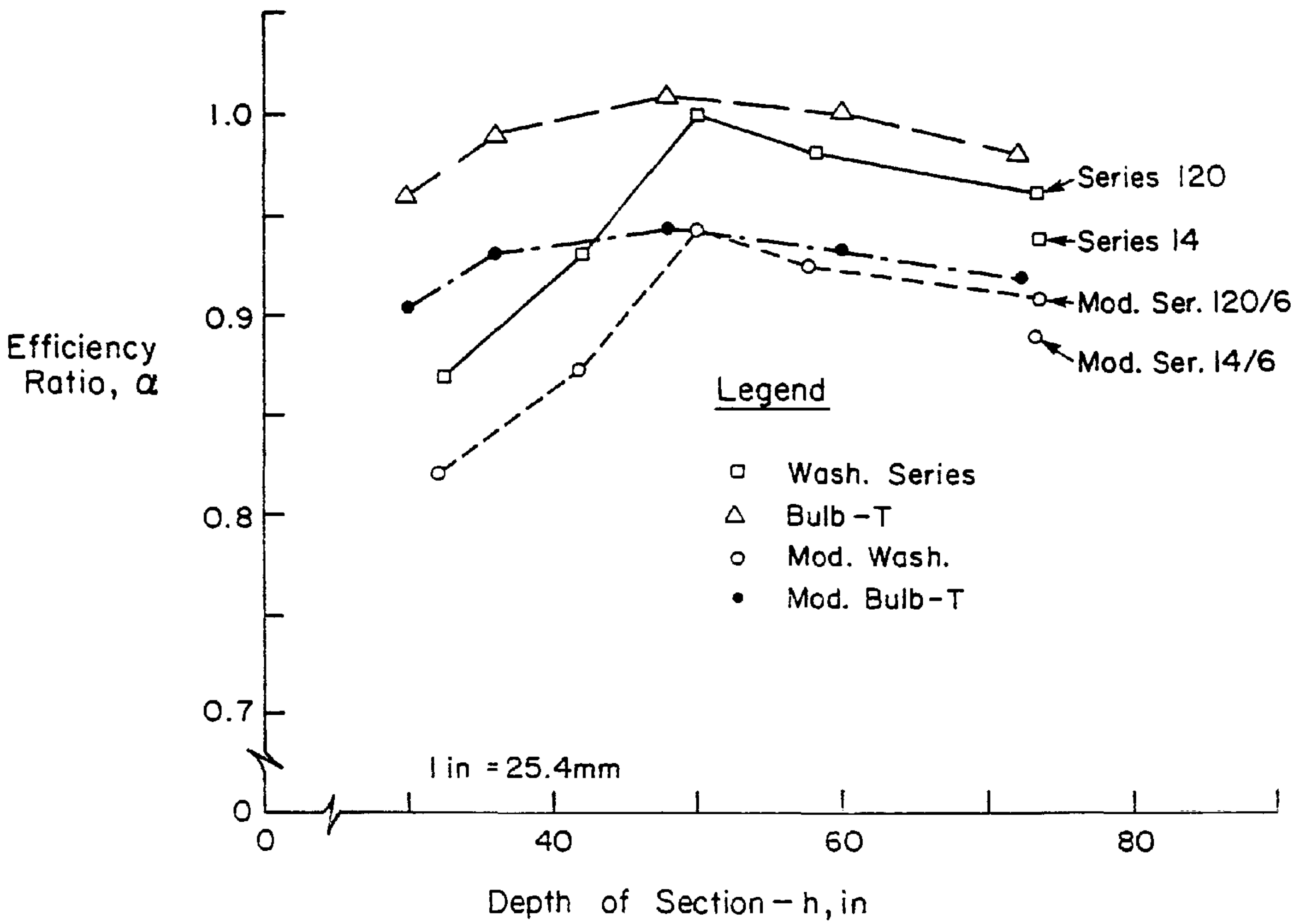


Figure 10 Variation of Efficiency Ratio with Depth of Section for Washington Series and Bulb-T's

Structural Parameters

The sections were evaluated through a detailed structural analysis. The following parameters were considered:

1. Girder spacing
2. Span length
3. Deck thickness
4. Concrete strength

Girder spacing was varied between 4.5 and 10 ft (1.37 and 3.05 m). Spans in excess of 80 ft (24.4 m) were considered. Deck thickness varied with girder spacing. Concrete strength for girders was varied between 5,000 and 7,000 psi (34.5 and 48.3 MPa).

Development of Computer Program

To evaluate the effect of each variable, a parametric study was carried out. The number of variables necessitated preparing a computer program to analyze each case and generate cost data.

The computer program, called BRIDGE, is described in detail in Appendix E. Program documentation, user's instructions, source listing and sample problems are all included in the Appendix. The following are highlights of the program.

Program BRIDGE requires input of the following data:

1. Geometric properties: Included are girder span, spacing, and cross section.
2. Materials properties: Included are concrete and strand characteristics.
3. Relative costs of materials: Materials considered are deck and girder concrete, deck transverse flexural reinforcement, and girder strands. Deck temperature reinforcement and girder web reinforcement are not considered in the cost analysis.

Data input is further simplified by making some of the above data optional. Where material properties and relative costs are not input in the program, default options are assigned internally by the program. These default options are summarized in the "User's Input Instructions" in Appendix E.

In addition to listing data input or information internally assigned, the computer program outputs the following information:

1. Allowable concrete stresses and strand stresses, both at prestress transfer and service load conditions
2. Deck thickness and reinforcement
3. Sectional properties for the girder (non-composite) section, and the composite (girder-deck) section
4. Dead and live load moments and impact factor
5. Required number of strands and corresponding midspan concrete and strand stresses, both at prestress transfer and service load conditions
6. Midspan camber or deflection
7. Weight of materials and cost index per unit surface area of bridge deck

The following assumptions were made in Program BRIDGE:

1. Design conforms to AASHTO Specifications.[8,12]
2. Live load consists of HS 20-44 loading.
3. Girders are simply supported.
4. A typical interior girder is considered.
5. Concrete deck is cast-in-place and acts compositely with the girder. Deck formwork is supported on the girder. In calculations of the composite section properties, the transformed area of strands is neglected.
6. Concrete compressive strength of the deck is constant and equal to 4000 psi (27.6 MPa) at 28 days.
7. Strands are Grade 270 (1,862 MPa) stress relieved with 1/2-in (12.7 mm) diameter and have an idealized trilinear stress-strain curve.
8. Total prestress losses are constant and equal 45,000 psi (310 MPa).
9. Cost of materials, labor, transportation, and erection of girders having concrete compressive strength between 5000 and 7000 psi (34.5 and 48.3 MPa) is assumed constant. The effect of increasing the girder concrete strength from 5000 to 7000 psi (34.5 to 48.3 MPa) on the in-place cost of the girder is negligible.

10. Relative unit costs of materials and labor are constant for the cost analysis.
11. Cost analysis comparisons are for the precast girder and a cast-in-place deck. Cost of substructure and approach fills are not considered.

Relative Unit Cost Indexes

Several factors affect the cost of the superstructure. They have been discussed under the heading "Availability and Cost". An assessment of local and regional factors was not possible within the scope of this project. However, a cost analysis was possible by comparing the cost of the recommended sections based on a common ground.

From survey data, an average cost for girder concrete, deck concrete, reinforcing steel, and prestressing strands was determined. Average costs included cost of materials and labor. For girder concrete, the cost also included transportation and erection. These average costs were then reduced to relative costs per pound of in-place material. The following relative unit costs for in-place materials (including labor) were used for the cost analysis.

Concrete (girders and deck)	1 unit/lb
Strands	8 units/lb
Reinforcing steel	9 units/lb
Epoxy coated reinforcing steel	12 units/lb

Girders were compared based on the same unit costs. In the cost analysis, top deck reinforcement was assumed to consist of epoxy coated bars. However, this is optional in Program BRIDGE. Top deck reinforcement can be specified as regular deformed bars.

The relative costs of materials were taken as the product of material weight and relative unit costs. The summation of relative costs of materials was then divided by deck area to give cost index per square foot.

Design Charts

Data generated by Program BRIDGE can be used to prepare different types of design aids. For preliminary designs, the relation between girder span, girder spacing, and required number of strands can be very useful.

Figure 11 shows the required number of strands versus span length for selected girder spacings of AASHTO Type VI section. A similar plot is shown in Figure 12 for Washington Series 14 girder. For comparable spans, the number of strands required in the Washington Series 14 section is considerably less than that required in the AASHTO Type VI.

Another type of design aid is shown in Figure 13. It depicts maximum spans that can be achieved at different girder spacing for four girder cross-sections. This figure indicates that for a given girder spacing, the longest spans are achieved using AASHTO Type VI girder. Design aids in the form of charts as shown in Figures 11 to 13 do not reflect the cost-effectiveness of the sections. The main purpose of Program BRIDGE is to generate cost analysis data for comparisons.

Optimum Cost Index Charts

Using Program BRIDGE, a cost chart was prepared for each of the sections considered in Phase II of the project. Same relative unit costs for in-place materials (material and labor) as well as material properties were assumed for all girders and decks. For the girders, the concrete compressive strength was assumed to be 4500 psi (31.0 MPa) at transfer (f'_{ci}), and 6000 psi (41.4 MPa) at 28 days (f'_c). For the deck, the concrete compressive strength at 28 days, f'_c, was assumed to be 4000 psi (27.6 MPa).

All cost charts were drawn to the same scale for comparison purposes. These charts are presented in Appendix F. A representative chart is given in Figure 14. It depicts cost index per square foot of deck versus span length for AASHTO Type VI girder. The solid lines are for selected girder spacing.

Cost curves can be drawn for different girder spacings varying between 4.5 and 10 ft (1.37 and 3.05 m). If for each span

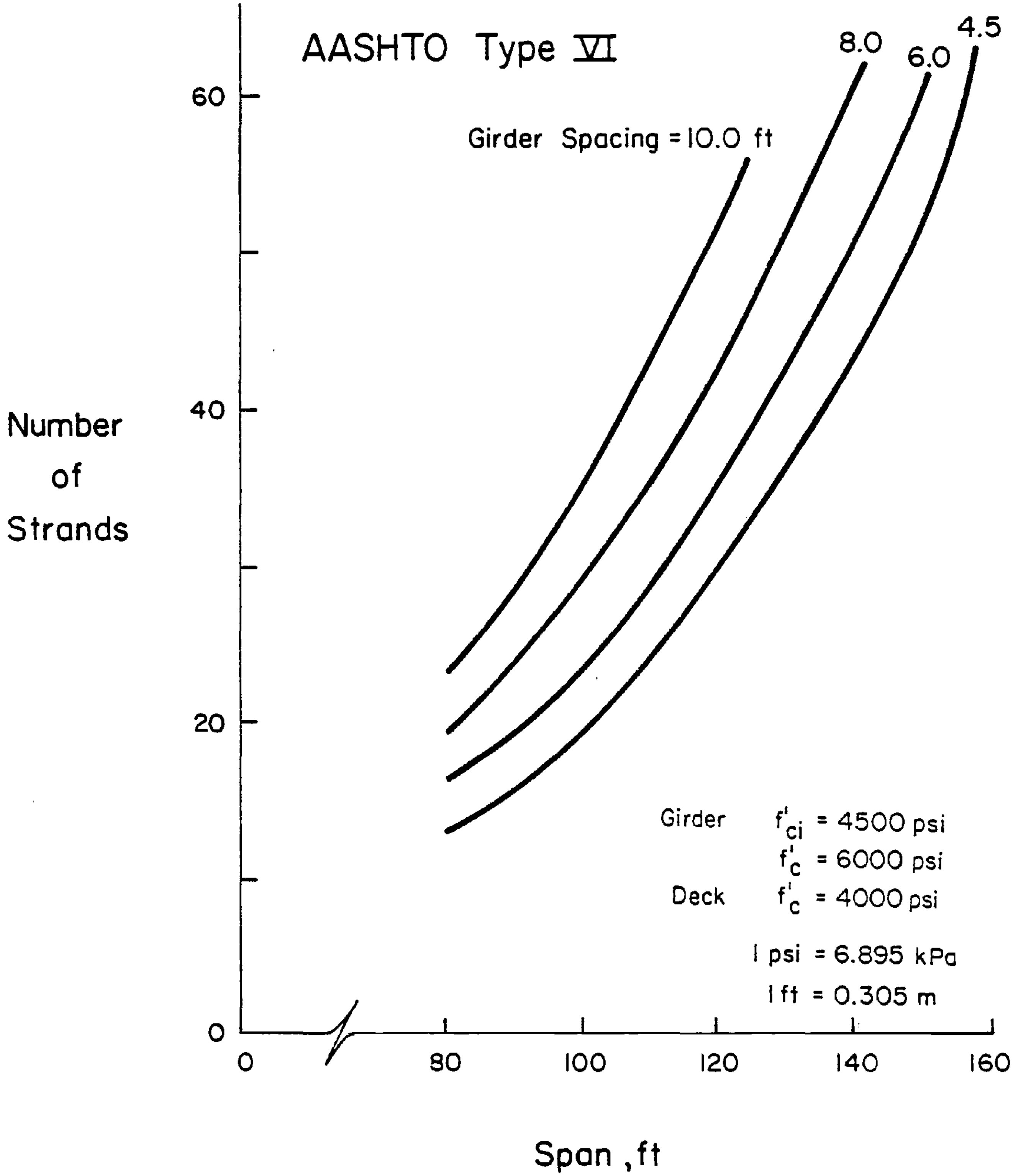


Figure 11 Required Number of Strands in AASHTO Type VI Girder

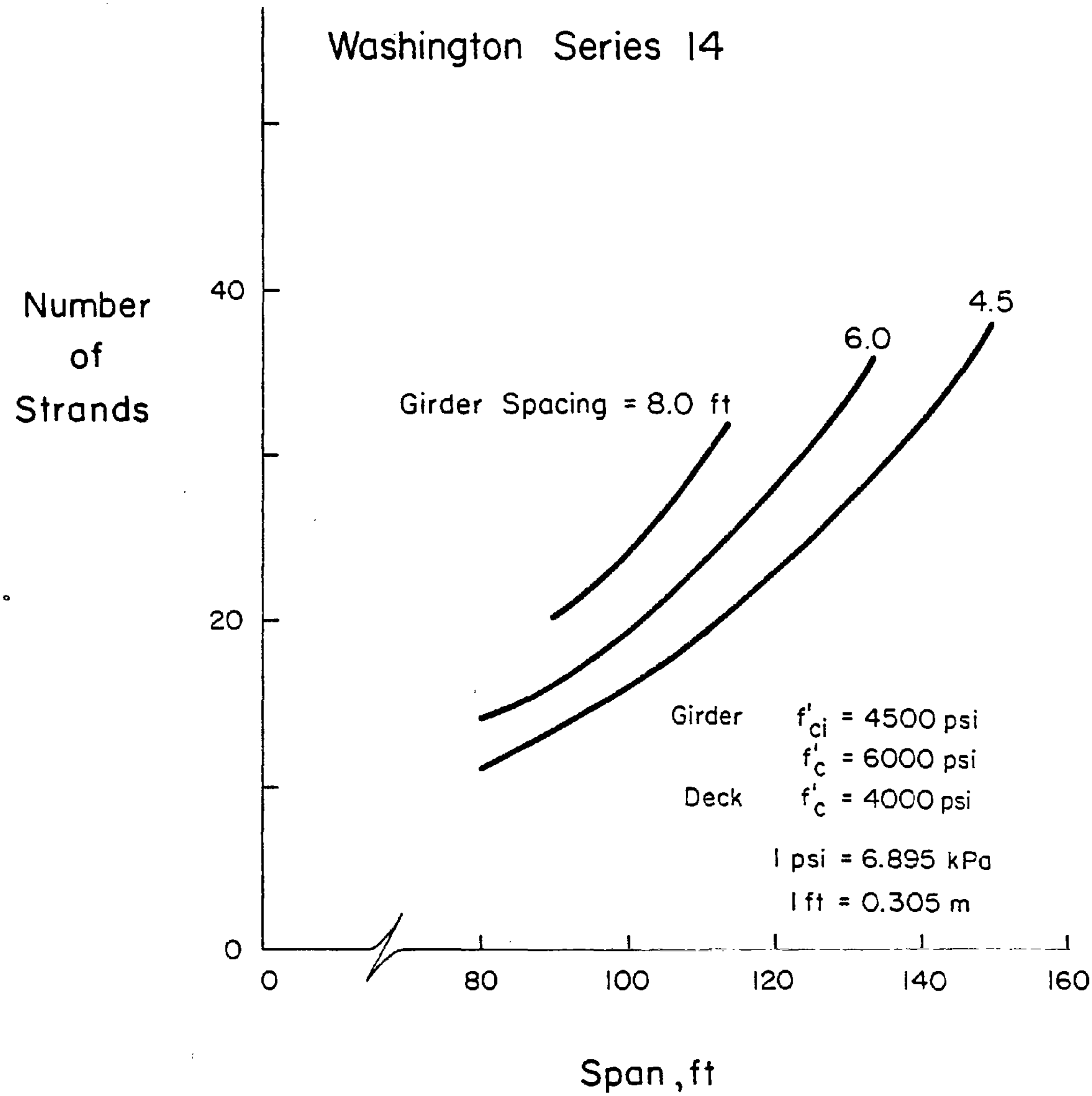


Figure 12 Required Number of Strands in Washington Series 14 Girder

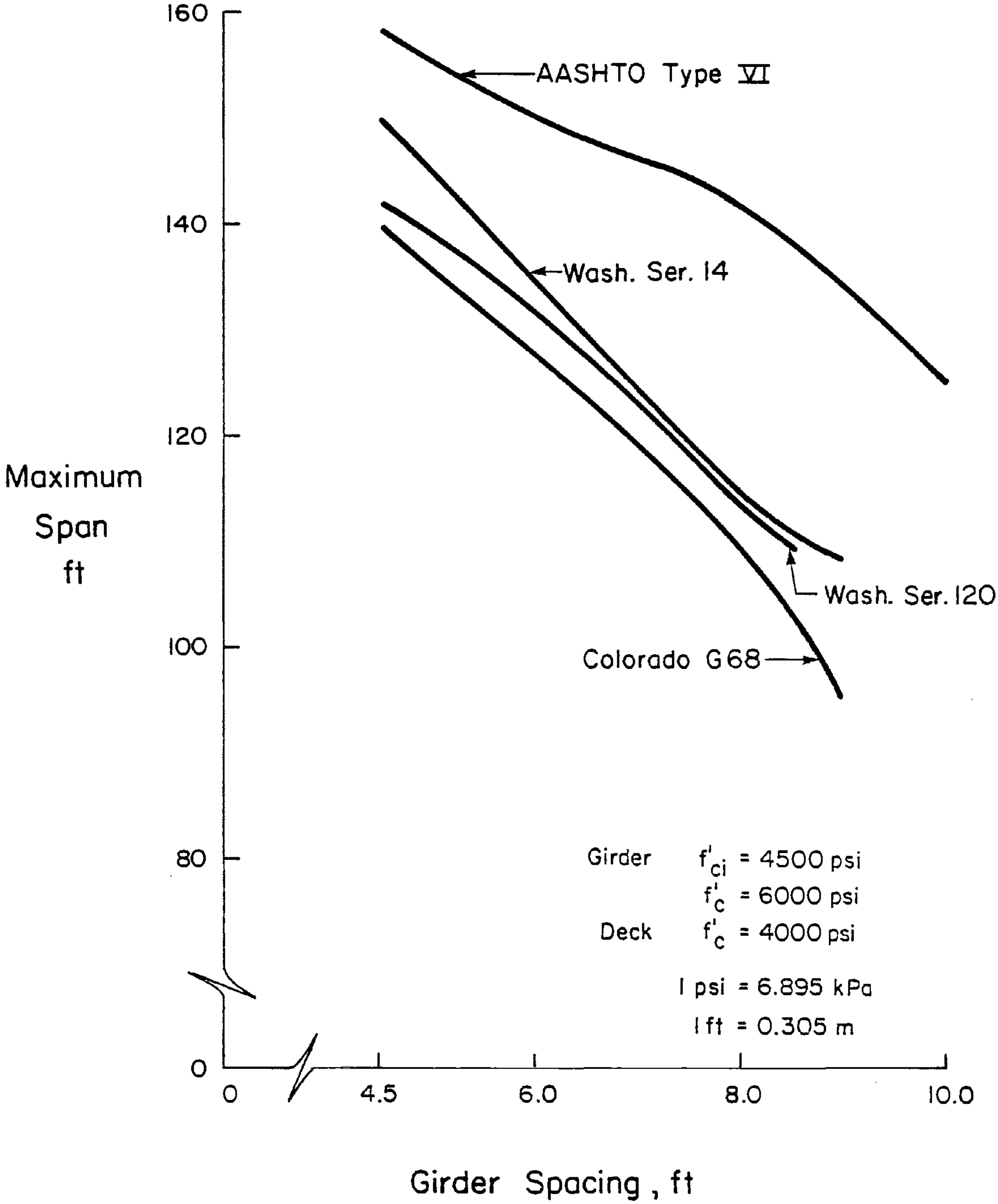


Figure 13 Variation of Maximum Span with Girder Spacing

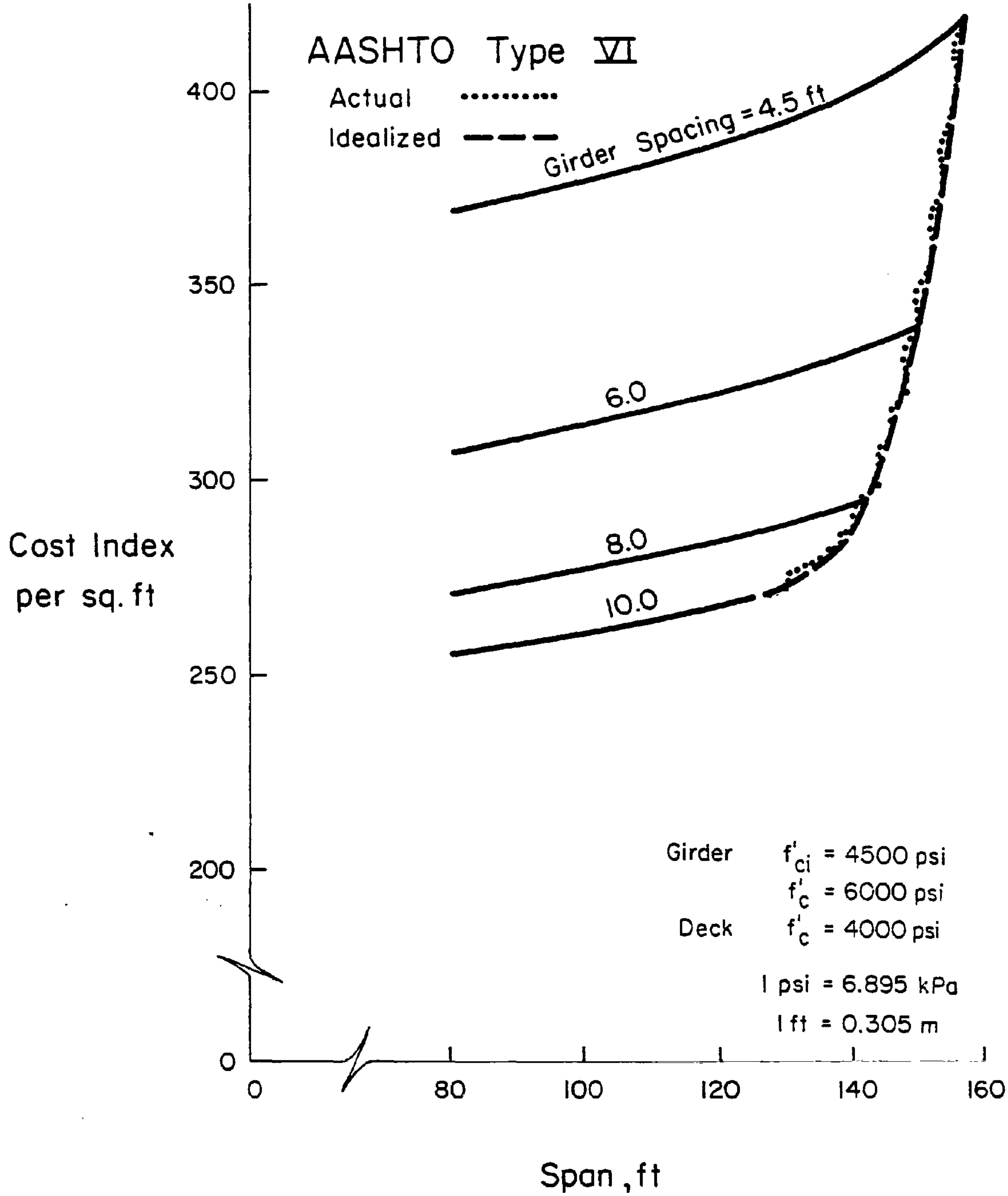


Figure 14 Cost Chart for AASHTO Type VI Girder

the least cost index points are joined, an "optimum cost curve" is obtained. These points will correspond to different girder spacings, except where the maximum girder spacing of 10 ft (3.05 m) controls.

In Figure 14, the dotted curve is an actual optimum cost curve. It was obtained through a detailed analysis. Discontinuities of this curve are due to several factors affecting the cost index. The main factor is the deck thickness.

As girder spacing increases, deck thickness increases in 1/2-in (12.7 mm) increments. The result is a sudden change in weight of deck and composite sectional properties. In turn, section properties affect member design and cost of materials. In addition required number of strands is computed to the whole nearest number. Although the cost of one strand is negligible compared to the overall cost, each strand has an important effect on the concrete stress level.

In this report, discontinuities in the optimum cost curves are ignored and an idealized curve is plotted for all cost charts. In Figure 14, this idealized curve is shown as a dashed line.

Figure 14 illustrates the effect of girder spacing on cost. For a given span, as girder spacing increases, unit cost per square foot of bridge deck decreases. For an AASHTO Type VI section, if girders are spaced 10 ft (3.05 m) apart, the cost per unit area of bridge deck is 30% less than if girders are spaced 4.5 ft (1.37 m) apart. Therefore it would be most economical to place girders at the largest practical girder spacing.

For sections analyzed in this report, the cost of girders represents a significant portion of the cost of the bridge superstructure. For example, if AASHTO Type VI girders are placed at a spacing of 10 ft (3.05 m), the in-place cost of the girders is about 40% of the overall in-place cost of girders and deck. However, if AASHTO Type VI girder spacing is reduced to 4.5 ft (1.37 m), then the cost of the girders is about 65% of the in-place cost of girders and deck. The overall in-place cost with a girder spacing of 4.5 ft (1.37 m) is 45% higher than with a girder spacing of 10 ft (3.05 m).

Cost Effectiveness Comparisons

Optimum cost curves were used to compare the cost effectiveness of selected girders. Spans in excess of 80 ft (24.4 m) were investigated. Girder spacing considered ranged between 4.5 and 10 ft (1.37 and 3.05 m).

Comparisons of Girder Cross Sections

AASHTO Girders - Optimum cost curves for AASHTO Types IV, V, and VI girders are shown in Figure 15. For spans from 80 to 100 ft (24.4 to 30.5 m), the cost index per square foot is about the same for the three sections, when used at maximum girder spacing. Detailed analysis reveals that a Type IV girder can be used at a maximum girder spacing of 9.0 ft (2.74 m). Types V and VI girders can be used at the limiting maximum girder spacing of 10 ft (3.05 m).

Figure 15 indicates that for spans larger than 100, 125 and 140 ft (30.5, 38.1, and 42.7 m), the cost index increases rapidly for AASHTO girders Types IV, V, and VI, respectively. Maximum spans that could be achieved at the limiting minimum girder spacing of 4.5 ft (1.37 m) are 119, 144, and 158 ft (36.3, 43.9, and 48.2 m).

Modified AASHTO Girders - Optimum cost curves for Modified AASHTO Types IV, V and VI girders are shown in Figure 16. These girders have a 6-in (152 mm) thick web. Span capabilities of the Modified sections are comparable to those of the corresponding AASHTO girders. However, the modified girders lead to savings of about 6% on the overall cost of the in-place girders and deck.

Colorado Sections - Colorado's G54 and G68 girders are compared in Figure 17. For an 80 ft (24.4 m) span, with maximum girder spacing, the cost index is the same for both girders. Detailed analysis shows that maximum girder spacing is 8.2 and 9.0 ft (2.5 and 2.74 m) for G54 and G68 girders, respectively. At a girder spacing of 4.5 ft (1.37 m), maximum girder spans are 122 and 140 ft (37.2 and 42.7 m) for G54 and G68, respectively.

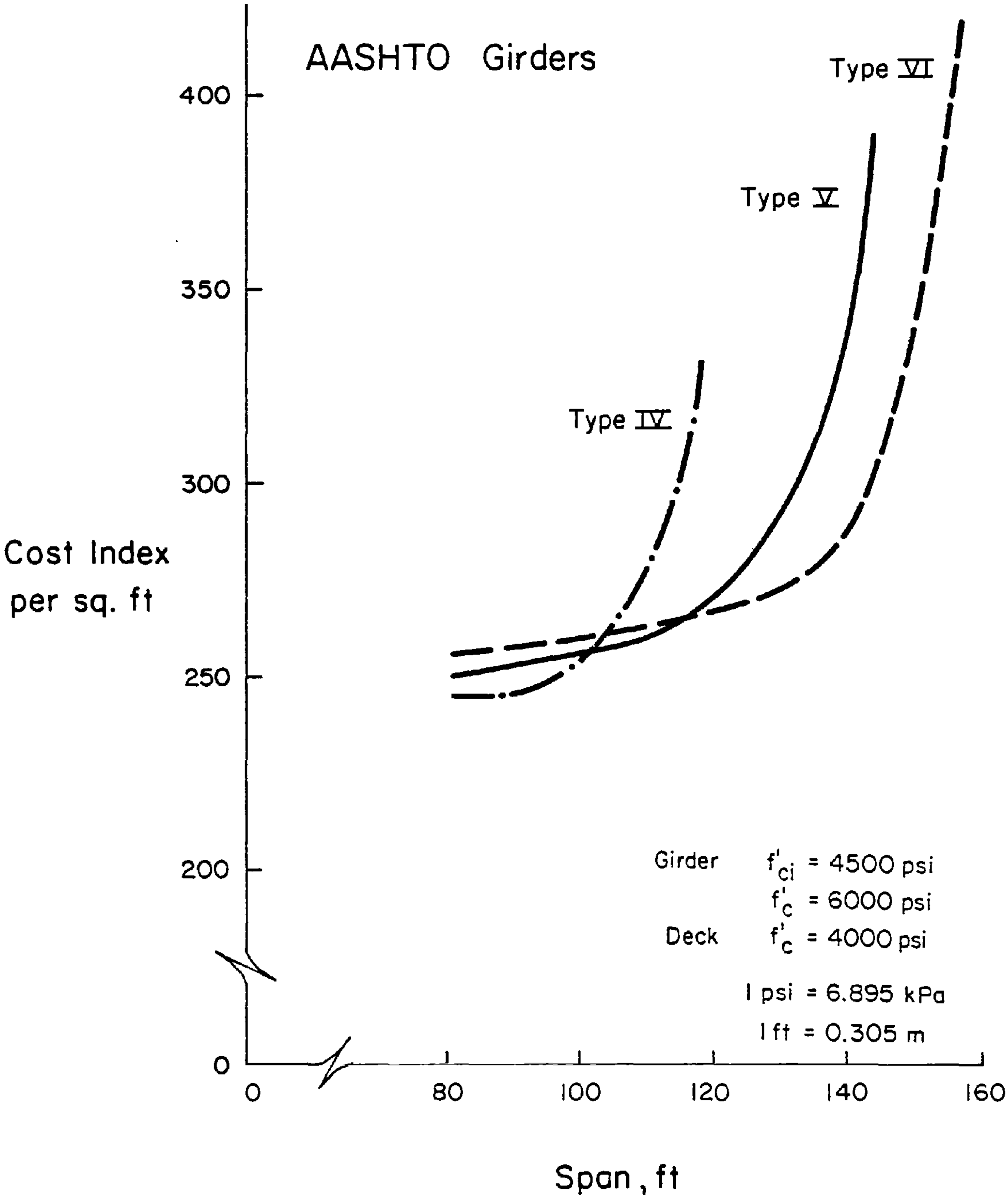


Figure 15 Optimum Cost Curves for AASHTO Girders

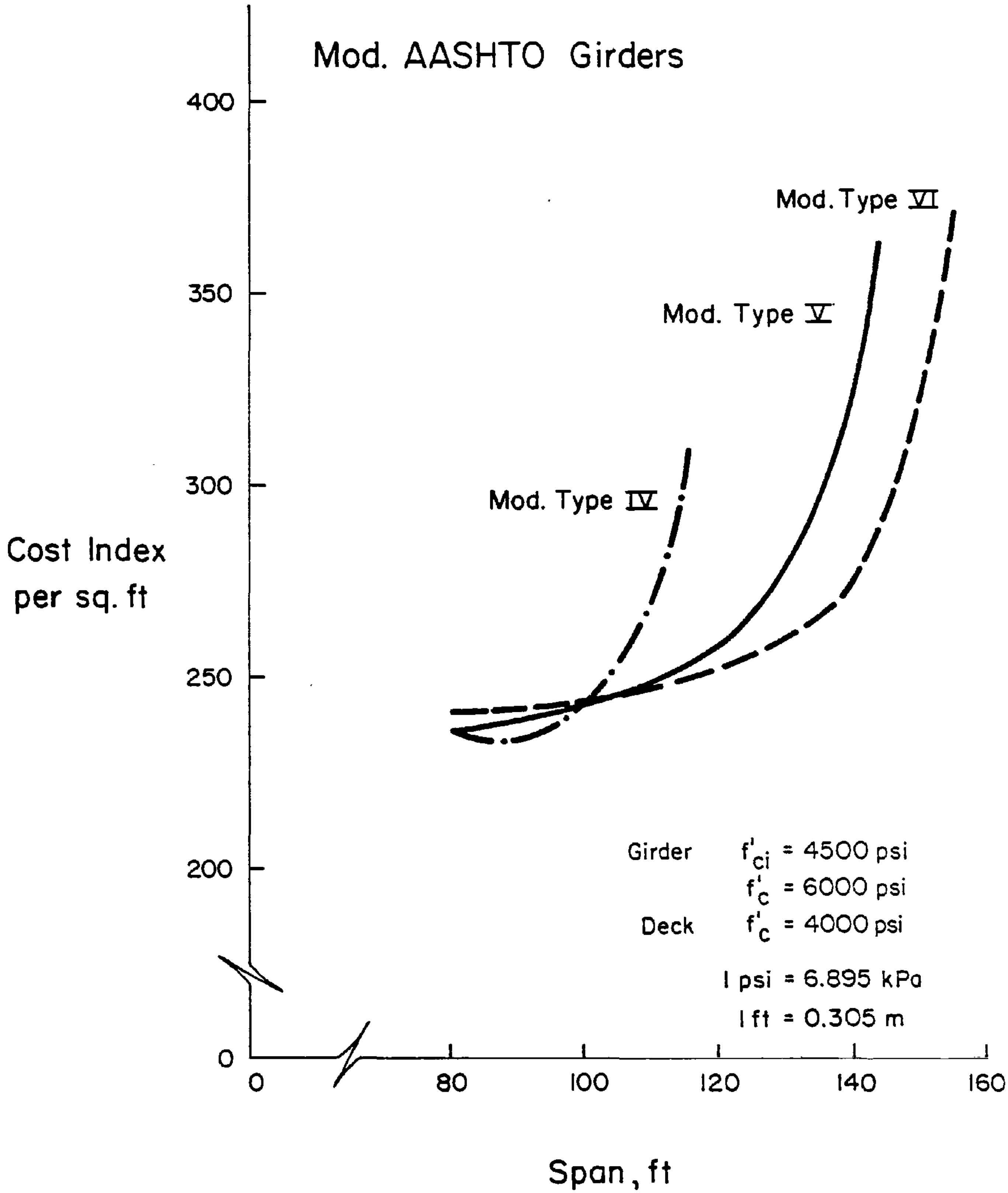


Figure 16 Optimum Cost Curves for Modified AASHTO Girders

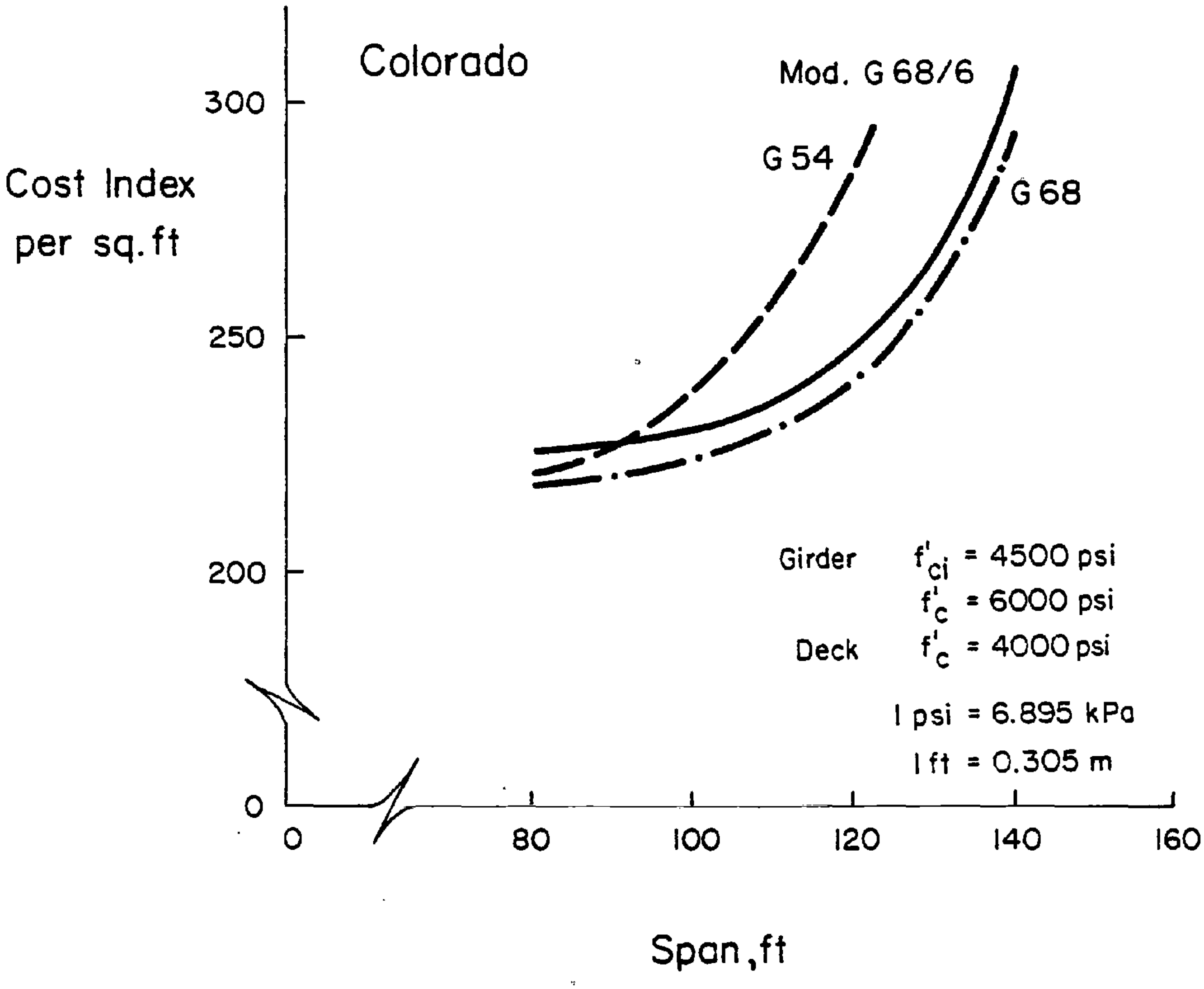


Figure 17 Optimum Cost Curves for Colorado Girders and Modified G68/6 Girder

Modified Colorado Section - An optimum cost curve for Modified G68/6 girder is also plotted in Figure 17. This section is similar to G68 but has a 6 in (152 mm) web. Span capabilities are the same for G68 and G68/6. However, for similar spans, Girder G68/6 costs about 3% more than G68.

Washington Series - Cost comparisons of Washington Series girders are made in Figure 18. In the range of 80 to 90 ft (24.4 to 27.4 m), the cost is about the same for Series 80, 100, 120 and 14. Maximum girder spacing is 7.0, 7.5, 8.5, and 9.0 ft (2.13, 2.29, 2.59, and 2.74 m), respectively. At a girder spacing of 4.5 ft (1.37 m), maximum spans are 106, 122, 142, and 150 ft (32.3, 37.2, 43.3, and 45.7 m) for Series 80, 100, 120, and 14, respectively. Figure 18 shows that Series 14 girder is slightly more cost-effective than Series 120 girder. Newly developed Series 14 girder is a replacement for Series 120 girder.

Modified Washington Series - Modified Washington Series 80/6, 100/6, 120/6 and 14/6 girders are compared with Washington Series 80, 100, 120, and 14 in Figures 19a and 19b. The modified sections with 6 in (152 mm) web cost 3 to 5% more than similar sections having 5 in (127 mm) web. The higher costs correspond to the heavier sections. Apart from this cost difference, Washington Series girders and their modified counterparts have about same maximum span and maximum girder spacing capabilities.

Bulb-T's - Optimum cost curves for Anderson's Bulb-T's are plotted in Figure 20. For spans of 80 to 85 ft (24.4 to 25.9 m), the cost is about the same for BT48, BT60, and BT72. Maximum girder spacing for 80 ft (24.4 m) spans is 7.0, 8.0, and 8.8 ft (2.13, 2.44, and 2.68 m). At a girder spacing of 4.5 ft (1.37 m), maximum achievable spans are 100, 127, and 142 ft (30.5, 38.7, and 43.3 m).

Modified Bulb-T's - Modified Bulb-T's BT60/6 and BT72/6 girders are compared with BT60 and BT72 girders in the cost chart of Figure 21. Bulb-T's with 6-in (152 mm) thick webs cost about 4% more than Bulb-T's with 5-in (127 mm) thick webs.

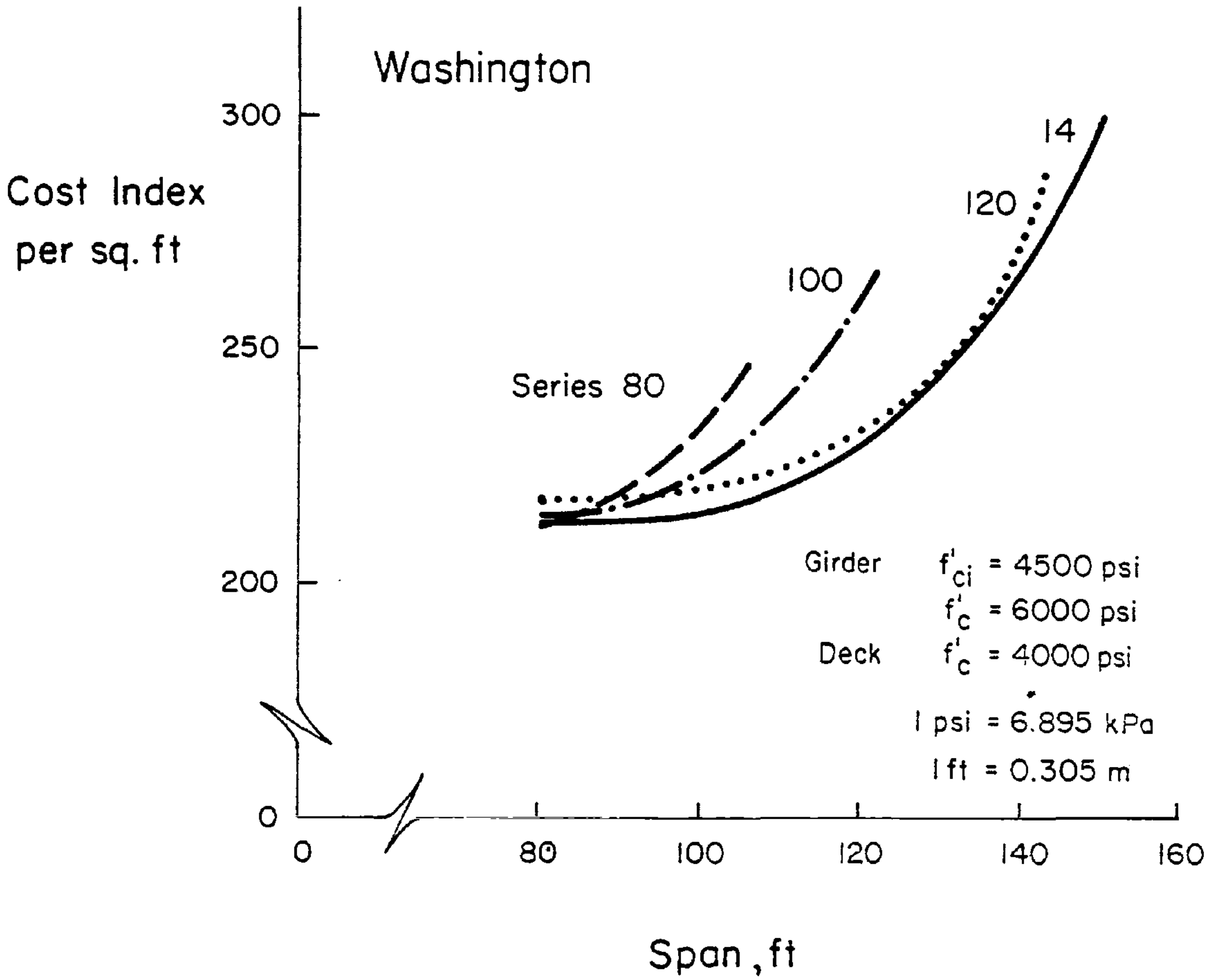


Figure 18 Optimum Cost Curves for Washington Series

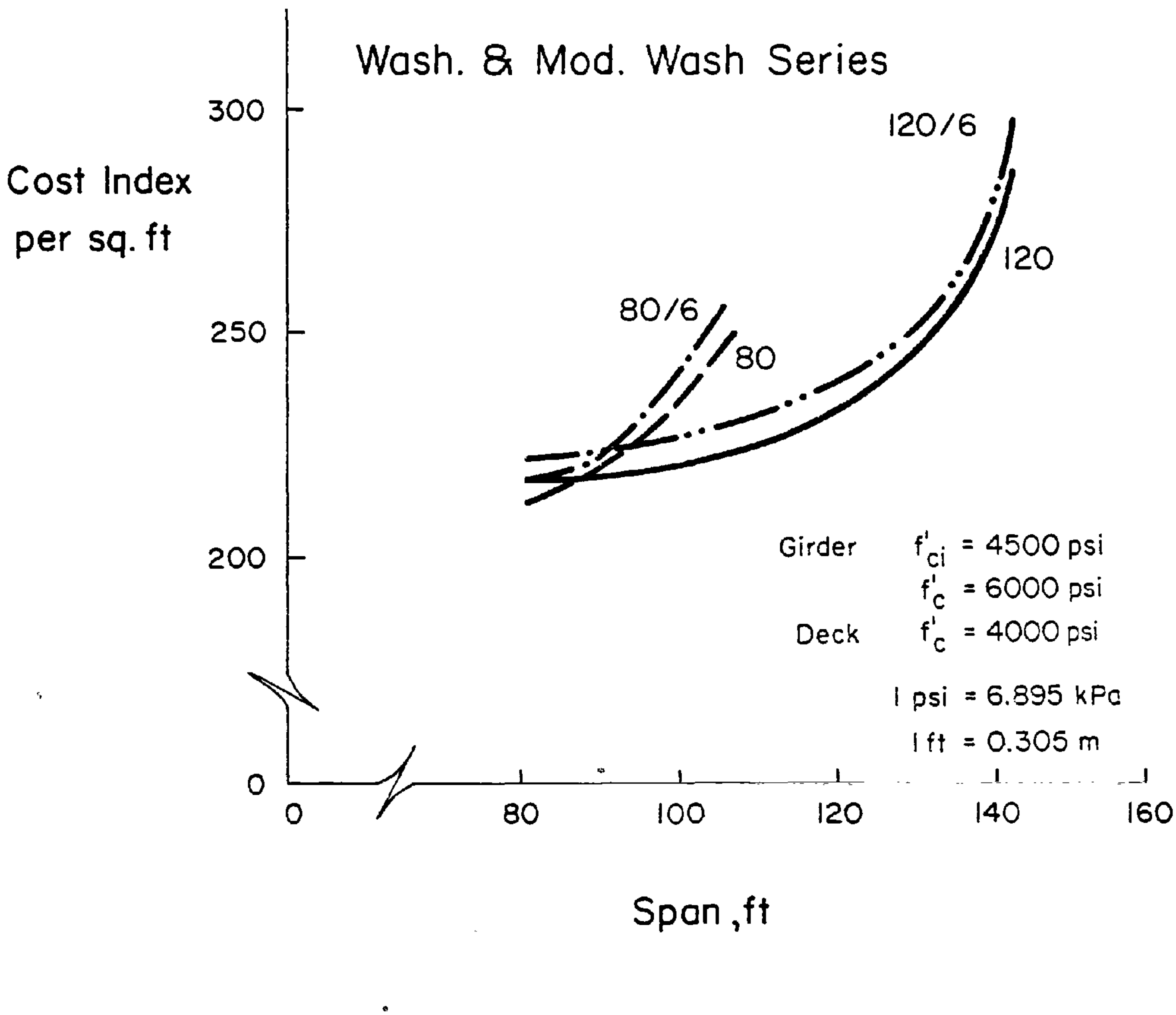


Figure 19.a Comparison of Optimum Cost Curves for Washington and Modified Washington Series

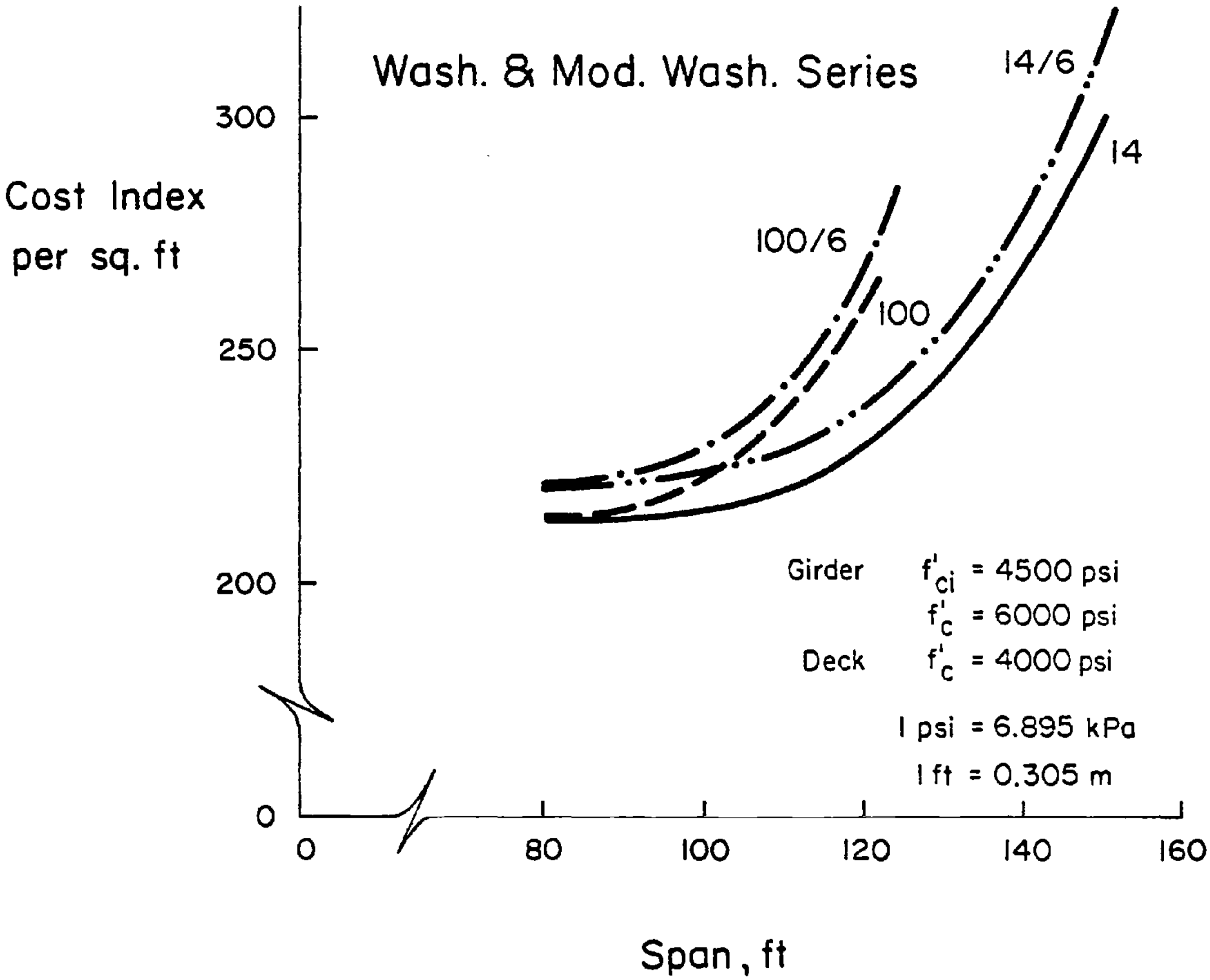


Figure 19.b Comparison of Optimum Cost Curves for Washington and Modified Washington Series

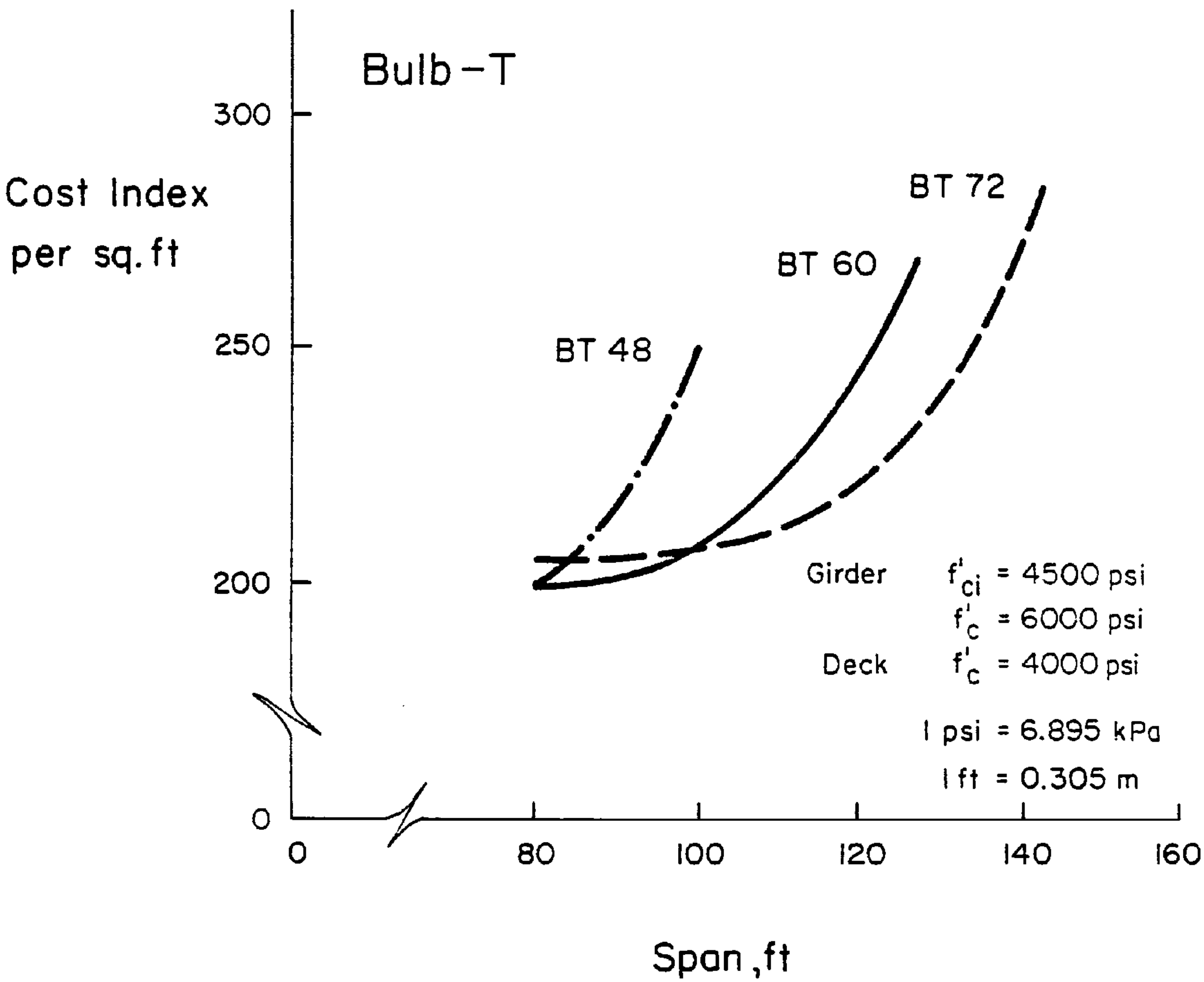


Figure 20 Optimum Cost Curves for Anderson's Bulb-T's

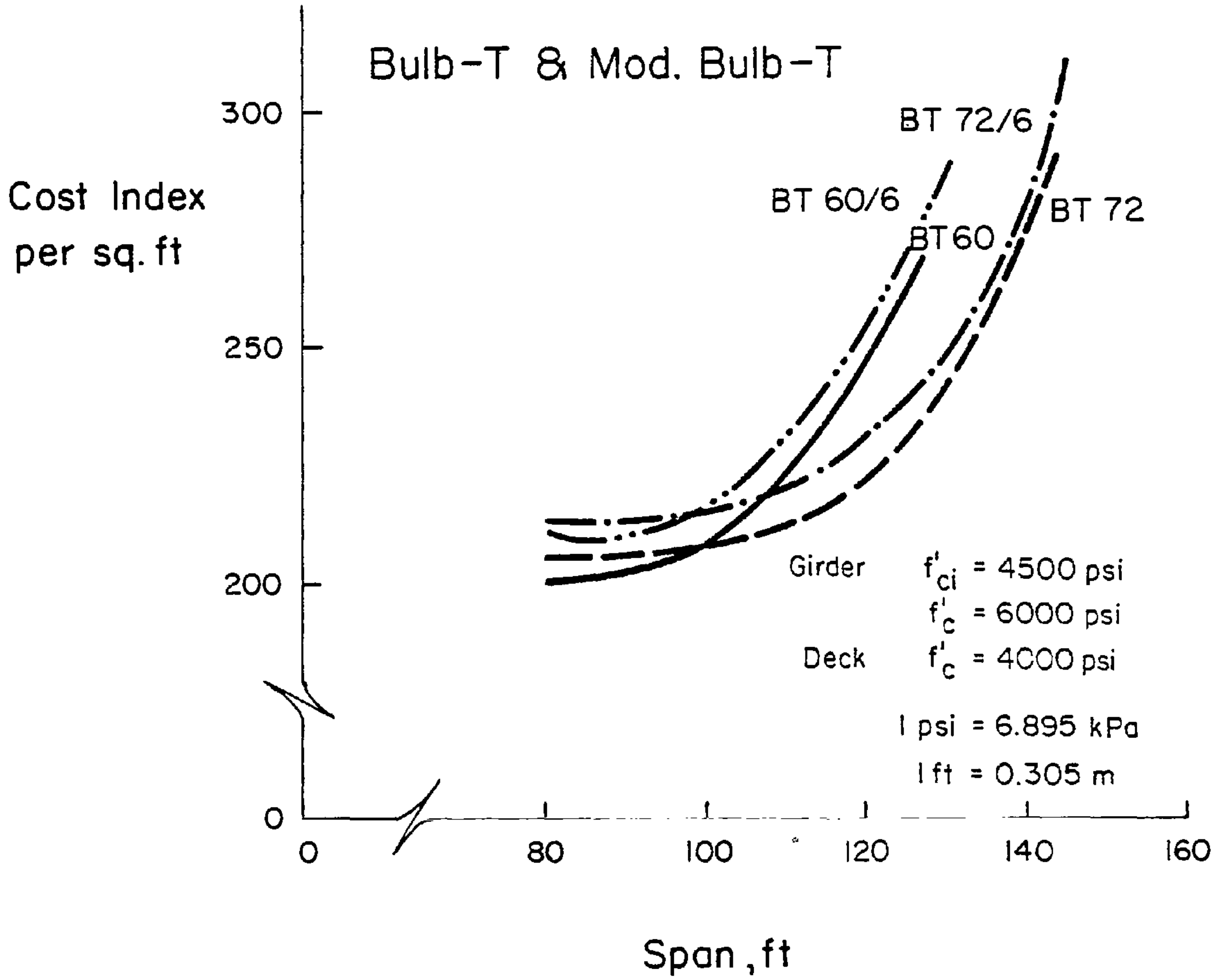


Figure 21 Comparison of Optimum Cost Curves for Bulb-T's and Modified Bulb-T's

Overall Comparisons - Optimum cost curves for AASHTO Type VI, Colorado's G68, Washington Series 14, and Bulb-T BT72 girders are compared in Figure 22. These girders are intended for use for spans in excess of 100 ft (30.5 m). Figure 22 indicates that Bulb-T BT72 is the most economical for spans up to 135 ft (41.2 m), and the AASHTO Type VI girder is the most expensive.

Modified Girders G68/6, Series 14/6, and BT72/6 are compared with AASHTO Type VI girder in Figure 23. For spans up to 140 ft (42.7 m), Modified Bulb-T BT72/6 is the most economic and is on the average about 3% cheaper than a Modified Series 14/6 girder.

Comparisons of cost index for sections plotted in Figures 22 and 23 relative to cost index of AASHTO Type VI girder are given in Table 1. These comparisons are made for different girder spans. Cost ratios for spans of 80 and 90 ft (24.4 and 27.4 m) are not shown because these heavy girders are not used for these spans. The cost ratios tabulated indicate that savings up to 20% can be achieved on the in-place total cost of girder and deck by using Bulb-T girders instead of the AASHTO Type VI girder. For spans in excess of 100 ft (30.5 m), Bulb-T 72, Washington Series 14, and Modified BT72/6 and Series 14/6 yield the least cost, the first being the most economical.

Web Thickness - The above comparisons between Bulb-T's, Washington Series, and Colorado G68 girders with 5-in (127 mm) thick webs and similar sections with 6-in (152 mm) thick webs indicate that girders with 6-in (152 mm) thick webs cost 3 to 5% more than similar girders having 5 in (127 mm) webs. However, sections with 6 in (152 mm) web would be easier to manufacture in all regions of the United States according to the survey results of Phase I. Their lateral stiffness is also improved. Therefore, they would be more stable during transportation.

Modified Bulb-T's and Modified Washington Series - Optimum cost curves for Modified Bulb-T's and Modified Washington Series girders are plotted in Figure 24. Up to spans of 140 ft (42.7 m), Modified Bulb-T's are most economical. For spans

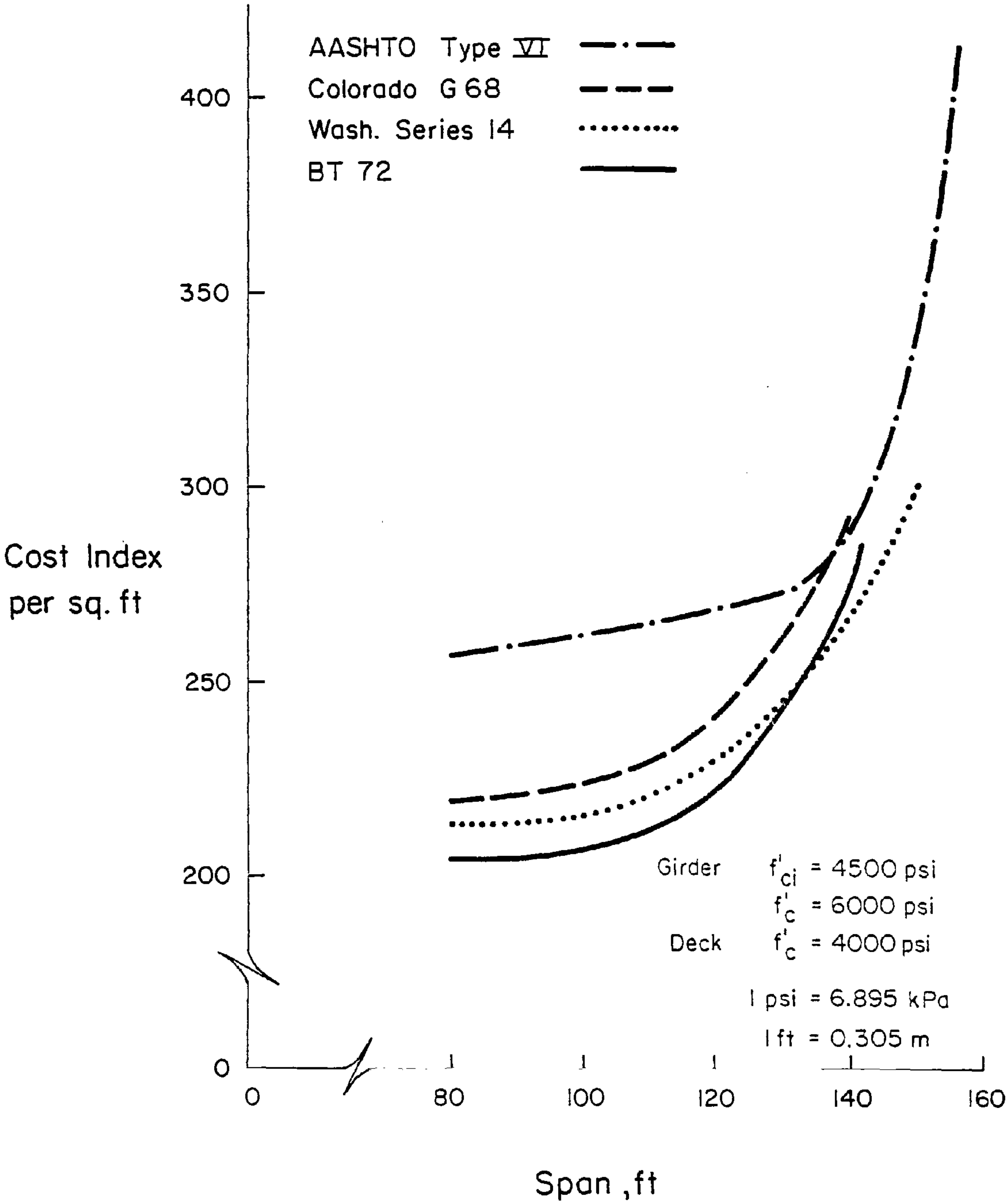


Figure 22 Comparison of Optimum Cost Curves for AASHTO Type VI, Colorado G68, Washington Series 14 and Bulb-T BT72

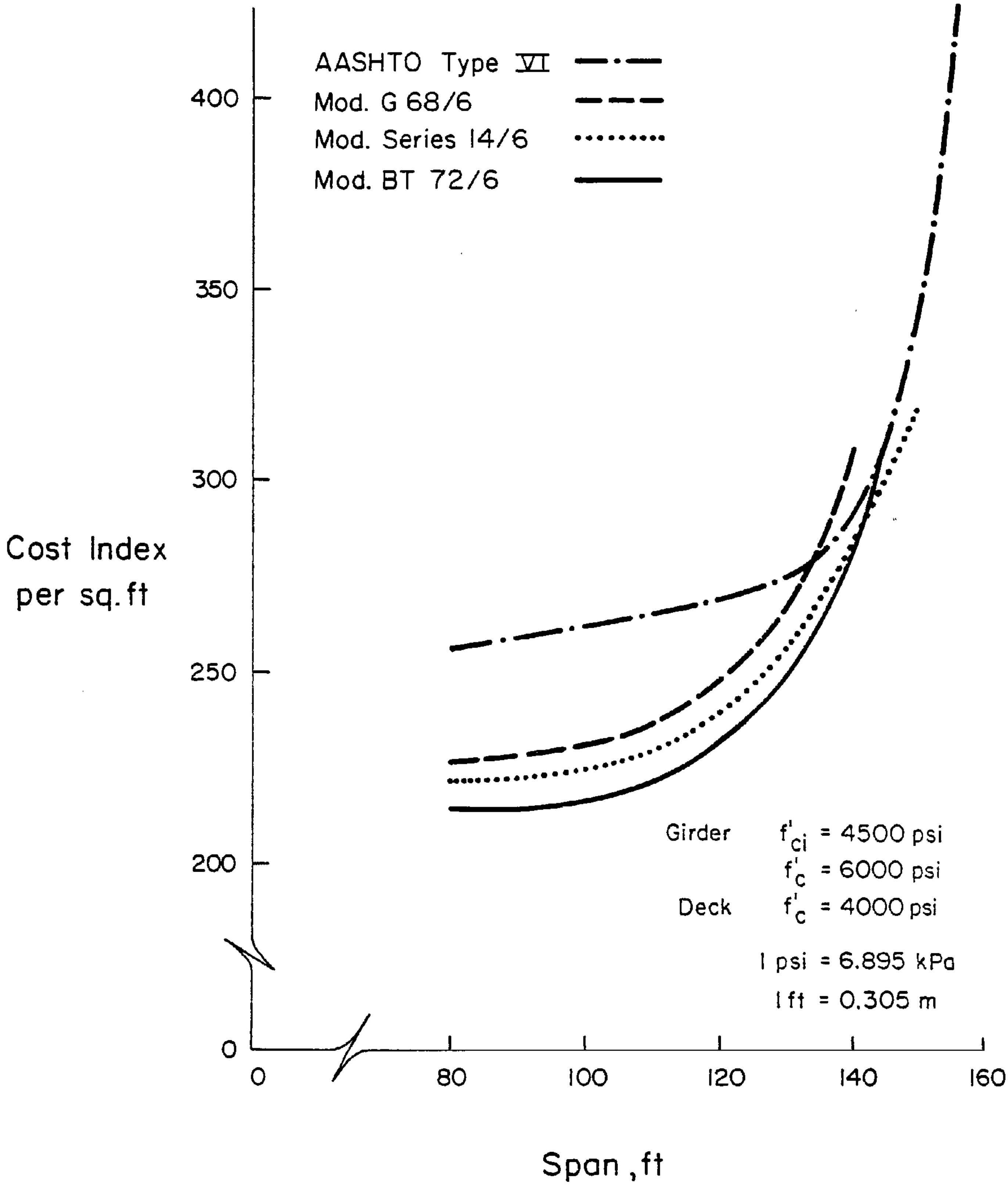


Figure 23 Comparison of Optimum Cost Curves for AASHTO Type VI and Modified G68/6, Series 14/6 and BT 72/6 Girders

TABLE 1 - COST RELATIVE TO TYPE VI GIRDER

Cross Section	Span, ft				
	100	110	120	130	140
AASHTO Type VI	1.00	1.00	1.00	1.00	1.00
Colorado G68	0.86	0.87	0.90	0.96	1.01
Wash. Series 14	0.83	0.83	0.86	0.90	0.92
Bulb-T BT72	0.79	0.80	0.83	0.89	0.95
Mod. G68/6	0.88	0.89	0.93	0.98	1.06
Mod. Ser. 14/6	0.86	0.86	0.90	0.94	0.98
Mod. BT72/6	0.83	0.83	0.87	0.92	0.98

1 ft = 0.305 m

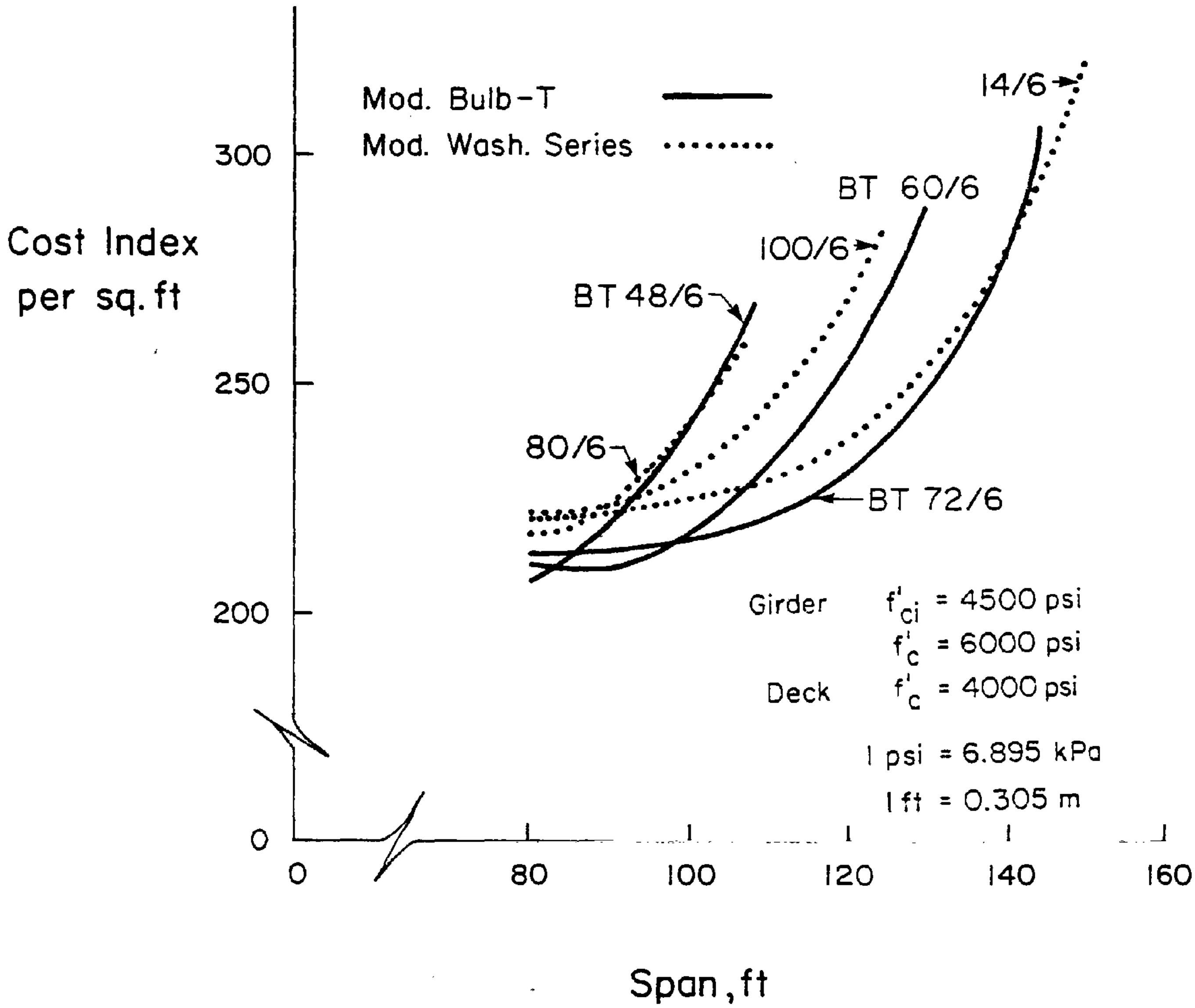


Figure 24 Comparison of Optimum Cost Curves for Modified Bulb-T's and Modified Washington Series

from 140 to about 150 ft (42.7 to 45.7 m), Modified Series 14/6 is slightly cheaper than Modified BT72/6.

Modified Bulb-T's and AASHTO Girders - Modified Bulb-T's are compared to the AASHTO Sections in Figure 25. For spans from 80 to 120 ft (24.4 to 36.6 m), Modified Bulb-T's yield savings of about 17% over the AASHTO girders. For spans of 120 to 140 ft (36.6 to 42.7 m), cost savings vary from 17 to 2%. These comparisons indicate that considerable savings can be achieved by using Modified Bulb-T's rather than AASHTO girders for spans up to 140 ft (42.7 m).

Effect of Concrete Strength

In all the above comparisons, the girder's concrete compressive strength was assumed to be 6000 psi (41.4 MPa). Some girders were analyzed assuming 5000 and 7000 psi (34.5 and 48.3 MPa) concrete. The effect of concrete compressive strength on optimum cost curves is illustrated in Figures 26 and 27. These figures indicate that by increasing the girder's concrete compressive strength, maximum span capability of a section is increased.

By increasing the concrete compressive strength from 5000 to 7000 psi (34.5 to 48.3 MPa), the maximum span of AASHTO Type IV girder is increased by about 15 ft (4.6 m). For AASHTO Type VI girder, maximum span range is increased by about 22 ft (6.7 m). This is equivalent to increasing the span capability by about 7% for each 1000 psi (6.9 MPa). However, this increase in span capability is associated with an increase in cost per unit area of bridge deck.

Effect of Bundling Strands

In all the above comparisons, strands were assumed spaced 2 in (50.8 mm) on center at midspan. Strands were positioned as low as possible in the section to obtain maximum eccentricity of prestressing force. Colorado G54 and AASHTO Type VI girders were also analyzed assuming that strands were bundled. They were positioned 1/2 in (127 mm) on center to produce maximum eccentricity of prestress. Minimum clear concrete cover for

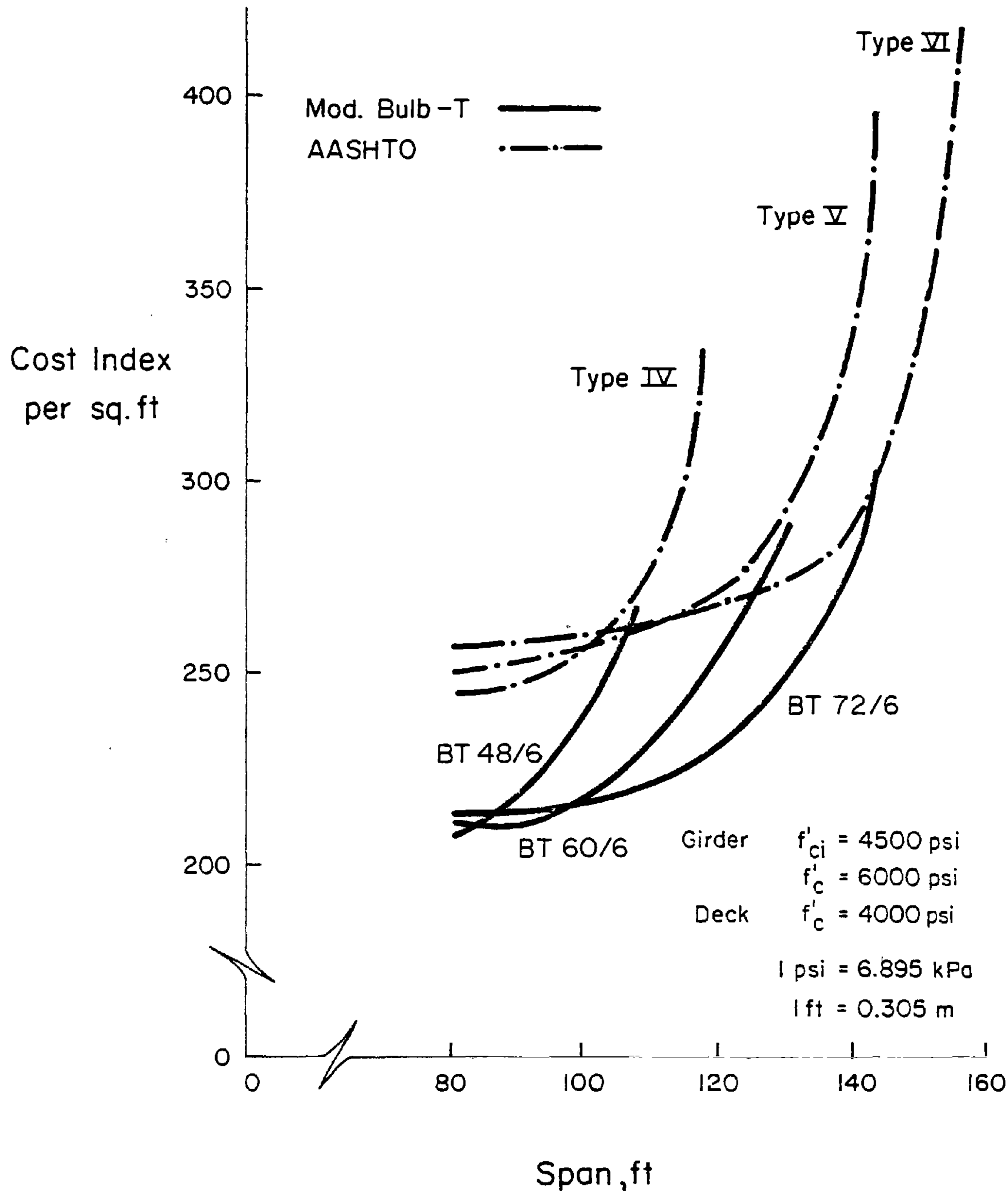


Figure 25 Comparison of Optimum Cost Curves for Modified Bulb-T's and AASHTO Girders

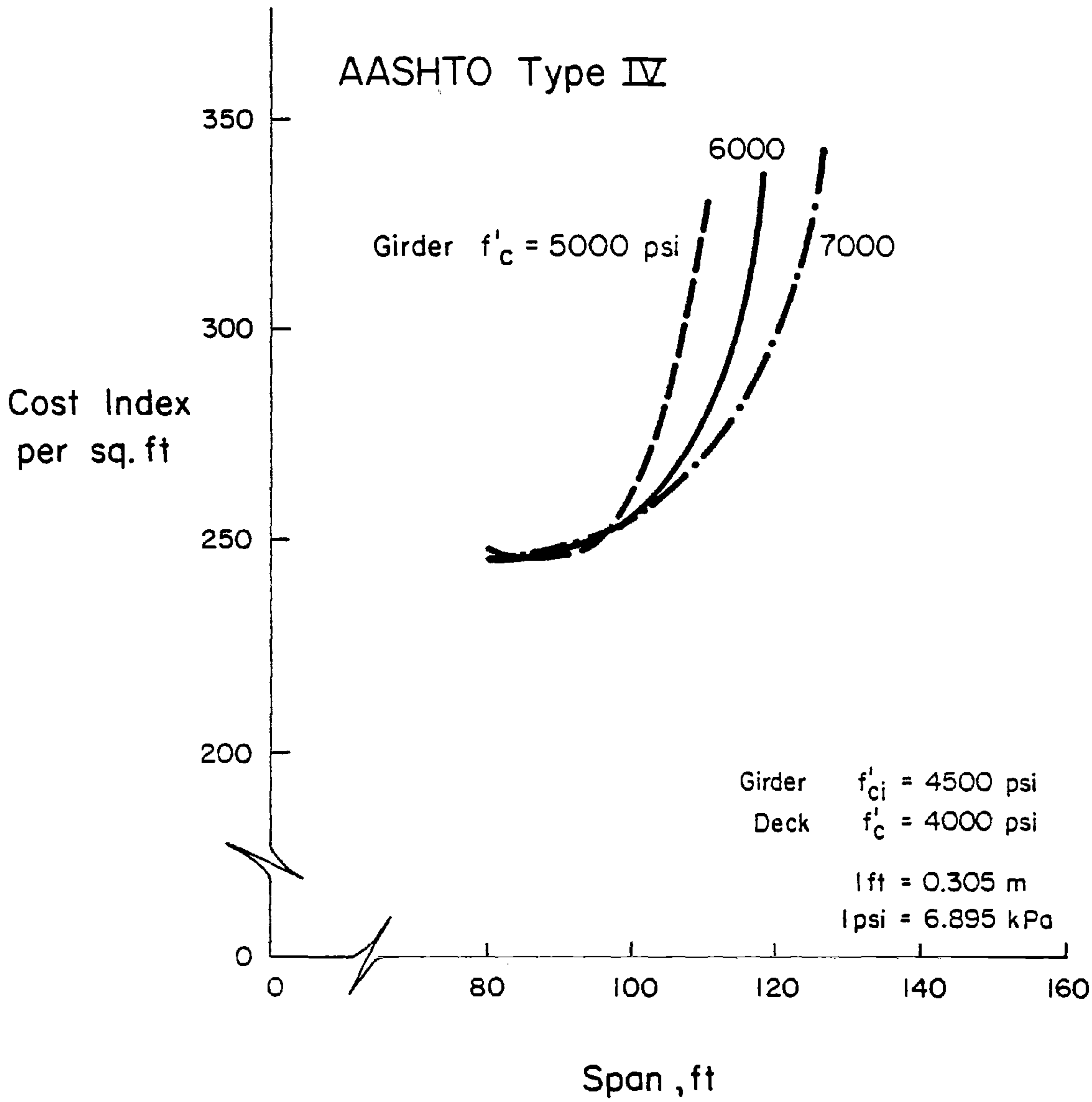


Figure 26 Variation of Optimum Cost Curves with Concrete Compressive Strength for AASHTO Type IV Girder

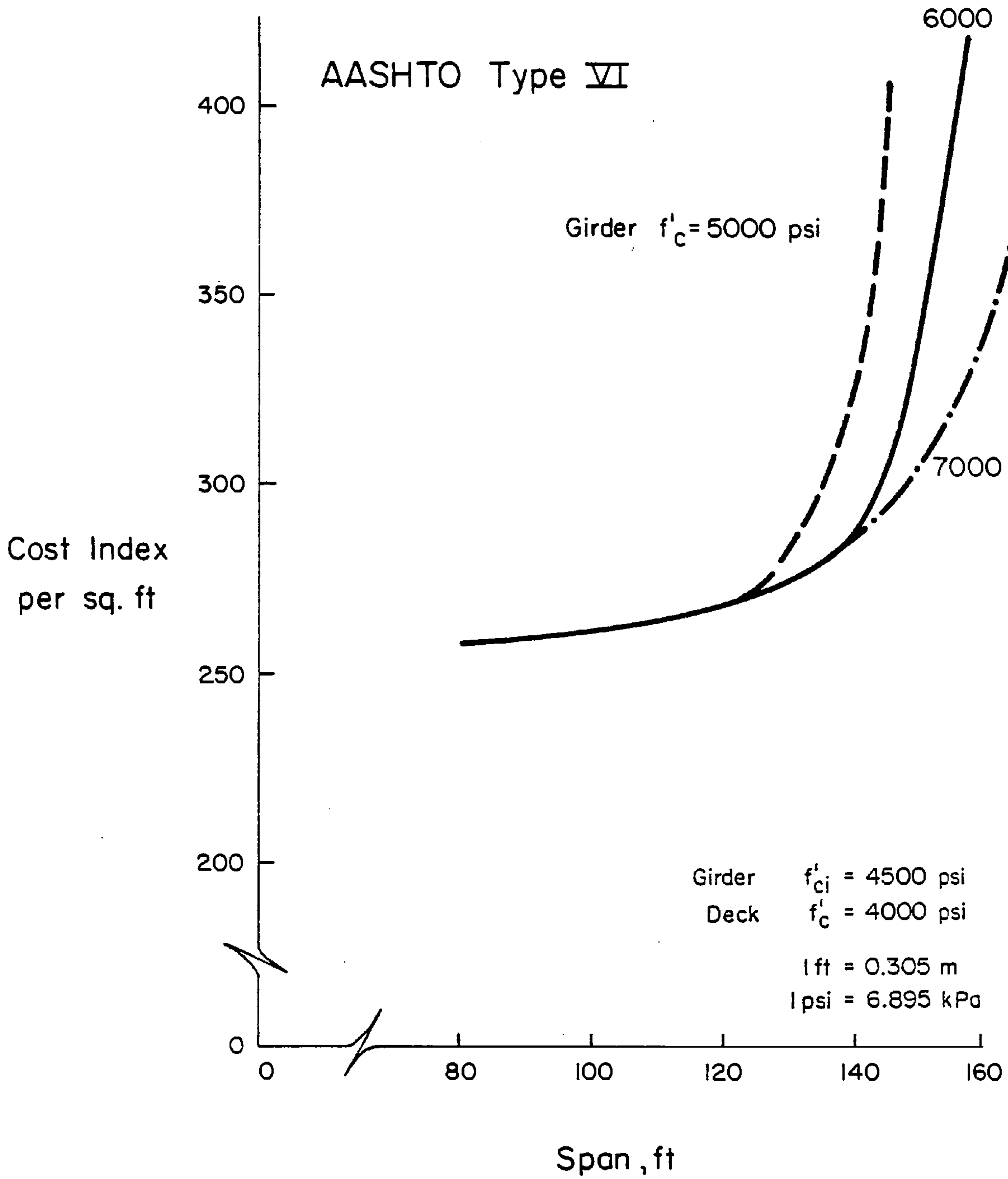


Figure 27 Variation of Optimum Cost Curves with Concrete Compressive Strength for AASHTO Type VI Girder

strands was assumed to be 1-3/4 in (44.5 mm). For analysis of bundled strands, strands were assumed placed next to each other in rows. The first row was assumed positioned 2 in (50.8 mm) from the bottom of the girder, and subsequent rows 1/2 in (12.7 mm) on center.

The effect of bundling strands on cost index is illustrated in Figures 28 and 29. Optimum cost curves for Colorado G54 girder with strands spaced 2 in (50.8 mm) on center, and with bundled strands are shown in Figure 28. Similar curves for AASHTO Type VI girder are shown in Figure 29.

Analysis of Colorado's G54 section assuming a girder spacing of 4.5 ft (1.37 m) and strands spaced 2 in (50.8 mm) on center indicates that maximum span is 122 ft (37.2 m) and corresponding number of strands is 37. When strands are bundled, maximum span is 125 ft (38.1 m) and corresponding number of strands is 35. Therefore, the effect of strand bundling is negligible.

Of all sections analyzed, AASHTO Type VI girder required the largest number of strands. When strands are spaced 2 in (50.8 mm) on center, maximum span is 158 ft (48.2 m). Corresponding number of strands is 67. When strands are bundled, maximum span is 160 ft (48.8 m) and number of strands is 55. The overall cost savings resulting from reduction in number of strands are negligible. Increase in span capability due to bundling of strands is also negligible.

Governing Design Criteria

Output from Program BRIDGE included concrete stress in top and bottom fibers at midspan at transfer and service load and flexural strength of member. A study of these concrete stresses and required flexural strength revealed that in all cases considered, design was governed by bottom concrete stress at midspan under service load. In Program BRIDGE, this concrete stress was limited to a tension of $6\sqrt{f'_c}$ psi ($0.5\sqrt{f'_c}$ MPa).

The above observation was also made by Aswad.[22] He found that for spans up to about 72 ft (22 m), strength design governs. For spans in excess of 72 ft (22 m), bottom concrete

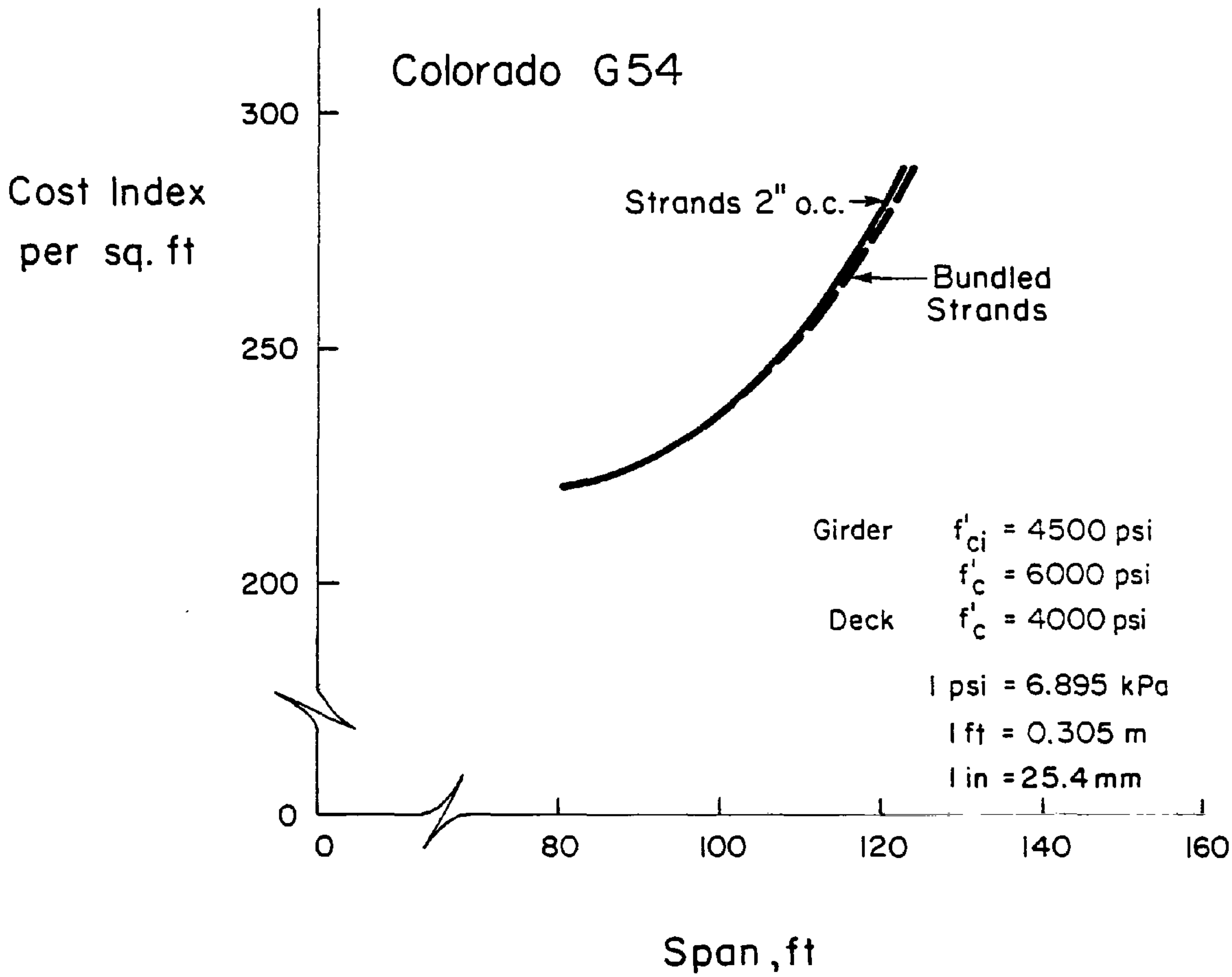


Figure 28 Effect of Strands Bundling on Optimum Cost Curves for Colorado G54 Girders

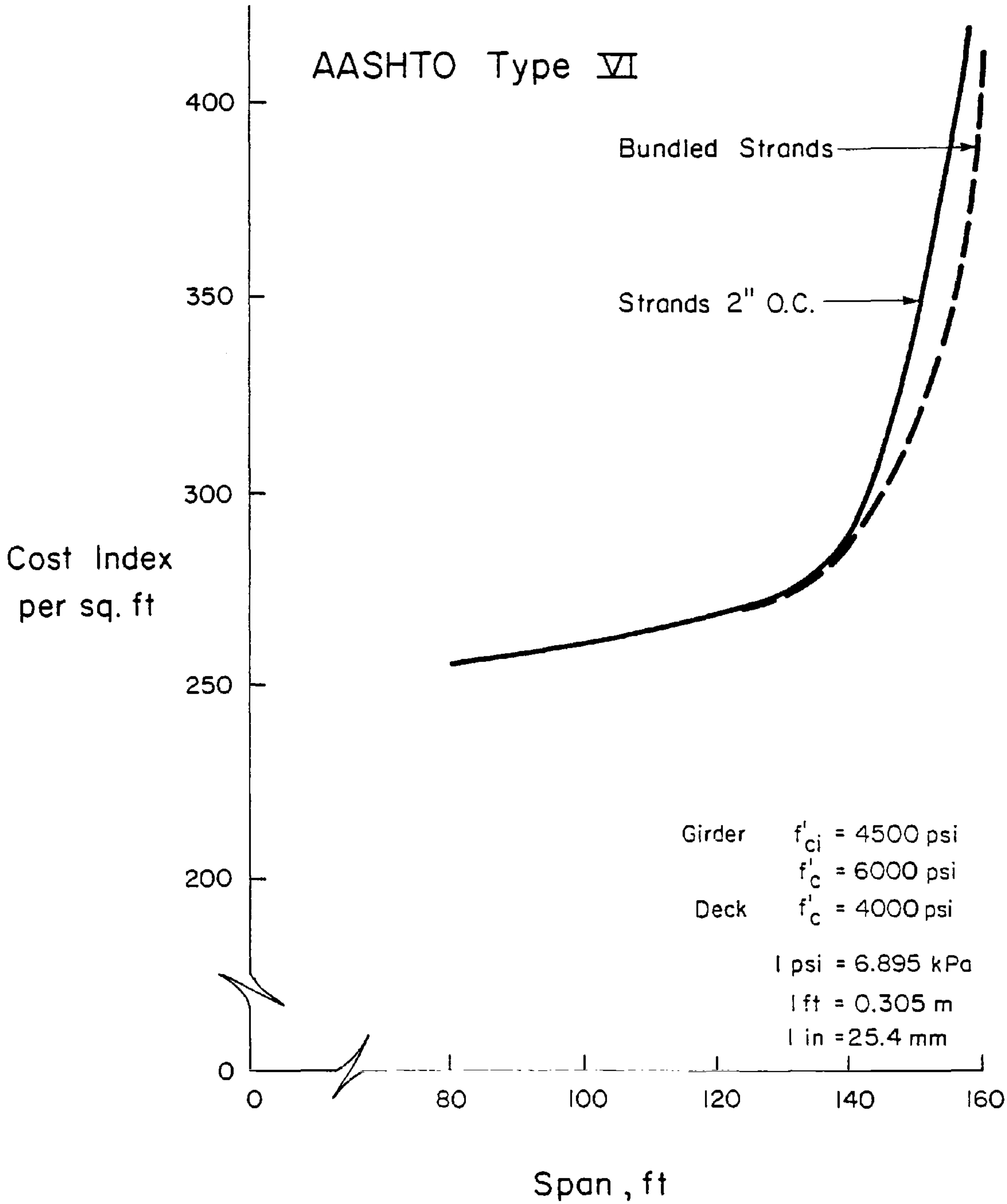


Figure 29 Effect of Strands Bundling on Optimum Cost Curves for AASHTO Type VI Girder

stress at service load governs. Based on the above observation, preliminary design of bridges can be accelerated by satisfying midspan bottom concrete stresses under service loads. Other stress and strength requirements can then be checked. Bottom concrete stress at midspan will also control when no tensile concrete stress is allowed under service load.

As the bottom concrete stress at midspan due to service load controls the design of spans in excess of 80 ft (24.4 m), the aim of any optimization should be to maximize the composite modulus of section for bottom fibers. Efficiency factor or efficiency ratio as discussed under "STRUCTURAL EFFICIENCY" should be a function of the composite modulus of section for bottom fibers. Therefore, girder spacing should be considered when determining efficiency of a girder cross-section.

Shear Design Considerations

Program BRIDGE determines the number of strands that satisfy flexural service load stresses and strength requirements. No attempt was made to incorporate shear design in the program because this was outside the scope of this project. Shear design is a tedious task for the prestressed concrete bridge designer. The problem is further complicated when adjacent spans built with precast prestressed members are made continuous for live load only.

Simplified calculations were made to check whether end blocks might be required for the sections recommended in this report. Nine selected cases judged to produce the highest nominal shear stress were analyzed. Girders were assumed simply supported. End shear was assumed equal to values of end reactions given in Appendix A of the AASHTO Specifications.[8]

Table 2 summarizes the nominal shear stress calculated for the nine cases analyzed. The level of nominal shear stress is well within the minimum allowable value of $9.7\sqrt{f'_c}$ psi ($0.81\sqrt{f'_c}$ MPa) given in the Interim 1980 AASHTO Specifications.[12] Based on shear design considerations, end blocks are not needed in any of these sections, including the Washington Series 14 and Bulb BT72 with 5 in (127 mm) webs. However, detailed shear design

TABLE 2 - NOMINAL SHEAR STRESS

Case No.	Girder	Span (ft)	Girder Spacing (ft)	Nominal Shear v_u, psi	$v_u/\sqrt{f'_c}$*
1	AASHTO Type IV	100	8.0	520	6.7
2	AASHTO Type VI	110	8.0	447	5.8
3	AASHTO Type VI	120	10.0	561	7.2
4	AASHTO Type VI	140	8.0	533	6.9
5	Bulb-T BT72	110	8.0	625	8.0
6	Mod BT72/6	110	8.0	533	6.9
7	Wash. Ser. 14	110	8.0	622	8.0
8	Wash. Ser. 14	140	4.5	475	6.1
9	Mod. Ser. 14/6	110	8.0	531	6.9

*Value of concrete compressive strength, f'_c, assumed 6000 psi.

1 ft = 0.305 m; 1 psi = 6.895 kPa

is necessary, for each bridge, and particularly when adjacent spans are made continuous for live load.

SI CONVERSION

Recently, new SI (metric) sections were adopted in Canada under an arrangement agreed to by the prestressed concrete producers. For an unspecified period of time, bridges in Canada will be designed using the new sections, but alternate designs will be provided based on existing non-metric sections. Since the new sections are more efficient than the existing ones, it was felt that the changeover would be accelerated by the competitive need to use the new sections. Dimensions and sectional properties of the old and new Canadian sections are given in Appendix C.

CONCLUSIONS

Based on the survey of Phase I and cost-analyses of Phase II, the following conclusions are made.

1. In all states surveyed except California, the most economical bridges for spans of approximately 70 to 130 ft (21.3 to 39.6 m) are constructed with pretensioned bridge girders. In California, cast-in-place post-tensioned box girder bridges are most economical.
2. When compared with other sections, AASHTO standard bridge girders are not the most structurally efficient or cost-effective for spans of 80 to 140 ft (24.4 to 42.7 m).
3. Because of transportation restrictions, maximum spans made of single units are limited to about 140 ft (42.7 m). Longer spans are possible by splicing girders.
4. Intermediate diaphragms are not needed. End diaphragms are sufficient.
5. For girders with 5-in (127 mm) thick webs, the most cost-effective sections are Bulb-T's. For spans from 80 to 120 ft (24.4 to 36.6 m), Bulb-T's have 20% less in-place cost of girder and deck compared to AASHTO

girders. For spans of 120 to 135 ft (36.6 to 41.2 m), the cost reduction for Bulb-T's varies from 20 to 5%. Next most cost-effective sections with 5-in (127 mm) thick webs are the Washington series.

6. In most regions of the United States, it may not be easy to consolidate the concrete in girders with 5-in (127 mm) thick webs. Moreover, in these girders, strands must be bundled at midspan and end blocks are needed to conform with minimum concrete cover requirements.
7. By using girders with 6-in (152 mm) thick webs, it will be possible to consolidate the concrete in these girders in all regions of the United States.
8. Use of 6-in (152 mm) thick webs instead of 5 in (127 mm) in Bulb-T's, Washington series, and Colorado G68 girders increases overall in-place cost of girder and deck by 3 to 5%.
9. For girders with 6-in (152 mm) thick webs, most cost-effective sections are Modified Bulb-T's. For spans of 80 to 120 ft (24.4 to 36.6 m), Modified Bulb-T's have 17% less in-place cost of girder and deck compared to AASHTO girders. For spans of 120 to 140 ft (36.6 to 42.7 m), the cost reduction varies from 17 to 2%.
10. Next to Modified Bulb-T's, modified Washington Series girders with 6-in (152 mm) thick webs are the most cost-effective sections. For spans from 80 to 120 ft (24.4 to 36.6 m), overall reduction of the in-place cost of girder and deck is 14%. For spans of 120 to 140 ft (36.6 to 42.7 m), cost reduction ranges from 14 to 2%, compared to the AASHTO girders.
11. Reduction of top and bottom flange widths and web thicknesses of AASHTO Types IV, V and VI girders by 2 in (50.8 mm) reduces overall in-place cost of girders and deck by about 6%. Span capability of the

modified sections is not affected by this change in width.

12. The overall in-place cost of girders and deck is decreased substantially by placing girders at the largest practical girder spacing.
13. Increase of girder's concrete compressive strength from 5000 to 7000 psi (34.5 to 48.3 MPa) increases the span capability of AASHTO girders by about 15%.
14. Bundling of strands at midspan in order to increase eccentricity of prestress does not lead to any significant overall cost reduction for girders considered.

RECOMMENDATIONS

Based on the cost-analysis results discussed in the report and the above conclusions, the following are recommended:

1. Modified Bulb-T girders with 6-in (152 mm) thick webs are recommended for use as national standard precast prestressed concrete bridge girders in the United States for spans from 80 to 140 ft (24.4 to 42.7 m).
2. If metrication is adopted, modification of the above sections to SI (metric) units should be considered as part of any standardization.
3. Girder spacing should be as large as possible.
4. A synthesis report is needed on techniques available for splicing girders for spans in excess of 135 ft (41.2 m).
5. Although lightweight concrete was not considered in this investigation, it should be given more consideration for bridges built with precast prestressed concrete girders and cast-in-place concrete deck. Lightweight concrete has a cost premium above that of normal weight concrete. However, overall weight reduction can lead to cost savings.
6. Lateral stability of long span girders should be investigated to determine critical lengths beyond which girders should be braced laterally during transportation and erection.

IMPLEMENTATION

Construction of the Interstate highway system has been completed in a few states. In most states, it is close to completion. Therefore, the rate of bridge construction on the interstate highway is much slower than in the period between late 1950's and early 1970's. However, according to statistics prepared by the Bridge Division, Federal Highway Administration, U.S. Department of Transportation, considerable new bridge construction and major reconstruction is ongoing.

The cost of new prestressed concrete bridge construction and major reconstruction with participation of federal funds authorized during calendar year 1980 totaled $695 million. Based on bridge inventory and inspection records, it is anticipated that "in the next 20 to 30 years, we will have over $30 billion worth of bridge construction based on the value of the dollar today."(26)

As mentioned earlier, selection of bridge type is based on economy. Safety standards for interstate and other high speed highways require greater clearances. Therefore, there is need for construction of bridges with spans of 110 to 130 ft (33.5 to 39.6 m). In all states surveyed except California, most economical bridges for spans of approximately 70 to 130 ft (21.3 to 39.6 m) were constructed with pretensioned bridge girders.

Cost analyses discussed earlier indicate that modified Bulb-T's can yield savings of 17% on the overall cost of girder and deck compared to AASHTO girders. This is in addition to the fact that the Modified Bulb-T's are about 35% lighter than AASHTO girders for comparable spans. A 140 ft (42.7 m) AASHTO Type VI girder is extremely heavy and therefore very difficult to transport on highways. Lighter sections with 140 ft (42.7 m) have been transported on highways.

Steel forms constitute a capital investment. However, their life span is limited to about 10 years. Cost savings resulting from use of optimized girders should be adequate to cover the cost of new forms over a period of a few years in areas where AASHTO girders are used. Where new forms are needed, new plants built, or improved sections sought, optimized sections should be considered.

Implementation of new sections should be gradual over a period of time. It will require effort on the part of both Departments of Transportation and producers. Preparation of design aids for the new sections will encourage and facilitate implementation of the new sections.

Highway agencies should be informed of economic benefits that can be achieved with optimized sections. Departments of Transportation will have to design with old and new sections over a transition period. The Canadian experience in switching to new metric sections sets an example of implementation of new sections under an arrangement agreeable to producers and highway agencies.

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APPENDIX A - SURVEY PARTICIPANTS

Highway agencies and producers selected for the survey are listed below. In all cases, the survey was initiated through telephone conversations. In addition, site surveys were made where indicated by "(S)".

Highway Agencies

1. State of California, Department of Transportation. (S)
2. State of Colorado, State Department of Highways.
3. State of Illinois, Department of Transportation. (S)
4. State of Louisiana, Department of Transportation and Development.
5. Oregon State Highway Division. (S)
6. Commonwealth of Pennsylvania, Department of Transportation. (S)
7. State of Tennessee, Department of Transportation.
8. State of Texas, State Department of Highways and Public Transportation. (S)
9. Commonwealth of Virginia, Department of Highways and Transportation. (S)
10. Washington State, Department of Transportation. (S)
11. State of Wisconsin, Department of Transportation. (S)

Producers

12. Concrete Technology Associates, Tacoma, Washington. (S)
13. Prestressed Concrete Operations, St. Regis Paper Company, Iowa Falls, Iowa.
14. Stanley Structures, Denver, Colorado. (S)

APPENDIX B - SURVEY OUTLINE

In order to conduct efficient and systematic site surveys, an outline of required information was prepared. It covers all items discussed under Phase I of the Research Approach. Survey outline is reproduced below. Results of the survey are presented in the body of the report.

Design and Construction Details

1. Solid-form sections used in the State
   - Which AASHTO-PCI sections
   - Other sections and corresponding span range
   - When developed
   - When first pretensioned bridge built
   - When first post-tensioned bridge built
   - Any effort presently to modify/improve sections
   - Would State consider improved sections, if developed
   - Obtain drawings of sections
2. Longest spans built using single pretensioned girders
3. End blocks required
   - When used
4. How bursting (end splitting) reinforcement designed
5. End diaphragms used
   - Policy for intermediate diaphragms
6. Is tension in concrete under service load permitted
   - How much
   - When adopted
7. Concrete Properties
   - Strength at release f'_{ci}
   - 28 day strength f'_c
   - Density of normal weight concrete
   - Lightweight concrete for bridge girders
   - Lightweight concrete for decks
   - f'_c lightweight
   - Density of lightweight concrete

8. Strand Properties
   - Size
   - Grade
   - Surface condition
   - Stress-relieved, low-relaxation
   - Where strands manufactured
9. Maximum girder spacing
10. Maximum allowable camber and differential camber
11. Types of decks
   - Cast-in-place
   - Precast deck panels
   - Permanent forms, steel or concrete
   - Maximum deck thickness
   - Bent-up bars in deck
12. Design Loads
   - HS20-44
   - Other loads
13. Design aids for bridge designers
   - Tables or charts
14. Does adequacy of manufacturers, erectors, contractors influence decision regarding type of bridge.
15. Does State permit (or encourage) contractor/supplier alternate designs to reduce overall cost? (emphasis on fabrication details and erection procedure)

Fabrication Details

1. Minimum spacing between strands (center to center)
2. Minimum concrete cover
3. Limitation on web thickness
4. Stability during handling
5. Restrictions on draping locations
6. Precasting plants
   - How many within State
   - Use of out of state
   - Licensed by State

7. Turn-around time for prestressing beds
8. Crane capacity in plant
9. Any new developments in prestressing hardware systems

Transportation Requirements and Restrictions

1. Maximum girder length without permit
   - Maximum transported
2. Maximum girder weight without permit
   - Maximum weight of girders transported
3. Availability of moving equipment
4. Instability during transportation

Availability and Costs

1. Cost in-place per lineal foot of girder (each section)
2. Cost in place per square foot of deck
3. Cost per square foot of superstructure
4. Cost variation due to concrete strength f'_c.
5. Prestressing beds, steel forms
   - Initial cost
   - Life span
   - End blocks - how incorporated
     - effect on cost
6. Labor (plant, site)
   - Availability, skilled
   - Wages
7. What could be done to cut down on cost
8. How manufacturers estimate cost
9. Haul costs \$/kip/mile - Freight tariff steps
10. State unit prices for cost estimate
11. Cranes at construction site
    - Availability
    - Capacity
    - Costs

Structural Durability

1. Design life span
2. Frequency of inspection
3. Problems encountered (and frequency)
4. Cracks in prestressed concrete girders

Aesthetics and Safety

1. Aesthetics requirements
2. Safety requirements

Opinions and Policies

1. Opinions and policies of local authorities, including industry regarding selection and use of major prestressed concrete girders

New Concepts

1. New concepts related to design, manufacture, erection or construction of pretensioned bridge girders
2. Blanketing or debonding of strands
   - Used
   - Why not
   - Draping satisfactory
3. Has splicing of pretensioned I, or T-segments been used
   - If yes, longest spans achieved
   - If not, has it been considered
4. Combination pretensioning and post-tensioning of single (non-spliced) units
5. Precast prestressed deck panels as permanent forms
6. Possible use of lightweight concrete

Other

1. Trade-offs between I, box and steel sections
2. Projections on future bridge construction (pretensioned girders)
   - Spans
   - Volume
3. Criteria for optimization
4. What can be done to make pretensioned bridge construction more attractive and more economical

APPENDIX C - GIRDER PROPERTIES

Dimensions and properties of standard bridge girder sections used by the survey participants are presented in this Appendix. Table 3 lists the sectional dimensions. Notations appearing in this table are identified in the sketches of Figure 30. Dimensions of the new Canadian standard metric prestressed I-Girders are listed in Table 4. These sections were approved by the Canadian Prestressed Concrete Institute in October 1979.

Properties of sections appearing in Tables 3 and 4 are listed in Tables 5 and 6, respectively. Standard sectional property notations have been used. They are defined as follows:

y_t = Distance from centroid of girder to top concrete fiber

y_b = Distance from centroid of girder to bottom concrete fiber

S_t = Modulus of section for top concrete fiber

S_t = Modulus of section for bottom concrete fiber

Efficiency factors α and ρ have been suggested by Aswad[22] and Guyon[21], respectively. They are defined as follows:

$$\alpha = \frac{3.46\ S_b}{A\ .\ h}$$

$$\rho = \frac{r^2}{y_t \cdot y_b} = \frac{S_b}{A\ .\ y_t}$$

Approximate span range capabilities of each girder section are also shown in Tables 5 and 6. The span range varies with the concrete compressive strength at transfer and at 28 days.

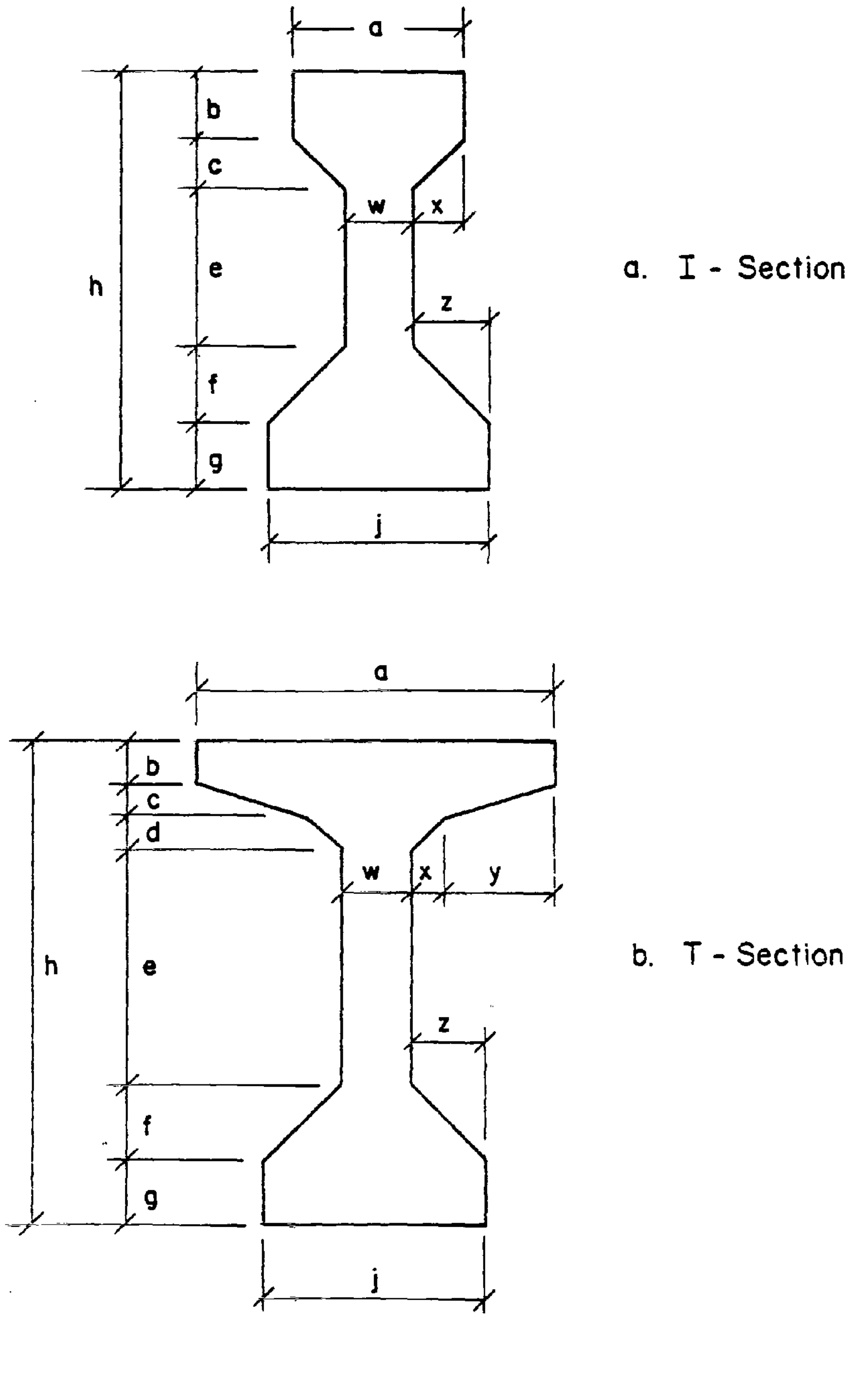

Figure 30 Nomenclature for Cross-Sectional Dimensions Shown in Tables 3 and 4

TABLE 3 - CROSS-SECTIONAL DIMENSIONS

Agency	Girder Type	Dimension*, in												
		a	b	c	d	e	f	g	h	j	w	x	y	z
AASHTO-PCI	Type I	12	4	3	-	11	5	5	28	16	6	3	-	5
	Type II	12	6	3	-	15	6	6	36	18	6	3	-	6
	Type III	16	7	4.5	-	19	7.5	7	45	22	7	4.5	-	7.5
	Type IV	20	8	6	-	23	9	8	54	26	8	6	-	9
	Type V	42	5	3	4	33	10	8	63	28	8	4	13	10
	Type VI	42	5	3	4	42	10	8	72	28	8	4	13	10
California	3'-0"	19	3	6	-	15	6	6	36	19	7	6	-	6
	3'-6"	19	3	6	-	21	6	6	42	19	7	6	-	6
	4'-0"	19	3	6	-	27	6	6	48	19	7	6	-	6
	4'-6"	19	3	6	-	33	6	6	54	19	7	6	-	6
	5'-0"	19	3	6	-	39	6	6	60	19	7	6	-	6
	5'-6"	19	3	6	-	45	6	6	66	19	7	6	-	6
Colorado	G54	28	5.5	1.5	2	33.5	5	6.5	54	24	6	2	9	8.5
	G68	28	5.5	1.5	2	49	3.5	6.5	68	24	5	2	9.5	8.5
Illinois	36"	12	4	3	-	17	6	6	36	18	6	3	-	6
	42"	16	4	2.5	-	21.5	8	6	42	22	6	5	-	8
	48"	18	4	2.63	-	27.12	7.25	7	48	22	7.5	5.25	-	7.25
	54"	20	6	3	-	30.75	7.25	7	54	22	6	7	-	8

*Notations are identified in Figure 30

1 in = 25.4 mm

TABLE 3 - CROSS-SECTIONAL DIMENSIONS

(Cont.)

Agency	Girder Type	Dimension*, in												
		a	b	c	d	e	f	g	h	j	w	x	y	z
Iowa	A30-A46	13	4	1	-	16	6	5	32	17	6	3.5	-	5.5
	A50-A55	16	4	1	-	16	6	5	32	20	9	3.5	-	5.5
	B34-B59	13	5	1	-	20	6	7	39	17	6	3.5	-	5.5
	B63-B67	16	5	1	-	20	6	7	39	20	9	3.5	-	5.5
	C30-C67	13	5	1	-	25	6	8	45	17	6	3.5	-	5.5
	C71-C80	16	5	1	-	25	6	8	45	20	9	3.5	-	5.5
	D35-D95	20	6	1	-	31.5	7.5	8	54	22	7	6.5	-	7.5
Louisiana	AASHTO-PCI Types II, III, and IV													
Oregon	AASHTO-PCI Types II, III, and IV													
	Type V	20	8	6	-	32	9	8	63	26	8	6	-	9
	BT 65"	48	2.5	2.5	3	48	4	5	65	24	7.5	3	17.25	8.25
	BT 72"	48	2	3	2	56	3	6	72	26	6	2	19	10
	I 84"	24	6	6	-	60	6	6	84	24	7.5	8.25	-	8.25
Pennsylvania	20/30	14	3	3	-	12	8	4	30	20	8	3	-	6
	20/33	14	4	3	-	12	8	6	33	20	8	3	-	6
	20/36	14	5	3	-	12	8	8	36	20	8	3	-	6
	20/39	14	8	3	-	12	8	8	39	20	8	3	-	6
	24/33	18	4	3	-	12	8	6	33	24	12	3	-	6
	24/36	18	5	3	-	12	8	8	36	24	12	3	-	6

*Notations are identified in Figure 30

1 in = 25.4 mm

TABLE 3 - CROSS-SECTIONAL DIMENSIONS

(Cont.)

Agency	Girder Type	Dimension*, in												
		a	b	c	d	e	f	g	h	j	w	x	y	z
Pennsylvania	24/42	18	4	4	-	17	10	7	42	24	8	5	-	8
	24/45	18	7	4	-	17	10	7	45	24	8	5	-	8
	24/48	18	8	4	-	17	10	9	48	24	8	5	-	8
	24/51	18	11	4	-	17	10	9	51	24	8	5	-	8
	24/54	18	14	4	-	17	10	9	54	24	8	5	-	8
	26/33	20	4	3	-	12	8	6	33	26	14	3	-	6
	26/36	20	5	3	-	12	8	8	36	26	14	3	-	6
	26/60	26	6	6	-	29	10	9	60	26	10	8	-	8
	26/63	26	9	6	-	29	10	9	63	26	10	8	-	8
	28/63	AASHTO-PCI Type V												
Tennessee		AASHTO-PCI Types I, II, III, and IV												
Texas	A	12	4	3	-	11	5	5	28	16	6	3	-	5
	B	12	5.5	2.75	-	14	5.75	6	34	18	6.5	2.75	-	5.75
	C	14	6	3.5	-	16	7.5	7	40	22	7	3.5	-	7.50
	48	14	3.5	4	-	29.5	4	7	48	14	6	4	-	4
	54	16	4	5	-	32	5	8	54	16	6	5	-	5
	60	18	4.5	5.5	-	35.5	5.5	9	60	18	7	5.5	-	5.5
	66	20	5	6.5	-	38	6.5	10	66	20	7	6.5	-	6.5
	72	22	5.5	7.5	-	40.5	7.5	11	72	22	7	7.5	-	7.5
	IV	AASHTO-PCI Type IV												

*Notations are identified in Figure 30

1 in = 25.4 mm

TABLE 3 - CROSS-SECTIONAL DIMENSIONS

(Cont.)

Agency	Girder Type	Dimension*, in												
		a	b	c	d	e	f	g	h	j	w	x	y	z
Virginia	AASHTO-PCI Types III, IV, V, and VI													
	T 34"	Max. 48"	4	3	-	27	-	-	34	12	12	3	15	-
Washington	Series 40	14	3.5	1.5	-	21	2	4	32	16	5	4.5	-	5.5
	Series 60	14	3.5	1.5	-	30	2	5	42	19	5	4.5	-	7
	Series 80	19	5	2	-	34	3	6	50	24	5	7	-	9.5
	Series 100	24	5	2	-	42	3	6	58	24	5	9.5	-	9.5
	Series 120	28	4	1.5	2	57	3	6	73.5	24	5	2	9.5	9.5
	Series 14	42	2.87	2.63	2	57	3	6	73.5	24	5	2	16.5	9.5
Wisconsin	AASHTO-PCI Types II, III, and IV													
	70"	30	6	1.5	2	49.5	3.5	7.5	70	26	6	2	10	10
Ontario	AASHTO-PCI Types II, and III													
(Canada)	CPCI IV	22	6	3	-	31.5	6.5	7	54	26	7	7.5	-	9.5
	CPCI IV+4	22	10	3	-	31.5	6.5	7	58	26	7	7.5		9.5
	ONT 90	36	5	3	3	63	8	8	90	26	8	3	11	9
	MOD. ONT 90	37	5	2	3	69	4	7	90	27	7	3	12	10

*Notations are identified in Figure 30

1 in = 25.4 mm

TABLE 3 - CROSS-SECTIONAL DIMENSIONS

(Cont.)

Cross Section Identification	Dimension*, in												
	a	b	c	d	e	f	g	h	j	w	x	y	z
Anderson BT 48	48	1	3.0	2	33	3	6	48	24	5	2	19.5	9.5
BT 60	48	1	3.0	2	45	3	6	60	24	5	2	19.5	-
BT 72	48	1	3.0	2	57	3	6	72	24	5	2	19.5	9.5
Modified AASHTO													
Type IV	18	8	6	-	23	9	8	54	24	6	6	-	9.0
Type V	40	5	3	4	33	10	8	63	26	6	4	13	10.0
Type VI	40	5	3	4	42	10	8	72	26	6	4	13	10.0
Modified Colorado													
G68/6	28	5.5	1.5	2	49	3.5	6.5	68	24	6	2	9.0	8.0
Modified Washington													
Series 80/6	19	5	2	-	34	3	6	50	24	6	6.5	-	9.0
Series 100/6	24	5	2	-	42	3	6	58	24	6	9.0	-	9.0
Series 120/6	28	4	1.5	2	57	3	6	73.5	24	6	2	9	9.0
Series 14/6	42	2.87	2.63	2	57	3	6	73.5	24	6	2	16.0	9.0
Modified Bulb-T													
BT 48/6	48	2	2	2	33	3	6	48	24	6	2	19.0	9.0
BT 60/6	48	2	2	2	45	3	6	60	24	6	2	19.0	9.0
BT 72/6	48	2	2	2	57	3	6	72	24	6	2	19.0	9.0

*Notations are identified in Figure 30

1 in = 25.4 mm

TABLE 4 - DIMENSIONS OF NEW CPCI METRIC SECTIONS

Agency	Girder Type	Dimension*, mm												
		a	b	c	d	e	f	g	h	j	w	x	y	z
CPCI Canadian Metric Sections	900	300	150	30	-	480	90	150	900	450	150	75	-	150
	1200	400	150	50	-	700	120	180	1200	550	150	125	-	200
	1200	400	150	50	-	700	120	180	1200	550	150	125	-	200
	1400	550	150	80	-	840	150	180	1400	650	150	200	-	250
	1900	900	125	50	75	1300	150	200	1900	650	150	75	300	250
	1900A	930	125	50	75	1300	150	200	1900	680	180	75	300	250
	2300	900	125	50	75	1700	150	200	2300	650	150	75	300	250
	2300A	930	125	50	75	1700	150	200	2300	680	180	75	300	250

*Notations are identified in Figure 30

1 mm = 0.0394 in

TABLE 5 - CROSS-SECTIONAL PROPERTIES

Agency	Girder Type	h (in)	y_t (in)	y_b (in)	Area (in^2)	S_t (in^3)	S_b (in^4)	Inertia (in^4)	α	ρ	Span Range* (ft)
AASHTO-PCI	Type I	28	15.41	12.59	276	1,476	1,807	22,750	0.81	0.42	30 - 45
	II	36	20.17	15.83	369	2,528	3,220	50,980	0.84	0.43	40 - 60
	III	45	24.73	20.27	560	5,070	6,186	125,390	0.85	0.45	55 - 80
	IV	54	29.27	24.73	789	8,908	10,543	260,730	0.86	0.46	70 - 120
	V	63	31.04	31.96	1,013	16,791	16,307	521,180	0.88	0.52	90 - 140
	VI	72	35.62	36.38	1,085	20,587	20,157	733,320	0.89	0.52	110 - 150
California	3'-0"	36	18.9	17.1	432	3,350	3,700	63,300	0.82	0.45	50 - 55
	3'-6"	42	22.0	20.0	474	4,320	4,750	95,000	0.83	0.46	55 - 65
	4'-0"	48	25.2	22.8	516	5,450	6,020	137,300	0.84	0.46	65 - 75
	4'-6"	54	28.3	25.7	558	6,640	7,310	187,800	0.84	0.46	75 - 80
	5'-0"	60	31.4	28.6	600	7,920	8,690	248,600	0.84	0.46	80 - 90
	5'-6"	66	34.4	31.6	642	9,240	10,070	318,000	0.82	0.46	90 - 100
Colorado	G54	54	27.33	26.67	631	8,877	9,095	242,585	0.92	0.53	80 - 120
	G68	68	34.09	33.91	648	12,120	12,185	413,184	0.96	0.55	105 - 140
Illinois	36"	36	20.63	15.37	357	2,358	3,165	48,648	0.85	0.43	30 - 70
	42"	42	24.35	17.65	465	3,736	5,153	90,956	0.91	0.46	40 - 85
	48"	48	26.91	21.09	570	5,355	6,834	144,117	0.86	0.45	50 - 95
	54"	54	29.03	24.97	599	7,362	8,559	213,715	0.92	0.49	60 - 110

*Varies with concrete compressive strength at transfer and at 28 days

1 in = 25.4 mm; 1 ft = 0.305 m

TABLE 5 - CROSS-SECTIONAL PROPERTIES

(Cont.)

Agency	Girder Type	h (in)	y_t (in)	y_b (in)	Area (in^2)	S_t (in^3)	S_b (in^4)	Inertia (in^4)	α	ρ	Span Range* (ft)
Iowa	A30-A46	32	17.95	14.05	312	1,899	2,426	34,082	0.84	0.43	30 - 46
	A50-A55	32	17.49	14.51	408	2,433	2,933	42,552	0.78	0.41	50 - 55
	B34-B59	39	21.94	17.06	383	2,826	3,634	62,000	0.84	0.43	34 - 59
	B63-B67	39	21.37	17.63	500	3,620	4,388	77,364	0.78	0.41	63 - 67
	C30-C67	45	25.48	19.52	530	3,637	4,747	92,659	0.85	0.43	30 - 67
	C71-C80	45	24.77	20.23	565	4,697	5,752	116,354	0.78	0.41	71 - 80
	D35-D95	54	29.63	24.37	639	7,255	8,821	214,974	0.88	0.47	35 - 95
Louisiana			AASHTO-PCI Types II, III, and IV								
Oregon			AASHTO-PCI Types II, III, and IV								
	Type V	63	34.06	28.94	861	11,595	13,648	394,941	0.87	0.47	90 - 120
	BT65"	65	30.23	34.77	771	13,864	12,052	419,064	0.83	0.52	100 - 140
	BT72"	72	35.32	36.68	739	14,835	14,285	523,962	0.93	0.55	100 - 145
	I84"	84	42.00	42.00	927	18,734	18,734	786,834	0.83	0.48	Up to 190
Pennsylvania	20/30	30	16.88	13.12	363	1,942	2,499	32,786	0.79	0.41	40 - 55
	20/33	33	18.65	14.35	417	2,400	3,119	44,757	0.78	0.40	44 - 60
	20/36	36	20.38	15.62	471	2,899	3,782	59,077	0.77	0.39	47 - 66
	20/39	39	21.59	17.41	513	3,593	4,456	77,576	0.77	0.40	50 - 70
	24/33	33	18.13	14.87	549	3,155	3,847	57,200	0.73	0.39	46 - 63

*Varies with concrete compressive strength at transfer and at 28 days

1 in = 25.4 mm; 1 ft = 0.305 m

TABLE 5 - CROSS-SECTIONAL PROPERTIES

(Cont.)

Agency	Girder Type	h (in)	y_t (in)	y_b (in)	Area (in^2)	S_t (in^3)	S_b (in^4)	Inertia (in^4)	α	ρ	Span Range* (ft)
Pennsylvania	24/36	36	19.83	16.17	615	3,795	4,654	75,256	0.73	0.38	50 - 68
	24/42	42	23.96	18.04	588	4,506	5,985	107,967	0.84	0.42	57 - 87
	24/45	45	24.82	20.18	642	5,643	6,941	140,065	0.83	0.44	60 - 83
	24/48	48	26.61	21.39	708	6,490	8,074	172,712	0.82	0.43	66 - 88
	24/51	51	27.62	23.38	762	7,690	9,085	212,399	0.81	0.43	69 - 92
	24/54	54	28.69	25.31	816	8,895	10,083	255,194	0.79	0.43	72 - 109
	26/33	33	17.96	15.04	615	3,527	4,212	63,346	0.72	0.38	48 - 64
	26/36	36	19.63	16.37	687	4,241	5,085	83,247	0.71	0.38	52 - 70
	26/60	60	31.48	28.52	968	12,436	13,727	391,487	0.82	0.45	78 - 105
	26/63	63	32.03	30.97	1,046	14,676	15,179	470,081	0.80	0.45	80 - 108
	28/63	AASHTO-PCI Type V									
Tennessee		AASHTO-PCI Types I, II, III, and IV									
Texas	A	28	15.39	12.61	275	1,472	1,797	22,658	0.81	0.42	28 - 45
	B	34	19.07	14.93	360	2,264	2,892	43,177	0.82	0.42	40 - 60
	C	40	22.91	17.09	495	3,606	4,833	82,602	0.84	0.43	50 - 85
	48	48	25.13	22.87	403	4,057	4,458	101,950	0.80	0.44	-
	54	54	28.47	25.53	493	5,761	6,425	164,022	0.84	0.46	60 - 100
	60	60	31.59	28.41	628	8,082	8,987	255,319	0.83	0.45	-

*Varies with concrete compressive strength at transfer and at 28 days

1 in = 25.4 mm; 1 ft = 0.305 m

TABLE 5 - CROSS-SECTIONAL PROPERTIES

(Cont.)

Agency	Girder Type	h (in)	y_t (in)	y_b (in)	Area (in^2)	S_t (in^3)	S_b (in^4)	Inertia (in^4)	α	ρ	Span Range* (ft)
Texas	66	66	34.93	31.07	741	10,727	12,059	374,688	0.85	0.47	-
	72	72	38.27	33.73	863	13,903	15,774	532,060	0.88	0.48	100 - 150
	IV	AASHTO-PCI Type IV									
Virginia	AASHTO-PCI Types III, IV, V, and VI										
	T34"	34	Varies with top flange width								30 - 60
Washington	Series 40	32	16.84	15.16	253	1,841	2,045	31,000	0.87	0.48	40 - 55
	Series 60	42	23.37	18.63	332	3,000	3,763	70,100	0.93	0.48	55 - 80
	Series 80	50	27.47	22.53	476	5,639	6,875	154,900	1.00	0.53	65 - 105
	Series 100	58	30.10	27.90	546	8,272	8,925	249,000	0.98	0.54	65 - 125
	Series 120	73.5	37.90	35.60	626	12,032	12,809	456,000	0.96	0.54	85 - 145
	Series 14	73.5	35.33	38.16	674	14,556	13,476	514,312	0.94	0.57	110 - 150
Wisconsin	AASHTO-PCI Types II, III and IV										
	70"	70	35.38	34.62	774	14,430	14,751	510,613	0.94	0.54	105 - 135

*Varies with concrete compressive strength at transfer and at 28 days

1 in = 25.4 mm; 1 ft = 0.305 m

TABLE 5 - CROSS-SECTIONAL PROPERTIES

(Cont.)

Agency	Girder Type	h (in)	y_t (in)	y_b (in)	Area (in^2)	S_t (in^3)	S_b (in^4)	Inertia (in^4)	α	ρ	Span Range* (ft)
Ontario (Canada)	AASHTO-PCI Types II, and III										
	CPCI IV	54	29.36	24.64	685	8,274	9,859	242,936	0.92	0.49	80 - 120
	CPCI IV+4	58	29.79	28.21	773	10,733	11,334	319,737	0.87	0.49	95 - 125
	ONT 90	90	45.11	44.89	1,136	25,619	25,743	1,155,669	0.87	0.50	up to 145
	MOD. ONT 90	90	44.16	45.84	1,005	23,981	23,103	1,059,014	0.88	0.52	140 - 170

*Varies with concrete compressive strength at transfer and at 28 days   1 in = 25.4 mm; 1 ft = 0.305 m

TABLE 5 - CROSS-SECTIONAL PROPERTIES

(Cont.)

Cross Section Identification	h (in)	y_t (in)	y_b (in)	Area (in^2)	S_t (in^3)	S_b (in^4)	Inertia (in^4)	α	ρ	Span Range* (ft)
Anderson BT 48	48	24.50	23.50	500	6,695	6,975	163.972	1.01	0.57	70 - 100
BT 60	60	30.71	29.29	560	9,224	9,669	283,229	1.00	0.56	80 - 125
BT 72	72	36.88	35.12	620	12,005	12,607	442,764	0.98	0.55	90 - 140
Modified AASHTO										
Type IV	54	29.63	24.37	681	7,894	9,594	233,854	0.90	0.48	70 - 115
Type V	63	30.98	32.02	887	15,477	14,973	479,458	0.93	0.55	90 - 140
Type VI	72	35.56	36.44	941	18,871	18,417	671,088	0.94	0.55	110 - 150
Modified Colorado G68/6	68	33.99	34.01	701	12,548	12,544	426,575	0.91	0.53	105 - 140
Modified Washington										
Series 80/6	50	27.24	22.76	513	5,844	6,994	159,191	0.94	0.50	70 - 105
Series 100/6	58	30.01	27.99	591	8,549	9,166	256,560	0.93	0.52	80 - 125
Series 120/6	73.5	37.68	35.82	688	12,619	13,275	475,502	0.91	0.51	85 - 140
Series 14/6	73.5	35.31	38.19	736	15,122	13,985	534,037	0.89	0.54	105 - 150
Modified Bulb-T										
BT 48/6	48	23.53	24.47	557	7,553	7,264	177,736	0.94	0.55	70 - 105
BT 60/6	60	29.59	30.41	629	10,432	10,154	308,722	0.93	0.54	80 - 130
BT 72/6	72	35.64	36.36	701	13,606	13,340	484,993	0.92	0.53	100 - 144

*Varies with concrete compressive strength at transfer and at 28 days   1 in = 25.4 mm; 1 ft = 0.305 m

TABLE 6 - PROPERTIES OF NEW CPCI METRIC SECTIONS

Agency	Girder Type	h (mm)	y_t (mm)	y_b (mm)	Area (mm^2)	S_t ($mm^3 \times 10^6$)	S_b ($mm^3 \times 10^6$)	Inertia ($mm^4 \times 10^6$)	α	ρ	Span Range* (ft)
CPCI Canadian Metric Sections	900	900	502	398	218,000	38.5	48.5	19,310	0.86	0.44	40 - 60
	1200	1200	673	527	320,000	80.0	102.2	53,868	0.92	0.47	55 - 80
	1400	1400	765	635	413,000	134.1	161.5	102,580	0.97	0.51	80 - 100
	1900	1900	960	940	544,000	279.6	285.6	268,420	0.96	0.55	100 - 135
	1900A	1900	960	940	601,000	297.5	303.8	285,570	0.92	0.53	100 - 135
	2300	2300	1165	1135	604,000	370.6	380.4	431,790	0.95	0.54	140 - 170
	2300A	2300	1163	1137	673,000	397.4	406.5	462,220	0.91	0.52	140 - 170

*Varies with concrete compressive strength at transfer and at 28 days

1 mm = 0.0394 in; 1 ft = 0.305 m

APPENDIX D - COST DATA

The following unit costs were obtained from seven agencies and one producer.

California

In the State of California, the majority of highway bridges were cast-in-place, post-tensioned concrete box girders. This type of bridge has been successfully built since the early 1950's. Contractors were experienced with this type of construction and the cost was competitive. These bridges were considered aesthetically pleasing and were cheaper to manufacture than precast girders. It was felt that as the cost of lumber increases the cost of cast-in-place boxes would increase and possibly precast bridge girders might become more competitive.

Precast, pretensioned girders were mostly used in coastal areas and where traffic could not be interrupted during construction. Cost data were compiled on a global unit cost basis. For 1978, the cost of precast, prestressed concrete was $714.18 per composite cubic yard. In October, 1979, this cost was estimated at about $800 per composite cubic yard. This included the cost of prestressing steel, reinforcing bars, Class A concrete, and labor.

Illinois

In the State of Illinois, unit costs for in-place girders, reinforcing steel, and concrete for decks were as follows:

Item	Cost	
	December 1978	June 1979
Girder Type 36 in	$46/ft	$67/ft
Girder Type 42 in	55	69
Girder Type 48 in	61	94
Girder Type 54 in	78	74
Reinforcing Bars	43¢/lb	52¢/lb
Epoxy coated (top bars)	67¢/lb	87¢/lb
Class X concrete	$250/cu yd.	$250/cu yd.

1 in = 25.4 mm; 1 ft = 0.305 m; 1 yd = 0.914 m; 1 lb = 4.45 N

Differences in unit costs occurred between December 1978 and June 1979. While the cost of 48 in (1.22 m) girders increased tremendously in 1979, cost of 54 in (1.37 m) girders decreased. These inconsistencies are due to factors discussed in the body of the report under "Availability and Costs."

<u>Oregon</u>

In the State of Oregon, unit costs used to estimate the cost of bridge construction during the fourth quarter of 1979 were as follows:

Girder	Girder Cost $/ft (excluding transportation)	Overall Bridge Cost $/sq ft of Deck Area
Type II	65	41
Type III	75	43
Type IV	95	44
Bulb-T	105	47

1 ft = 0.305 m

<u>Texas</u>

In the State of Texas, unit costs were based on bids received during the period of January to September 1979. These costs were as follows:

Item	Cost
Girder Type A	$59.80/ft
Girder Type B	44.68
Girder Type C	51.40
Girder Type 54	54.80
Girder Type IV	68.68
Cast-In-Place Decks	$7.30/sq ft
In-Place Concrete for Decks	$214/cu yd
In-Place Steel for Decks	41¢/lb
In-Place Strands for Decks	20¢/ft

1 ft = 0.305 m; 1 yd = 0.914 m; 1 lb = 4.45 N

In 1978, 74 state highway bridges were built in the State of Texas at a cost of $16.86/sq ft for the superstructure.

Girder Type A, identical to AASHTO-PCI standard girder Type I was the smallest section. It required the least amount of concrete, strands, and reinforcing steel. However, the unit cost was relatively very high because this girder has been used mainly ror bridge widening.

Virginia

As of June 1979, unit costs in the State of Virginia were as follows:

Item	Cost
AASHTO Type II	$31-47/ft
AASHTO Type III	37-56
AASHTO Type IV	56-76
AASHTO Type V	69-77
AASHTO Type VI	89-92
Class A4 Concrete for Decks	$240/cu yd
Reinforcing Steel	27¢/lb
Epoxy Coated Bars	52¢/lb

1 ft = 0.305 m; 1 yd = 0.914 m; 1 lb = 4.45 N

The wide range of cost for the girders reflected cost variation due to factors discussed in the report.

Washington

As of August 1979, unit costs for comparative cost estimates of alternate designs were as follows:

Item	Cost
Girders 40 Series	$50/ft
Girders 60 Series	51
Girders 80 Series	57
Girders 100 Series	64
Girders 120 Series	70
Concrete Class AX for Decks	$200-250

1 ft = 0.305 m

Wisconsin

As of August 1979, costs in the State of Wisconsin were as follows:

Item	Cost
Girder Type II	$49/ft
Girder Type III	62
Girder Type IV	79
Girder Type 70 inch	84
Finished Superstructure	$11.68-14.04/sq ft

1 ft = 0.305 m

Stanley Structures

In October of 1979, pretensioned girders produced by Stanley Structures, located in Denver, Colorado, were delivered at sites within metropolitan Denver at the following cost:

Girder G54	$65/ft ($213/m)
Girder G68	$70/ft ($230/m)

These costs varied slightly with number of strands in each girder. They did not include cost of bearings or contractors' profit.

APPENDIX E - COMPUTER PROGRAM "BRIDGE"

To perform the parametric studies discussed in this report, a computer program was developed. This appendix contains Program Documentation, Source Listing, and Sample Problems.

Program Documentation

Program Name: BRIDGE

Language: Fortran IV

Purpose and Capabilities

The main purpose of Program BRIDGE is to compute a cost index per unit surface area of simply supported bridges built with precast prestressed I or T-girders and cast-in-place concrete deck. Program BRIDGE generates additional information including:

1. Deck thickness and main deck reinforcement
2. Non-composite and composite sectional properties
3. Dead and live load moments and impact factor
4. Required number of strands
5. Stress levels in concrete and strands at prestress transfer and service load conditions
6. Midspan deflection
7. Total concrete and reinforcement quantities

All computations are made for an interior girder. Design procedures are based on 1977 AASHTO Specifications(8) and 1978 to 1980 Interim Specifications(12), and for HS20 loading. Details of data input and output, design assumptions, capabilities, and limitations of Program BRIDGE are discussed below in the solution steps.

Solution Steps

Program BRIDGE is divided into eight solution steps as follows:

1. Input of geometric and material properties, and relative unit costs of materials.
2. Allowable material properties are computed.
3. Deck thickness and reinforcement are determined.

4. Non-composite and composite sectional properties are calculated.
5. Design loads and moments are computed.
6. Number of strands required to satisfy service and strength conditions are determined.
7. Midspan deflections are computed.
8. Total material quantities and cost index per unit surface area of bridge are calculated.

Data input and values computed within each step are printed with adequate explanation as output. Each step is now described in detail.

Step 1 - Data Input Each case problem requires input of data on a set of 12 punched cards. The last card is a control card to determine whether a new problem follows. Therefore, several problems can be solved consecutively.

For each sample problem, input of some data is optional. Where no information is provided, the program assumes values stored as default options. Minimum information needed is girder dimensions, span length, and girder spacing.

A detailed description of data input and default options is presented in the following paragraphs. After discussing each of the eight solution steps, a summary of data input for each case problem is presented under the heading "User's Input Instructions."

Twelve punched cards contain the data input of each case problem. Each card has 80 columns. Two formats are used.

The A format code is convenient for input of character strings. These may consist of combinations of letters, digits or symbols. The A format is useful for titles identifying case problems.

The F format code is used to input real values. In Program BRIDGE all values or quantities are input using F12.0 format. Therefore, twelve columns are allocated for each value. Each value should have a decimal point and may have up to 11 digits. If no value is punched within the allocated columns, a value of zero is assumed. As will be seen later, if a variable is given a value of zero, then a default option value is assumed.

Format code for data on each of the 12 cards is summarized in the "User's Input Instructions" section. Data input on each card follows:

Cards 1, 2 and 3 are for identification of case problem. These are job description or title cards.

Card 4 is to input the span length, SL, in feet and girder spacing, GS, in feet.

SL is the span length, center to center of supports for simply supported girders. Program BRIDGE was designed to handle spans of 70 to 180 ft (2.14 to 54.9 m).

GS is the center to center spacing of girders. Maximum girder spacing is a function of the effective deck span. This is discussed in Step 3. Maximum girder spacing accepted by Program BRIDGE is approximately 11 ft (3.35 m).

Cards 5 and 6 are to input dimensions defining the girder cross section, in inches. These are identified in Figure 31. Card 5 reads horizontal dimensions B1, B2, B3, and B4. Card 6 reads vertical dimensions D1, D2, D3, D4, D5 and D6. For sections with profile as shown in Figure 31.b, dimensions B4 and D4 are equal to zero.

Card 7 reads parameters CTC, CSC and SWW. CTC is the center-to-center spacing of strands in inches at midspan. It is assumed to be the same in both vertical and horizontal directions and depends on the strand size. In Program BRIDGE, size of all strands is assumed to be 1/2 in (12.7 mm). If the minimum clear spacing between strands is three times the diameter of the steel[8], then CTC is equal to 2 in (50.8 mm) This was discussed earlier under Fabrication Details. If strands are bundled, then CTC equals 0.5 in (12.7 mm). Default option for CTC is 2 in (50.8 mm), i.e. if CTC is input as zero, then the program assumes CTC equal to 2 in (50.8 mm).

CSC is the concrete surface to center of strand distance in inches. This dimension reflects the amount of concrete cover

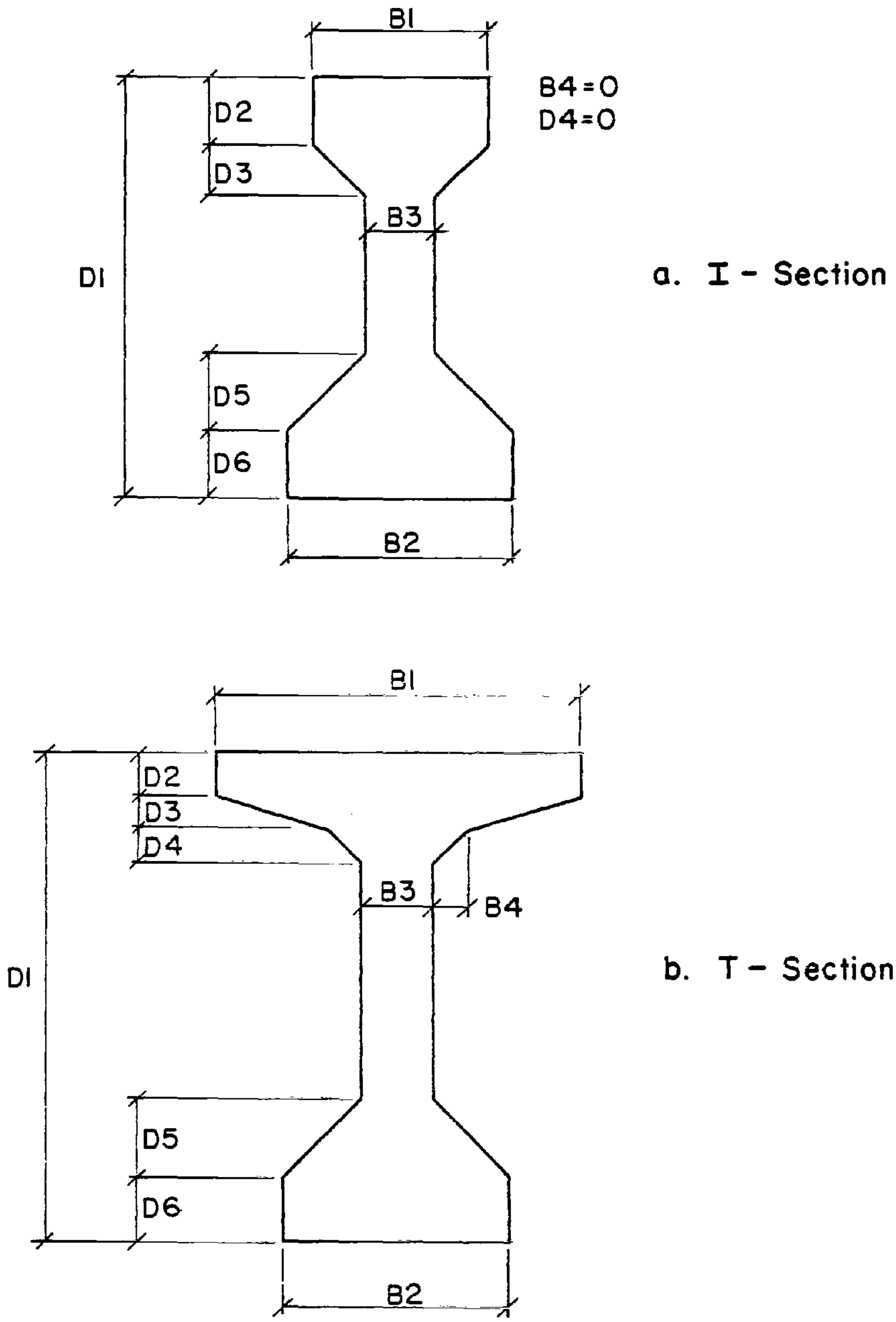

Figure 31 Dimensions Defining Girder Cross Section for Computer Input

in the bottom flange at midspan. In most states, this dimension is 2 in (50.8 mm). However, some states use 1.75 in (44.5 mm). The default option for CSC is 2 in (50.8 mm).

SWW is the number of strands in one row, within the web width at midspan. This parameter is needed in Step 6, where number of required strands is determined. If all strands are located within the bottom flange at midspan, SWW is ignored. But if strands are needed in the web, the maximum number of strands within a row depends on the value of CTC and CSC defined above. For example, if web width is 8 in (203 mm), and both concrete cover, CSC, and spacing of strands, CTC, are equal to 2 in (50.8 mm), then it is possible to place three strands within the web width. But common practice in several states is to place only 2 strands, side by side, within the web. Therefore, the default option of SWW = 2 is introduced in the program.

Card 8 provides the value of FCP, FCPI, WC, WCD, WG, and WD. FCP is the girder concrete compressive strength at 28 days in psi units. This value could be as high at 8,000 psi (55.2 MPa). Default option for FCP is 6000 psi (41.4 MPa).

FCPI is the concrete compressive strength at transfer of prestress in psi units. Default option is 4500 psi (31 MPa).

WC and WCD are the unit weights of the girder and deck concretes, respectively, in pcf. Default options set these concrete unit weights at 145 pcf (2,323 kg/m^3).

WG and WD are the unit weights of the girder and deck in pcf units. It accounts for weight of concrete, reinforcing steel, and strands. Default option values are 150 pcf (2,403 kg/m^3).

Card 9 is to input the strand's yield stress, FPY, in psi, the strand's modulus of elasticity, EPS, in psi, and total prestress losses, XLS, in psi. Default options are 23,000 psi (159 MPa), 28,000,000 psi (193,000 MPa), and 45,000 psi (310 MPa), respectively.

Card 10 inputs a stiffness reduction factor, SRF, that is used to compute the girder deflections. It accounts for the concrete creep and prestress losses. The value of SRF = 0.55

suggested in State of Illinois Prestressed Concrete Design Manual[27] is adopted in this computer program as a default option. Further explanation about the stiffness redution factor is discussed below in Step 7.

Card 11 provides the relative unit cost indexes of materials as discussed earlier:

RUCG for girder concrete (Default Option 1 unit/lb)

RUCD for deck concrete (Default Option 1 unit/lb)

RUS for strand (Default Option 8 units/lb)

RUR for deck bottom reinforcing steel (Default Option 9 units/lb)

RUE for deck top epoxy coated reinforcing steel (Default Option 12 units/lb). If top flexural reinforcement of deck is not epoxy coated, RUE should be equal to RUR.

Card 12 is a control card. If a new set of data is to be input, CONTINUE is punched, starting in Column 1. In the last data set, Card 12 reads END starting in Column 1.

In addition to the data input, other data are assigned internally and therefore assumed constant for all case problems.

Concrete compressive strength of deck at 28 days is assumed 4000 psi. Strands are assumed to be Grade 270. Stress-strain characteristics of strand shown in Figure 32 are used to compute the nominal flexural strength of the composite section in Step 6.

Step 2 - Allowable Stresses Allowable concrete stresses at transfer and at service condition are computed according to AASHTO Specifications.[8] Strand effective stress after losses, is computed. A check is made to verify that this value falls within the range specified.[8] If it is outside this range, a message is printed to this effect as discussed below under "Error Messages." In this case all other computations are by-passed, and a new case problem is processed. If the strand effective stress is within the acceptable range, the program proceeds with the next step.

Step 3 - Deck Thickness and Reinforcement Determination of the cast-in-place slab (or deck) thickness and reinforcement have been adopted from design aids prepared by Washington State

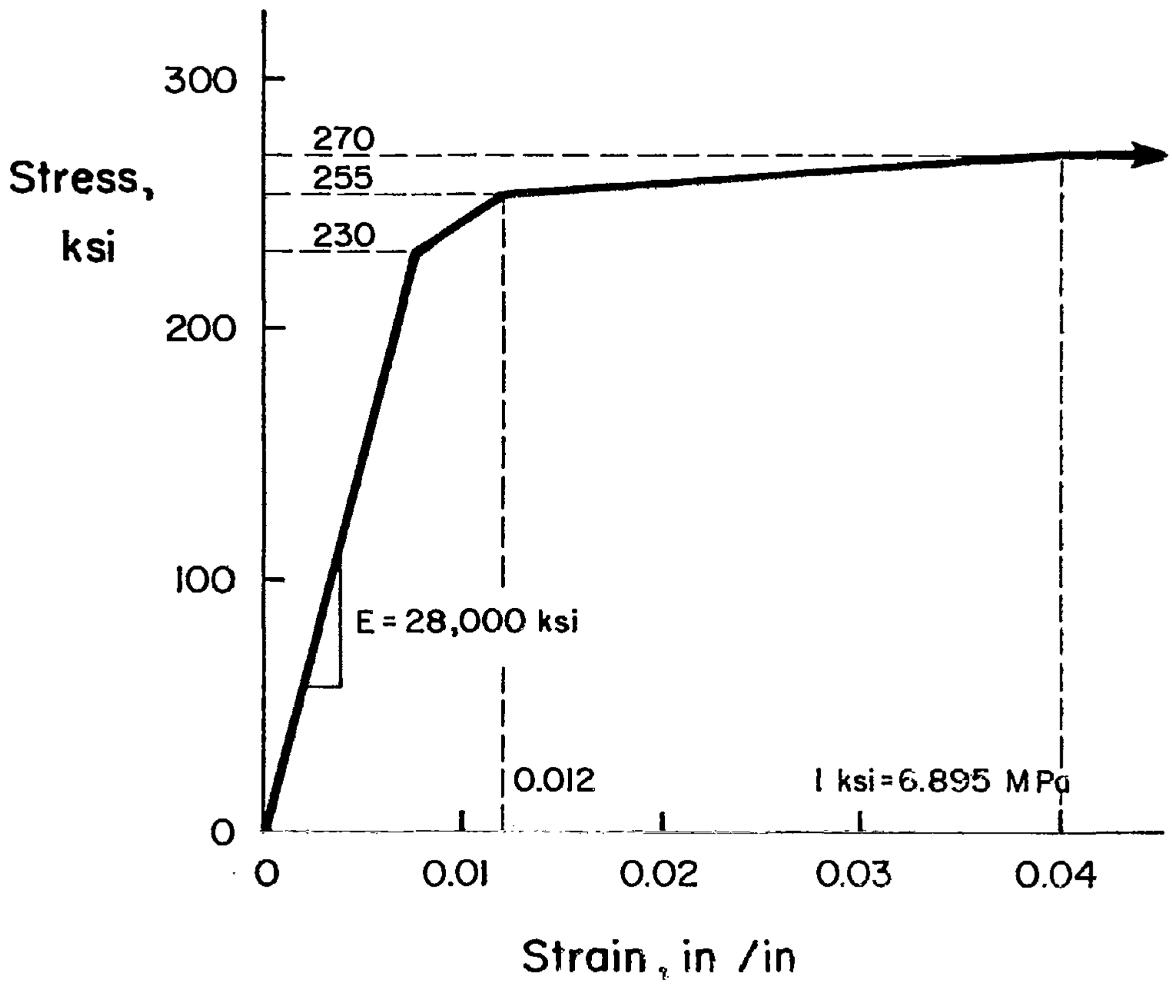

Figure 32 Assumed Stress-Strain Characteristics of Strands

Department of Transportation.[28] It is based on the following assumptions:

1. Concrete compressive strength at 28 days is 4000 psi (27.6 MPa).
2. Reinforcing steel is Grade 60 (414 MPa).
3. Interior spans are considered with equal top and bottom flexural reinforcement.
4. Reinforcing steel is perpendicular to traffic direction.

An effective slab span is computed based on center-to-center spacing of girders, width of top flange and deck thickness. The effective slab span is illustrated in Figure 33. Information given in Table 7 was stored in Program BRIDGE. It served as a basis for deck design. If the effective slab span exceeds 10 ft (3.05 m), a message is printed to that effect.

<u>Step 4 - Sectional Properties</u> Properties determined for girder (non-composite) and girder slab (composite) sections are:

1. Location of center of gravity
2. Cross sectional area
3. Modulus of section for top and bottom fibers
4. Moment of inertia

In the composite section, the effective top flange width is the smallest of:

1. Girder span divided by four
2. Girder spacing
3. Twelve times deck thickness plus web width, B3, shown in Figure 31.

In calculations of the composite section properties, a transformed deck-girder section is considered. However, transformed area of strands is neglected.

<u>Step 5 - Design Loads and Moments</u> Dead loads are based on cross sectional area of girder and deck calculated in the previous step and concrete unit weight specified in Step 1. Live load considered in Program BRIDGE is HS 20-44 loading. Impact factor is based on 50/(Span length in feet + 125) , and does not exceed 30 percent.

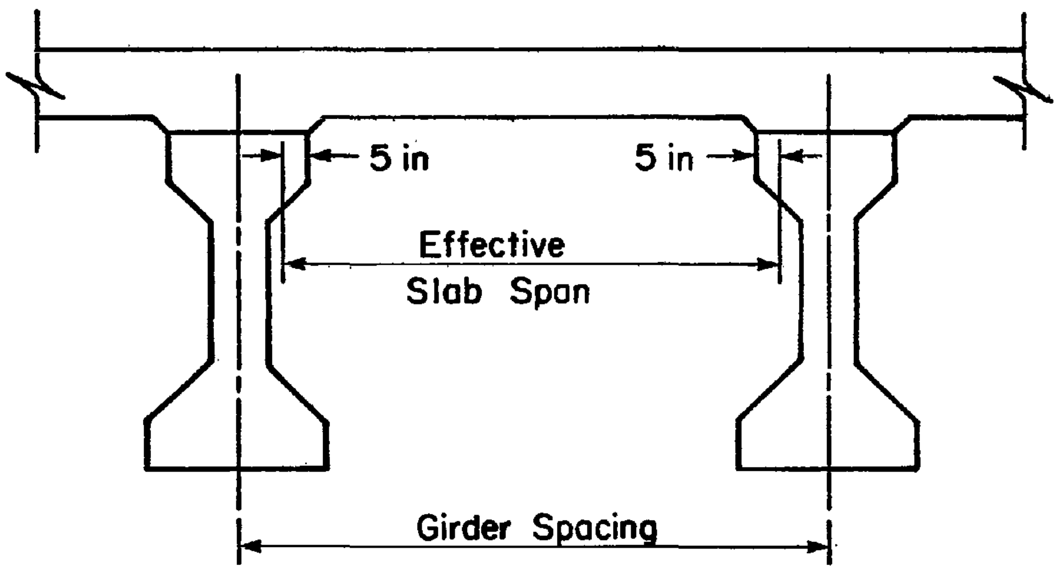

Figure 33 Effective Slab Span

TABLE 7 - DECK DESIGN[28]

Effective Slab Span (ft)	Slab Thickness (in)	Slab Reinforcement*	
		Bar Size	Spacing (in)
1 to 3 inclusive	7	No. 5	10.0
3 to 4 inclusive	7	No. 5	8.5
4 to 5 inclusive	7	No. 5	7.5
5 to 6 inclusive	7	No. 5	6.5
6 to 7 inclusive	7	No. 6	8.0
7 to 8 inclusive	7-1/2	No. 6	8.0
8 to 9 inclusive	8	No. 6	8.0
9 to 10 inclusive	8-1/2	No. 6	8.0

* Reinforcement shown is for each of top and bottom layers.

1 in = 25.4 mm

Midspan dead and live load moments are computed for simple spans. Live load moments have been adopted from Appendix A of the AASHTO Specifications.[8] They are summarized in Table 8. For intermediate spans, moments are computed through linear interpolation. These moments are per lane width of 10 ft (3.05 m). They are proportioned to determine live load moment per girder spacing. The strength required is based on 1.3 Dead Load + $\frac{5}{3}$ (Live Load + Impact) .

Step 6 - Required Number of Strands Required number of strands is determined through incremental analysis. For each analysis step, the total number of strands is increased by one. Top and bottom concrete stresses are checked at midspan for transfer and service conditions. Flexural strength is also computed. Required number of strands is obtained when concrete stresses and flexural strength conditions[8] are satisfied at midspan.

Location of strands is chosen to achieve maximum prestress eccentricity. These locations are governed by allowable concrete cover and strand spacing. These values are specified in Step 1. They include the concrete surface to center of strand distance CSC, and the center to center spacing of strands CTC.

Strands are placed in rows as shown in Figure 34. The first row is located at a distance CSC from the bottom of the girder. Subsequent rows are spaced CTC apart, Within each row, strands are spaced a distance CTC apart. Side concrete cover is governed by distance CSC.

Strands are positioned in the bottom row first, and by moving to higher rows as required. This is to achieve maximum eccentricity. If the total number of strands required is large, strands may be placed within the web. Common practice is to place two strands side-by-side in each row within the web width. This is discussed under data input for Card 7, in Step 1. If a different number of strands in one row within the web width is desirable, it should be specified in the data input of Card 7.

During the incremental analysis to determine the number of strands, initial and effective prestress levels are computed.

TABLE 8 - HS20-44 MOMENTS[8]

Span (ft)	Bending Moment (ft. kip)
70	985.6
80	1,164.9
90	1,344.4
100	1,524.0
110	1,703.6
120	1,883.3
130	2,063.1
140	2,242.8
150	2,475.1
160	2,768.0
170	3,077.1
180	3,402.1

1 ft = 0.305 m
1 ft.kip = 1.36 kN.m

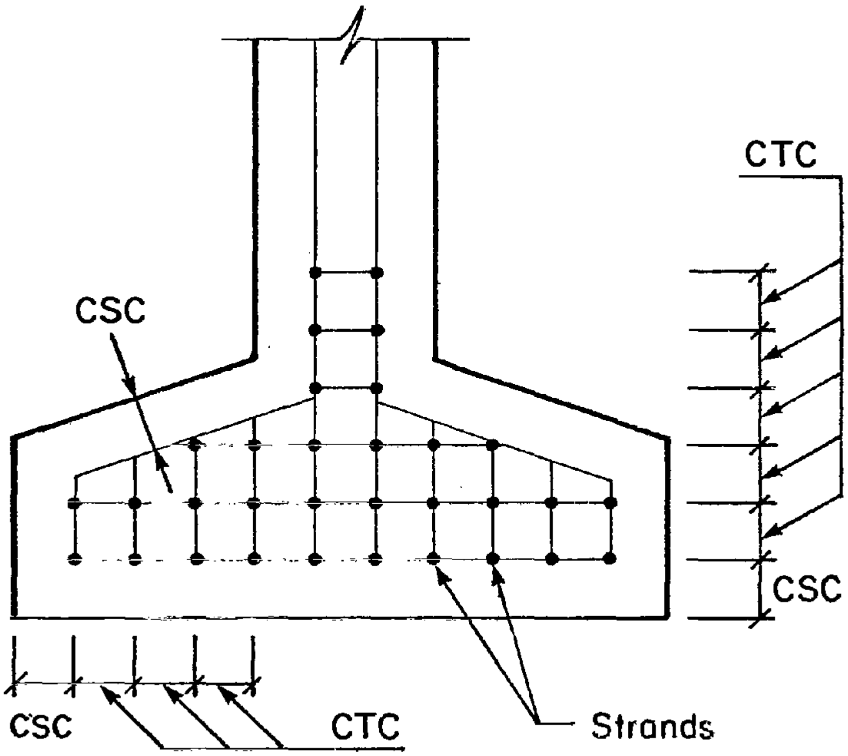

Figure 34 Strands Locations

For each prestress level, top and bottom concrete stresses are checked against the allowable stresses computed in Step 2. When the required number of strands satisfying allowable concrete stresses is reached, the nominal flexural strength, M_n, is calculated through an iteration process.[29] The required flexural strength, M_u is checked against the nominal flexural strength, M_n, times the strength reduction of 0.9. Minimum steel requirement of Section 1.6.10.B of the AASHTO Specifications[8] is also checked. If these requirements are not satisfied, the number of strands is increased by one strand.

Program BRIDGE contains two criteria governing the number of strands that can be placed in a section:

1. No strands can be placed above the top core or kern[1] of the section.
2. An arbitrary upper limit of 100 strands has been set in the program. This limit exceeds by about 30 percent the maximum number of strands needed in any of the sections analyzed in this report. This criterion may control where strands are bundled. This condition is obtained by specifying a center-to-center spacing of strands, CTC, equal to the strand diameter of 0.5 in (12.7 mm) in Step 1.

If any of the above limits on number of strands is reached without satisfying concrete stress and flexural strength requirements a message is printed to this effect. If the reinforcement index exceeds 0.3[8], a warning message is printed. When stress and strength requirements are satisfied, the program proceeds with the next step.

Step 7 - Midspan Deflections The effect of prestress in a pretensioned member is to produce an upward deflection, or camber. The weight of the girder counteracts this camber. The net effect is usually a camber, but it could be a downward deflection or sag. Due to prestress, the concrete creeps. As a result the camber or sag increases until the girder is transported to the final position and the cast-in-place concrete deck is placed. The net effect of the cast-in-place deck is a downward deflection.

Due to creep, the level of prestress decreases. The net effect of creep and loss of prestress on the girder deflection at erection time is an increase in the camber or sag. A magnification of the elastic deflection by 1.82 is suggested in the State of Illinois, Department of Transportation Design manual.[27] This increase in camber or sag can be accounted for by decreasing the stiffness of the girder. This stiffness reduction factor, SRF, is equal to 1/1.82, i.e. 0.55. This is the default option for data input of SRF.

Camber due to prestress depends on the magnitude and eccentricity of the prestressing force, number and location of draped strands, or number and length of blanketed strands. In Program BRIDGE, camber due to prestress is computed assuming that all strands are straight and bonded over their entire length. The effect of draping or blanketing strands is to decrease the magnitude of the camber.[30]

Step 8 - Cost Index Per Unit Surface Area of Bridge Cost index per unit surface area of bridge computed in Program BRIDGE provides a means of comparing the cost effectiveness of girder cross sections. Total weight of materials is computed for a width of deck equal to the girder spacing. These weights are reduced to weights of materials per unit surface area of bridge. The cost index per unit surface area of bridge is based on the relative unit costs input in Card 11 of Step 1.

Warning Messages

The following is a list of all warning messages and their explanations:

1. EFFECTIVE PRESTRESS IS OUT OF RANGE
   This message appears if the effective prestress after losses is less than 50 percent of the strand strength or greater than 80 percent of the yield strength of the strands.[8]

2. EFFECTIVE DECK SPAN EXCEEDS 10 FT.
   Effective deck slab span computed in Step 3 is limited to 10 ft (3.05 m). Values up to 10 ft (3.05 m) are handled by the program.
3. GIRDER SPAN IS OUT OF RANGE
   The program computes midspan moments due to HS20-44 loading for spans between 70 and 180 ft (21.3 and 54.9 m).
4. CANNOT SATISFY STRESS AND STRENGTH REQUIREMENTS
   Concrete stress and flexural strength requirements are violated at midspan.
5. REINFORCEMENT INDEX EXCEEDS 0.3
   Reinforcement index should not exceed 0.3 to prevent brittle failures.[8]

If any of the above messages is encountered, processing of the problem is terminated, and control is transferred to the subsequent case problem.

User's Input Instructions

This section summarizes data input for each case problem. Detailed explanations of variables and default options are given in the preceding section entitled "Solution Steps." For each case problem, data input consists of 12 punched cards containing the information shown in Table 9. Each variable is described below:

Cards 1,2, and 3

Title to identify case problem

Card 4

SL - Span length, ft

GS - Girder spacing, ft

Card 5

B1, B2, B3, and B4 - Horizontal dimensions, defining girder profile as identified in Figure 31, in

Card 6

D1, D2, D3, D4, D5, D6 - Vertical dimensions defining girder pro- profile as identified in Figure 31, in

Card 7

CTC - center to center spacing of strands, in (D.O.* = 2)

CSC - concrete cover, distance from concrete surface to center of strand, in (D.O. = 2)

SWW - number of strands in a row within web width (D.O. = 2)

Card 8

FCP - Girder concrete compressive strength at 28 days, psi (D.O. = 6000)

FCPI - Girder concrete compressive strength at transfer, psi (D.O. = 4500)

WC - Unit weight of girder concrete, pcf (D.O. = 145)

WCD - Unit weight of deck concrete, pcf (D.O. = 145)

WG - Unit weight of girder, pcf (D.O. = 150)

WD - Unit weight of deck, pcf (D.O. = 150)

*Default option, i.e., if zero value is assigned, the default value is assumed.

TABLE 9 - DATA INPUT

Card Number	Parameters	Format
1, 2, 3	TITLE	20A4
4	SL, GS	2F12.0
5	B1, B2, B3, B4	4F12.0
6	D1, D2, D3, D4, D5, D6	6F12.0
7	CTC, CSC, SWW	3F12.0
8	FCP, FCPI, WC, WCD, WG, WD	6F12.0
9	FPY, EPS, XLS	3F12.0
10	SRF	1F12.0
11	RUCG, RUCD, RUS, RUR, RUE	5F12.0
12	CONTINUE or END (Starting in Column 1)	20A4

Card 9

FPY - Yield stress of strand, psi (D.O. = 230,000)

EPS - Modulus of elasticity of strand, psi (D.O. = 28,000,000)

XLS - Total prestress losses, psi (D.O. = 45,000)

Card 10

SRF - Stiffness reduction factor due to creep (D.O. = 0.55)

Card 11

RUCG - Relative unit cost index of girder concrete, unit per lb (D.O. = 1)

RUCD - Relative unit cost index of deck concrete, unit per lb (D.O. = 1)

RUS - Relative unit cost index of strands, unit per lb (D.O. = 8)

RUR - Relative unit cost index of bottom deck reinforcement, unit per lb (D.O. = 9).

RUE - Relative unit cost index of epoxy coated top deck reinforcement, unit per lb (D.O. 12)

Card 12

This is a control card.

CONTINUE is entered starting in column 1 if a new set of data is input.

END is entered starting in column 1 if it is the last set of data.

<u>Source Listing</u>

```
C     PROGRAM BRIDGE(INPUT,OUTPUT,TAPE8=INPUT,TAPE3=OUTPUT)
C     ***  STRUCTURAL DESIGN OF PRESTRESSED CONCRETE BRIDGE GIRDERS  ***
C   THIS PROGRAM CALCULATES THE NUMBER OF STRANDS AND THE COST INDEX PER
C   UNIT SURFACE AREA FOR A GIVEN PRESTRESSED CONCRETE BRIDGE GIRDER
C   FHWA CONTRACT NO. DOT-FH-11-9598  / CR4475/4321
C
      DIMENSION TITLE(20),BS(2),ABAR(2),X(3),A(7),Y(7),XJ(7),DM(12),
     1             SLL(12),NR(50)
      DATA ABAR/4HNO.5,4HNO.6/,CONT/4HCONT/
      DATA DM/985.6,1164.9,1344.4,1524.0,1703.6,1883.3,2063.1,2242.8,
     1         2475.1,2768.0,3077.1,3402.1/
      DATA SLL/70.,80.,90.,100.,110.,120.,130.,140.,150.,160.,170.,180./
      IW=3
      IR=8
C***********************************************************************
C
C **STEP(1) INPUT DATA **
C
C   GEOMETRIC PROPERTIES
C     SL=GIRDER SPAN LENGTH(FT)
C     GS=GIRDER SPACING(FT)
C     B1,B2,B3, AND B4=HORIZONTAL DIMENSIONS OF GIRDER CROSS SECTION(IN)
C     D1,D2,D3,D4,D5,AND D6=VERTICAL DIMENSIONS OF GIRDER  (IN)
C    CTC=CENTER TO CENTER DISTANCE OF STRANDS(IN)
C       =2(DEFAULT OPTION)
C    CSC=CONCRETE SURFACE TO CENTER OF STRANDS(IN)
C       =2(DEFAULT OPTION)
C    SWW=MAXIMUM NUMBER OF STRANDS WITHIN WEB WIDTH
C       =2(DEFAULT OPTION)
C   MATERIAL PROPERTIES
C    CONCRETE
C     FCP=SPECIFIED GIRDER CONCRETE STRENGTH(PSI)
C     FCP=6000 (DEFAULT OPTION)
C     FCPI=CONCRETE STRENGTH AT PRESTRESS TRANSFER(PSI)
C         =4500(DEFAULT OPTION)
C     WC=UNIT WEIGHT OF GIRDER CONCRETE(PCF)
C       =145(DEFAULT OPTION)
C     WCD=UNIT WEIGHT OF DECK CONCRETE(PCF)
C       =145(DEFAULT OPTION)
C     WG=UNIT WEIGHT OF GIRDER(PCF)
C       =150(DEFAULT OPTION)
C     WD=UNIT WEIGHT OF DECK(PCF)
C       =150(DEFAULT DOPTION)
C    STRAND
C     FPY=SPECIFIED YIELD STRESS OF STRAND(PSI)
C       =230000(DEFAULT OPTION)
C     EPS=MODULUS OF ELASTICITY OF STRAND(PSI)
C       =28000000(DEFAULT OPTION)
C     XLS=TOTAL PRESTRESS LOSSES(PSI)
C       =45000(DEFAULT OPTION)
C
C     SRF=STIFFNESS REDUCTION FACTOR DUE TO CREEP AND PRESTRESS LOSS
C       =0.55(DEFAULT OPTION)
C
C   RELATIVE UNIT COSTS
C     RUCG=RELATIVE UNIT COST OF GIRDER CONCRETE(UNIT/LB)
C       =1(DEFAULT OPTION)
C     RUCD=RELATIVE UNIT COST OF DECK CONCRETE(UNIT/LB)
C       =1(DEFAULT OPTION)
C     RUS=RELATIVE UNIT COST OF STRAND(UNIT/LB)
C       =8(DEFAULT OPTION)
C     RUR=RELATIVE UNIT COST OF DECK REINFORCEMENT(UNIT/LB)
C       =9(DEFAULT OPTION)
C     RUE=RELATIVE UNIT COST OF EPOXY COATED DECK REINFORCEMENT
C       =12(DEFAULT OPTION)   IF NO EPOXY COATED BAR IS USED, THE VALUE
C                             EQUAL TO RUR IS TO BE GIVEN FOR RUE
C
C  READ EXAMPLE IDENTIFICATION
   10 WRITE(IW,20)
   20 FORMAT(1H1)
      DO 50 I=1,3
      READ(IR,30) (TITLE(J),J=1,20)
   30 FORMAT(20A4)
      WRITE(IW,40) (TITLE(J),J=1,20)
   40 FORMAT(1H ,5X,'** ',20A4,' **')
   50 CONTINUE
```

```
C  READ GEOMETRIC PROPERTIES
      READ(IR,60) SL,GS
   60 FORMAT(2F12.0)
      READ(IR,70) B1,B2,B3,B4
   70 FORMAT(4F12.0)
      READ(IR,80) D1,D2,D3,D4,D5,D6
   80 FORMAT(6F12.0)
      READ(IR,90) CTC,CSC,SWW
   90 FORMAT(3F12.0)
      IF(CTC.EQ.0.) CTC=2.0
      IF(CSC.EQ.0.) CSC=2.0
      IF(SWW.EQ.0.) SWW=2.0
      WRITE(IW,100)
  100 FORMAT(1H0,4X,'**(1) INPUT DATA **')
      WRITE(IW,110)
  110 FORMAT(1H0, 5X,'GEOMETRIC PROPERTIES')
      WRITE(IW,120) SL, GS
  120 FORMAT(1H0,10X,'GIRDER SPAN LENGTH(FT)    SL=',F10.2,/
     1              11X,'GIRDER SPACING(FT)        GS=',F10.2)
      WRITE(IW,130) B1,B2,B3,B4
  130 FORMAT(1H0,10X,'HORIZONTAL DIMENSIONS OF GIRDER CROSS SECTION',/
     1              12X,'B1=',F10.2,2X,'B2=',F10.2,2X,'B3=',F10.2,2X,'B4=',
     2                        F10.2)
      WRITE(IW,140) D1,D2,D3,D4,D5,D6
  140 FORMAT(1H0,10X,'VERTICAL DIMENSIONS OF GIRDER CROSS SECTION',/
     1              12X,'D1=',F10.2,2X,'D2=',F10.2,2X,'D3=',F10.2,2X,'D4=',
     2                        F10.2,2X,'D5=',F10.2,2X,'D6=',F10.2)
      WRITE(IW,150) CTC,CSC,SWW
  150 FORMAT(1H0,10X,'CENTER TO CENTER SPACING OF STRANDS(IN)      CTC=',
     1F6.2,/     11X,'CONCRETE SURFACE TO CENTER OF STRANDS(IN)   CSC=',
     2F6.2,/     11X,'NUMBER OF STRANDS WITHIN WEB WIDTH          SWW=',
     3F6.2)
C
C  READ MATERIAL PROPERTIES
C
C   CONCRETE
      READ(IR,160) FCP,FCPI,WC,WCD,WG,WD
  160 FORMAT(6F12.0)
      IF(FCP.EQ.0.) FCP=6000.
      IF(FCPI.EQ.0.) FCPI=4500.
      IF(WC.EQ.0.) WC=145.
      IF(WCD.EQ.0.) WCD=145.
      IF(WG.EQ.0.) WG=150.
      IF(WD.EQ.0.) WD=150.
C    FIXED DATA
C    FCD=SPECIFIED CONCRETE STRENGTH FOR DECK(PSI)
      FCD=4000.
C
C   STRAND
      READ(IR,170) FPY,EPS,XLS
  170 FORMAT(3F12.0)
      IF(FPY.EQ.0.) FPY=230000.
      IF(EPS.EQ.0.) EPS=28000000.
      IF(XLS.EQ.0.) XLS=45000.
C    FIXED DATA
C    FPU=SPECIFIED ULTIMATE STRENGTH OF STRAND(PSI)
C    FPM=SPECIFIED STRAND STRESS LEVEL, WHICH IS POSITIONED BETWEEN
C         FPU AND FPY TO FORM TRILINEAR STRESS-STRAIN CURVE(PSI)
C    SY=STRAIN CORRESPONDING TO FPY
C    SM=STRAIN CORRESPONDING TO FPM
C    SU=STRAIN CORRESPONDING TO FPU
C    ASTD=NOMINAL AREA OF STRAND(IN2)
C    UST=NOMINAL WEIGHT OF STRAND(LBS/FT)
C
C   DECK REINFORCEMENT
C    UR5=NOMINAL WEIGHT OF NO.5 BAR(LBS/FT)
C    UR6=NOMINAL WEIGHT OF NO.6 BAR(LBS/FT)
      FPU=270000.
      FPM=255000.
      SY=FPY/EPS
      SM=0.012
      SU=0.04
      ASTD=0.153
      UST=0.53
      UR5=1.043
      UR6=1.502
      WRITE(IW,180)
  180 FORMAT(1H0,5X,'MATERIAL PROPERTIES',3X,
     1                '(*=INTERNALLY ASSIGNED VALUES)')
      WRITE(IW,190)
```

```
  190 FORMAT(1H0,7X,'CONCRETE')
      WRITE(IW,200) FCP,FCPI,FCD
  200 FORMAT(1H0,10X,'SPECIFIED GIRDER CONCRETE STRENGTH(PSI)       FCP =
     1',F7.1,/ 11X,'CONCRETE STRENGTH AT PRESTRESS TRANSFER(PSI) FCPI=
     2',F7.1,/ 11X,'SPECIFIED DECK CONCRETE STRENGTH(PSI)         FCD =
     3',F7.1,'*')
      WRITE(IW,210) WC,WCD,WG,WD
  210 FORMAT(1H0,10X,'UNIT WEIGHT OF GIRDER CONCRETE(PCF)          WC =
     1',F6.1,/     11X,'UNIT WEIGHT OF DECK CONCRETE(PCF)          WCD=
     2',F6.1,/     11X,'UNIT WEIGHT OF GIRDER(PCF)                 WG =
     3',F6.1,/     11X,'UNIT WEIGHT OF DECK(PCF)                   WD =
     4',F6.1)
      WRITE(IW,220)
  220 FORMAT(1H0,7X,'STRAND')
      WRITE(IW,230) FPU,SU,FPM,SM,FPY,SY
  230 FORMAT(1H0,10X 'STRESS-STRAIN CURVE OF STRAND',/ 42X,
     1       'STRESS(PSI)', 6X,'STRAIN',/
     2  11X,'SPECIFIED ULTIMATE STRENGTH FPU=',F9.1,'*',2X,'SU=',F7.4,
     3 '*',/ 11X,'INTERMEDIATE STRESS          FPM=',F9.1,'*',2X,'SM=',
     4 F7.4,'*'/11X,'SPECIFIED YIELD STRESS       FPY=',F9.1,3X,'SY=',
     5 F7.4)
      WRITE(IW,240)EPS,XLS
  240 FORMAT(1H0,10X,'MODULUS OF ELASTICITY OF STRAND(PSI)    EPS=',F11.1
     1,      /    11X,'TOTAL PRESTRESS LOSSES (PSI)            XLS=',F11.1)
C
C  READ STIFNESS REDUCTION FACTOR
      READ(IR,250) SRF
  250 FORMAT(F12.0)
      IF(SRF.EQ.0.) SRF=0.55
      WRITE(IW,260) SRF
  260 FORMAT(1H0,5X,'GIRDER STIFFNESS REDUCTION FACTOR',10X,'SRF=',F6.3)
C
C  READ RELATIVE UNIT COST INDEX
      READ(IR,270) RUCG,RUCD,RUS,RUR,RUE
  270 FORMAT(5F12.0)
      IF(RUCG.EQ.0.) RUCG=1.
      IF(RUCD.EQ.0.) RUCD=1.
      IF(RUS.EQ.0.) RUS=8.
      IF(RUR.EQ.0.) RUR=9.
      IF(RUE.EQ.0.) RUE=12.
      WRITE(IW,280) RUCG,RUCD,RUS,RUR,RUE
  280 FORMAT(1H0,5X,'RELATIVE COST INDEX(UNIT/LB)',/
     1           11X,'GIRDER CONCRETE                    RUCG=',F6.2,/
     2           11X,'DECK CONCRETE                      RUCD=',F6.2,/
     3           11X,'STRAND                             RUS =',F6.2,/
     4           11X,'DECK REINFORCEMENT                 RUR =',F6.2,/
     5           11X,'EPOXY COATED DECK REINFORCEMENT    RUE =',F6.2)
C*****************************************************************************
C
C **STEP(2) MATERIAL PROPERTIES **
C  * INPUT *
C     FROM STEP(1)
C        FCP,FCPI,FCD,WC,WCD,WG,WD
C        FPU,FPY,EPS,XLS
C  * OUTPUT *
C   CONCRETE
C    FCCI= ALLOWABLE COMPRESSIVE STRESS OF GIRDER CONCRETE AT PRESTRESS
C          TRANSFER(PSI)
C    FCTI= ALLOWABLE TENSILE STRESS OF GIRDER CONCRETE AT PRESTRESS
C          TRANSFER(PSI)
C    FCC =ALLOWABLE COMPRESSIVE STRESS OF GIRDER CONCRETE AT SERVICE
C          LOAD(PSI)
C    FCT =ALLOWABLE TENSILE STRESS OF GIRDER CONCRETE AT SERVICE LOAD
C          (PSI)
C    FCR =MODULUS OF RUPTURE OF GIRDER CONCRETE(PSI)
C    EC =MODULUS OF ELASTICITY OF GIRDER CONCRETE(PSI)
C    ECI=MODULUS OF ELASTICITY OF GIRDER CONCRETE AT PRESTRESS TRANSFER
C          (PSI)
C    ECD=MODULUS OF ELASTICITY OF DECK CONCRETE(PSI)
C    XNE=MODULAR RATIO (DECK CONCRETE TO GIRDER CONCRETE)
C   STRAND
C    FPS=ALLOWABLE STRESS OF STRAND AT SERVICE LOAD(PSI)
C    FSE=EFFECTIVE PRESTRESS AT SERVICE LOAD(PSI)
C
C  CALCULATION OF CONCRETE PROPERTIES
      FCCI=0.6*FCPI
      FCTI=0.0
      FCC=0.4*FCP
      FCT=6.0*SQRT(FCP)
      FCR= 7.5*SQRT(FCP)
```

```
      EC=33.0*WC**1.5*SQRT(FCP)
      ECI=33.0*WC**1.5*SQRT(FCPI)
      ECD=33.0*WCD**1.5*SQRT(FCD)
      XNE=ECD/EC
C
      WRITE(IW,290)
  290 FORMAT(1H0,4X,'**(2) CALCULATED MATERIAL PROPERTIES **')
      WRITE(IW,300)
  300 FORMAT(1H0,5X,'CONCRETE')
      WRITE(IW,310)
  310 FORMAT(1H0,7X,'ALLOWABLE GIRDER CONCRETE STRESSES(PSI)')
      WRITE(IW,320) FCCI,FCTI,FCC,FCT
  320 FORMAT(1H0,9X,'AT PRESTRESS TRANSFER',/  13X,
     1 'COMPRESSIVE STRESS     FCCI=',F7.1 ,/  13X,
     2 'TENSILE STRESS         FCTI=',F7.1 ,/  10X,
     3 'AT SERVICE LOAD'                   ,/  13X,
     4 'COMPRESSIVE STRESS     FCC =',F7.1 ,/  13X,
     5 'TENSILE STRESS         FCT =',F7.1)
      WRITE(IW,330) FCR
  330 FORMAT(1H0,7X,'MODULUS OF RUPTURE OF GIRDER CONCRETE(PSI)  FCR=',
     1F7.1)
      WRITE(IW,340)
  340 FORMAT(1H0,7X,'MODULUS OF ELASTICITY(PSI)')
      WRITE(IW,350) EC,ECI,ECD
  350 FORMAT(1H ,10X,'GIRDER CONCRETE                          EC = ',F10.1
     1, /  11X,    'GIRDER CONCRETE AT PRESTRESS TRANSFER  ECI= ',F10.1
     2, /  11X,     'DECK CONCRETE                          ECD= ',F10.1)
      WRITE(IW,360) XNE
  360 FORMAT(1H0,7X,'MODULAR RATIO,DECK TO GIRDER     XNE=',F6.3)
C
C  CALCULATION OF STRAND PROPERTIES
      FPS=0.80*FPY
      FSE=0.7*FPU-XLS
C
      WRITE(IW,370)
  370 FORMAT(1H0,5X,'STRAND')
      WRITE(IW,380) FPS,FSE
  380 FORMAT(1H ,10X,'ALLOWABLE STRESS AT SERVICE LOAD(PSI)      FPS= ',
     1F9.1,/  11X,  'EFFECTIVE PRESTRESS AT SERVICE LOAD(PSI)  FSE=',
     2F9.1)
      IF((0.5*FPU).GT.FSE) GO TO 390
      IF(FSE.GT.FPS) GO TO 390
      GO TO 410
  390 WRITE(IW,400) FSE
  400 FORMAT(1H0,10X,'*** WARNING ***',// 11X
     1'EFFECTIVE PRESTRESS IS OUT OF RANGE   FSE=',F12.1)
      GO TO 1290
  410 CONTINUE
C*************************************************************************
C
C **STEP(3) THICKNESS AND REINFORCEMENT OF DECK **
C  * INPUT *
C      FROM STEP(1)
C         GS,B1
C  * OUTPUT *
C    SE=EFFECTIVE SPAN OF DECK(FT)
C    TD=THICKNESS OF DECK(IN)
C    BS=BAR SPACING(IN) OF NO.5 BAR OR NO.6 BAR
C        BS(1)=NO.5 , BS(2)=NO.6
C
C   SELECTION OF THICKNESS AND REINFORCEMENT FROM A LIST
      SE=GS-B1/12.0+10.0/12.0
      BS(1)=0.
      BS(2)=0.
C    IBAR=INDICATOR OF SIZE OF BAR USED IN DECK
      TD=7.
      IBAR=1
      IF(SE.GT.3.) GO TO 420
      BS(1)=10.
  420 IF(SE.GT.4.) GO TO 430
      BS(1)=8.5
      GO TO 510
  430 IF(SE.GT.5.) GO TO 440
      BS(1)=7.5
      GO TO 510
  440 IF(SE.GT.6.) GO TO 450
      BS(1)=6.5
      GO TO 510
  450 BS(2)=8.
      IBAR=2
```

```
      IF(SE.GT.7.) GO TO 460
      GO TO 510
  460 IF(SE.GT.8.) GO TO 470
      TD=7.5
      GO TO 510
  470 IF(SE.GT.9.) GO TO 480
      TD=8.
      GO TO 510
  480 IF(SE.GT.10.) GO TO 490
      TD=8.5
      GO TO 510
  490 WRITE(IW,500) SE
  500 FORMAT(1H0,10X,'*** WARNING ***',// 11X,
     1      'EFFECTIVE DECK SPAN EXCEEDS 10 FT.    SE=',F12.2)
      GO TO 1290
  510 CONTINUE
C
      WRITE(IW,520)
  520 FORMAT(1H0,4X,'**(3) THICKNESS AND REINFORCEMENT OF DECK **')
      WRITE(IW,530) SE,TD
  530 FORMAT(1H0,10X,'EFFECTIVE SPAN OF DECK(FT)     SE=' , F6.2,/,11X,
     1                 'THICKNESS OF DECK(IN)         TD= , F6.2)
      WRITE(IW,540) ABAR(IBAR),BS(IBAR)
  540 FORMAT(1H ,10X,'BAR SPACING(IN) OF ',A4,' BAR    BS=',F6.2)
C***********************************************************************
C
C **STEP(4) SECTIONAL PROPERTIES OF GIRDER **
C  * INPUT *
C      FROM STEP(1)
C        SL,GS,B1-B4,D1-D6
C      FROM STEP(2)
C        XNE
C      FROM STEP(3)
C        TD
C  * OUTPUT *
C    AG=AREA OF NONCOMPOSITE SECTION(IN2)
C    AC=AREA OF COMPOSITE SECTION(IN2)
C    BE=EFFECTIVE TOP FLANGE(DECK) WIDTH(IN) -COMPOSITE SECTION-
C    YT=DISTANCE FROM CENTROID TO TOP FIBER(IN) -NONCOMPOSITE SECTION-
C    YB=DISTANCE FROM CENTROID TO BOTTOM FIBER(IN)-NONCOMPOSITE SECTION-
C    YTC=YT FOR COMPOSITE SECTION(IN)
C    YBC=YB FOR COMPOSITE SECTION(IN)
C    XIG=MOMENT OF INERTIA OF GIRDER SECTION(IN4) -NONCOMPOSITE SECTION-
C    XIGC=XIG FOR COMPOSITE SECTION(IN4)
C    ST=SECTION MODULUS FOR TOP FIBER(IN3)-NONCOMPOSITE SECTION-
C    SB=SECTION MODULUS FOR BOTTOM FIBER(IN3) -NONCOMPOSITE SECTION-
C    STC=ST FOR COMPOSITE SECTION(IN3)
C    SBC=SB FOR COMPOSITE SECTION(IN3)
C
C   EFFECTIVE TOP FLANGE WIDTH
      X(1)=SL*12./4.
      X(2)=GS*12.
      X(3)=TD*12.+B3
      BE=10000.
      DO 550 I=1,3
      IF(BE.GT.X(I)) BE=X(I)
  550 CONTINUE
C
C   NONCOMPOSITE SECTION
C    A(7)=AREA OF EACH ELEMENT OF SECTION
      A(1)=B1*D2
      A(2)=(2.*B4+B3)*D3
      A(3)=(B1-2.*B4-B3)*D3/2.
      A(4)=2.*B4*D4/2.
      A(5)=B3*(D1-D2-D3-D6)
      A(6)=(B2-B3)*D5/2.
      A(7)=B2*D6
C    Y(7)=DISTANCE FROM BOTTOM FIBER TO CENTROID OF EACH ELEMENT
      Y(1)=D1-D2/2.
      Y(2)=D1-D2-D3/2.
      Y(3)=D1-D2-D3/3.
      Y(4)=D1-D2-D3-D4/3.
      Y(5)=(D1-D2-D3-D6)/2.+D6
      Y(6)=D5/3.+D6
      Y(7)=D6/2.
C    XJ(7)=MOMENT OF INERTIA OF EACH ELEMENT
      XJ(1)=B1*D2**3/12.
      XJ(2)=(2.*B4+B3)*D3**3/12.
      XJ(3)=(B1-2.*B4-B3)*D3**3/36.
      XJ(4)=2.*B4*D4**3/36.
```

```
      XJ(5)=B3*(D1-D2-D3-D6)**3/12.
      XJ(6)=(B2-B3)*D5**3/36.
      XJ(7)=B2*D6**3/12.
      AG=0.
      DO 560 I=1,7
  560 AG=AG+A(I)
      TJ=0.
      DO 570 I=1,7
  570 TJ=TJ+XJ(I)
      YBB=0.
      DO 580 I=1,7
  580 YBB=YBB+A(I)*Y(I)
      YB=YBB/AG
      YT=D1-YB
      XIGG=0.
      DO 590 I=1,7
  590 XIGG=XIGG+A(I)*(Y(I)-YB)**2
      XIG=TJ+XIGG
      ST=XIG/YT
      SB=XIG/YB
C
C    COMPOSITE SECTION
      A(1)=XNE*BE*TD
      A(2)=AG
      Y(1)=D1+TD/2.
      Y(2)=YB
      XJ(1)=XNE*BE*TD**3/12.
      XJ(2)=XIG
      AC=A(1)+A(2)
      YBC=(A(1)*Y(1)+A(2)*Y(2))/AC
      YTC=D1-YBC
      XIGC=XJ(1)+XJ(2)+A(1)*(Y(1)-YBC)**2+A(2)*(Y(2)-YBC)**2
      STC=XIGC/YTC
      SBC=XIGC/YBC
C
      WRITE(IW,600)
  600 FORMAT(1H0,4X,'**(4) SECTIONAL PROPERTIES OF GIRDER **')
      WRITE(IW,610) BE,AG,AC
  610 FORMAT(1H0,10X,'EFFECTIVE TOP FLANGE(DECK) WIDTH(IN)   BE=',F8.2,//
     1    11X,         'AREA OF NONCOMPOSITE SECTION(IN2)      AG=',F8.2,/
     2    11X,         'AREA OF COMPOSITE SECTION(IN2)         AC=',F8.2)
      WRITE(IW,620) YT,YB,YTC,YBC
  620 FORMAT(1H0,10X,'DISTANCE FROM CENTROID TO TOP FIBER(IN)      YT=',
     1F7.2,2X,'-NONCOMPOSITE SECTION-',
     2      / 11X,    'DISTANCE FROM CENTROID TO BOTTOM FIBER(IN)  YB=',
     3F7.2,2X,'-NONCOMPOSITE SECTION-',
     4      / 11X,    'YT FOR COMPOSITE SECTION(IN)               YTC=',
     5F7.2,/ 11X,    'YB FOR COMPOSITE SECTION(IN)               YBC=',
     6F7.2)
      WRITE(IW,630) XIG,XIGC
  630 FORMAT(1H0,10X,'MOMENT OF INERTIA OF NONCOMPOSITE SECTION(IN4)
     1XIG=' ,F12.1,/ 11X,'MOMENT OF INERTIA OF COMPOSITE SECTION(IN4)
     2   XIGC=',F12.1)
      WRITE(IW,640) ST,SB,STC,SBC
  640 FORMAT(1H0,10X,'SECTION MODULUS FOR TOP FIBER(IN3)      ST=',F10.1,
     1 2X, '-NONCOMPOSITE SECTION-',/
     2              11X,'SECTION MODULUS FOR BOTTOM FIBER(IN3)   SB=',F10.1,
     3 2X,'-NONCOMPOSITE SECTION-',/
     4              11X,'ST FOR COMPOSITE SECTION(IN3)          STC=',F10.1,
     5           /,11X,'SB FOR COMPOSITE SECTION(IN3)           SBC=',F10.1)
C*********************************************************************
C
C **STEP(5) DESIGN LOADS AND MOMENTS **
C  * INPUT *
C      FROM STEP(1)
C        SL,GS,WG,WD
C      FROM STEP(3)
C        TD
C      FROM STEP(4)
C        AG
C  * OUTPUT *
C     XMD=MOMENT DUE TO DECK PLUS GIRDER WEIGHT(FT-KIP)
C     XMDG=MOMENT DUE TO GIRDER WEIGHT(FT-KIP)
C     XML=MOMENT DUE TO LIVE LOAD(FT-KIP)
C     XMU=FACTORED MOMENT(FT-KIP)
C     XIP=IMPACT LOAD COEFFICIENT
C     WUG=UNIFORMLY DISTRIBUTED LOAD DUE TO GIRDER WEIGHT(KIP/FT)
C     WUD=UNIFORMLY DISTRIBUTED LOAD DUE TO DECK WEIGHT(KIP/FT)
C
C     DM(12)=HS 20-44 MOMENTS FOR 10FT.WIDE LANE (FT-K)
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C     SLL(12)=SPANS CORRESPONDING TO HS 20-44 MOMENTS (FT)
C
C   CALCULATION OF DESIGN LOADS AND MOMENTS
      XIP=50./(SL+125.)
      IF(XIP.GT.0.3) XIP=0.3
      WUG=WG*AG/(12.**2)/1000.
      WUD=WD*GS*TD/12./1000.
      XMDG=WUG*SL**2/8.
      XMD=XMDG+WUD*SL**2/8.
      IF(SL.GT.SLL(1)) GO TO 650
      IF(SL.LT.SLL(1)) GO TO 660
      XXM=DM(1)
      GO TO 680
  650 CONTINUE
      DO 660 I=2,12
      IF(SL.GT,SLL(I)) GO TO 660
      XXM=(SL-SLL(I-1))/(SLL(I)-SLL(I-1))*(DM(I)-DM(I-1))+DM(I-1)
      GO TO 680
  660 CONTINUE
      WRITE(IW,670) SL
  670 FORMAT(1H0,10X,'*** WARNING ***',// 11X,
     1    'GIRDER SPAN IS OUT OF RANGE    SL=',F12.2)
      GO TO 1290
  680 CONTINUE
      XML=GS/10.*XXM
      XMU=1.3*(XMD+1.67*XML*(1.+XIP))
C
      WRITE(IW,690)
  690 FORMAT(1H0,4X,'**(5) DESIGN LOADS AND MOMENTS **')
      WRITE(IW,700) XIP,WUG,WUD
  700 FORMAT(1H0,10X,'IMPACT LOAD FACTOR',30X,'XIP=',F6.3,/ 11X,
     1'UNIFORM LOAD DUE TO GIRDER WEIGHT(KIP/FT)', 7X,'WUG=',F7.3,/,11X,
     2'UNIFORM LOAD DUE TO DECK WEIGHT(KIP/FT)   ', 7X,'WUD=',F7.3)
      WRITE(IW,710) XMDG,XMD,XML,XMU
  710 FORMAT(1H0,10X,'MOMENT DUE TO GIRDER WEIGHT(FT-KIP)                XMD
     1G=',F8.2,/ 11X,'MOMENT DUE TO DECK PLUS GIRDER WEIGHT(FT-KIP)    XM
     2D=',F8.2,/ 11X,'MOMENT DUE TO LIVE LOAD(FT-KIP)                  XM
     3L=',F8.2,/ 11X,'FACTORED MOMENT(FT-KIP)                          XM
     4U=',F8.2)
C*********************************************************************
C
C **STEP(6) REQUIRED NUMBER OF STRANDS **
C
C **SUBSTEP(6-A) ALLOWABLE STRESSES CHECK **
C  * INPUT *
C       FROM STEP(1)
C         FPU,B2,B3,D5,D6,ASTD,CTC,CSC,SWW
C       FROM STEP(2)
C         FCCI,FCTI,FCC,FCT,FSE
C       FROM STEP(4)
C         AG,ST,SB,STC,SBC,YB
C       FROM STEP(5)
C         XMD,XMDG,XML,XIP
C  * OUTPUT *
C     NS=NO. OF STRANDS REQUIRED
C     ET=DISTANCE FROM CENTROID OF STRANDS TO CENTROID OF GIRDER SECTION
C          -NONCOMPOSITE-(IN)
C     AS=TOTAL AREA OF STRANDS REQUIRED(IN2)
C     FI=TOTAL INITIAL PRESTRESSING FORCE(KIP)
C     FS=TOTAL PRESTRESSING FORCE AT SERVICE LOAD(KIP)
C     CE=DISTANCE FROM CENTROID OF STRANDS TO BOTTOM FIBER(IN)
C     NR(I) =MAX.NO.OF STRANDS PLACED IN I-TH ROW (MAX.I=50)
C              THE FIRST ROW IS LOCATED NEXT TO THE BOTTOM SURFACE
C
C   STRAND ARRANGEMENT
      XA=(SWW-1.)*CTC/2.
      XB=B2/2.-CSC
      DO 720 I=1,50
  720 NR(I)=0
      A1=-D5/(B2/2.-B3/2.)
      B=-A1/2.*B2+D6-CSC*SQRT((B2/2.-B3/2.)**2+D5**2)/(B2/2.-B3/2.)
C     I=NO.OF ROWS
      I=0
      YBM = SB/AG+YB
  730 I=I+1
      YI=I
      Y1=CTC*(YI-1.)+CSC
      IF(Y1.GT.YBM) GO TO 760
      IF(I.GE.50) GO TO 760
C     J=NO.OF COLUMNS
```

```
      J=0
  740 J=J+1
      X2=J-1
      X1=CTC*X2/2.
      IF(X1.LE.XA) GO TO 740
      IF(Y1.GT.(A1*X1+B)) GO TO 750
      IF(X1.GT.XB) GO TO 750
      GO TO 740
  750 NR(I)=J-1
      GO TO 730
  760 CONTINUE
C
      WRITE(IW,770)
  770 FORMAT(1H0, 4X,'**(6) REQUIRED AMOUNT OF STRAND **')
C
      NS=0
C+++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
C   ITERATION START (THE NO. OF STRANDS IS INCREASED ONE BY ONE)
  780 CONTINUE
      NS=NS+1
      IF (NS.GT.100) GO TO 810
      XNS=NS
      AS=ASTD*XNS
      A2=0.
      AY=0.
      I=0
      NRT=0
  790 I=I+1
      IF(NR(I).EQ.0) GO TO 810
      XI=I
      XN=NS-NRT
      NRT=NRT+NR(I)
      IF(NS.GT.NRT) GO TO 800
      A2=A2+ASTD*XN
      AY=AY+XN*ASTD*(CSC+CTC*(XI-1.))
      CE=AY/A2
      GO TO 840
  800 XNR=NR(I)
      A2=A2+XNR*ASTD
      AY=AY+XNR*ASTD*(CSC+CTC*(XI-1.))
      GO TO 790
  810 WRITE(IW,820)
  820 FORMAT(1H0,10X,'*** WARNING ***',// 11X,
     1   'CANNOT SATISFY STRESS AND STRENGTH REQUIREMENTS')
      WRITE(IW,830) NS
  830 FORMAT(1H0,10X,'NO. OF STRANDS=',I4)
      GO TO 1290
  840 CONTINUE
      ET=YB-CE
      FI=0.7*AS*FPU/1000.
      FS=FSE*AS/1000.
C
C   CHECK INITIAL STRESSES
C        COMPRESSION=POSITIVE,  TENSION=NEGATIVE
C    TOP FIBER STRESS
      Z1=-FCTI/1000.
      Z2=FCCI/1000.
      SIT=FI/AG-FI*ET/ST+XMDG*12./ST
      IF(Z1.GT.SIT) GO TO 780
      IF(Z2.LT.SIT) GO TO 780
C    BOTTOM FIBER STRESSES
      SIB=FI/AG+FI*ET/SB-XMDG*12./SB
      IF(Z1.GT.SIB) GO TO 780
      IF(Z2.LT.SIB) GO TO 780
C
C   CHECK SERVICE LOAD STRESSES
C    TOP FIBER STRESSES
      Z1=-FCT/1000.
      Z2=FCC/1000.
      SST=FS/AG-FS*ET/ST+XMD*12./ST+XML*12./STC*(1.+XIF)
      IF(Z1.GT.SST) GO TO 780
      IF(Z2.LT.SST) GO TO 780
C    BOTTOM FIBER STRESSES
      SSB=FS/AG+FS*ET/SB-XMD*12./SB-XML*12./SBC*(1.+XIF)
      IF(Z1.GT.SSB) GO TO 780
      IF(Z2.LT.SSB) GO TO 780
C************************************************************************
C
C **SUBSTEP(6-B) ULTIMATE POSITIVE MOMENT **
C  * INPUT *
```

```
C       FROM STEP(1)
C         B1,B2,B3,B4,D1,D2,D3,D4,D5,D6,FPU,FCD,FCP,EPS,FPY,FPM,SY,SM,SU
C       FROM STEP(2)
C         FSE,FCR
C       FROM STEP(3)
C         TD
C       FROM STEP(4)
C         BE,AG,SB,SBC
C       FROM STEP(5)
C         XMU,XMD
C       FROM SUBSTEP(6-A)
C         AS,CE,FS,ET
C   * OUTPUT *
C     XMCR=CRACKING MOMENT(FT-KIP)
C     XMN=FLEXURAL DESIGN STRENGTH OF COMPOSITE SECTION(FT-KIP)
C     AFSU=AVERAGE STRESS IN STRANDS AT ULTIMATE MOMENT(PSI)
C     DE=DISTANCE FROM EXTREME COMPRESSION FIBER(COMPOSITE SECTION) TO
C        CENTROID OF THE STRANDS(IN)
C     PA=STRAND RATIO
C     EPO=EFFECTIVE STRAIN OF THE STRAND DUE TO PRESTRESS ONLY AT SERVICE
C        LOAD CONDITION
C     EP1=STRAIN OF THE STRANDS AT ULTIMATE MOMENT EXCLUDING EPO
C     EP2=TOTAL STRAIN OF THE STRANDS AT ULTIMATE MOMENT
C     BP= EQUIVALENT WIDTH OF WEB(IN)
C     BET1=RATIO OF THE DEPTH OF COMPRESSION ZONE TO THE DISTANCE FROM
C          EXTREME COMPRESSION FIBER TO NUETRAL AXIS
C     CD=DISTANCE FROM EXTREME COMPRESSION FIBER TO NEUTRAL AXIS(IN)
C     RIX=REINFORCEMENT INDEX
C     CDI=INCREMENT OF CD(IN)
C     FCE=EQUIVALENT SPECIFIED CONCRETE STRENGTH - WEIGHT AVERAGE OF
C         GIRDER CONCRETE STRENGTH AND DECK CONCRETE STRENGTH(PSI)
C     CBC=DISTANCE FROM EXTREME COMPRESSION FIBER TO THE CENTROID OF
C         COMPRESSION STRESS BLOCK (IN)
C     AD=DEPTH OF EQUIVALENT RECTANGULAR COMPRESSION STRESS BLOCK(IN)
C     TF=TENSION FORCE IN STRANDS(LBS)
C     CF=COMPRESSION FORCE IN COMPRESSION STRESS BLOCK(LBS)
C
C  PRECALCULATED VARIABLES USED IN SUBSTEP(6-B)
      G1=BE*TD
      G2=TD+D2
      G3=TD+D2+D3
      G4=TD+D2+D3+D4
      G5=B1*D2
      G6=(B1+2.*B4+B3)*D3/2.
      G7=(2.*B3+2.*B4)*D4/2.
      G8=0.85*FCD*TD*BE
      G9=0.85*FCP*D2*B1
      G10=0.85*FCP*(B1+2.*B4+B3)*D3/2.
      G11=0.85*FCP*(2.*B3+2.*B4)*D4/2.
      G12=TD/2.
      G13=D2/2.+TD
      G14=D3*(B1+4.*B4+2.*B3)/3./(B1+2.*B4+B3)+TD+D2
      G15=D4*(2.*B4+B3+2.*B3)/3./(2.*B4+B3+B3)+TD+D2+D3
C
C  ITERATION PROCESS
      DE=TD+D1-CE
      EPO=FSE/EPS
C   INITIAL VALUES OF CD AND CDI
      CD=0.
      CDI=1.
C   IPI=POINTER FOR ITERATION NO(=0, THE FIRST ITERATION,=1, THE SECOND)
      IPI=0
C-----------------------------------------------------------------------
C
C   START OF LOOP  (ULTIMATE MOMENT)
  850 CONTINUE
      CD=CD+CDI
C   CALCULATE AFSU AND TF
      EP1=0.003*(DE-CD)/CD
      EP2=EP1+EPO
      IF(EP2.GT.SY) GO TO 860
      AFSU=FPY*EP2/SY
      GO TO 890
  860 IF(EP2.GT.SM) GO TO 870
      AFSU=FPY+(EP2-SY)/(SM-SY)*(FPM-FPY)
      GO TO 890
  870 IF(EP2.GT.SU) GO TO 880
      AFSU=FPM+(EP2-SM)/(SU-SM)*(FPU-FPM)
      GO TO 890
  880 AFSU=FPU
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```
  890 TF=AS*AFSU
C     CALCULATE FCE,BET1 AND AD
C      ADT=TENTATIVE VALUE FOR AD(IN2)
C      ADD=DECK CONCRETE PORTION OF STRESS BLOCK AREA(IN2)
C      AGG=GIRDER CONCRETE PORTION OF STRESS BLOCK AREA(IN2)
      ADD=G1
      ADT=0.85*CD
      IF(ADT.GT.TD) GO TO 900
      AGG=0.
      GO TO 940
  900 IF(ADT.GT.G2) GO TO 910
      AGG=(ADT-TD)*B1
      GO TO 940
  910 IF(ADT.GT.G3) GO TO 920
      Y1=G3-ADT
      Y3=D3
      X3=(B1-(2.*B4+B3))/2.
      X1=X3*Y1/Y3
      Y2=D3-Y1
      AGG=G5+(B1+2.*X1+B3+2.*B4)*Y2/2.
      GO TO 940
  920 IF(ADT.GT.G4) GO TO 930
      X3=B4
      Y3=D4
      Y1=G4-ADT
      X1=X3*Y1/Y3
      Y2=D4-Y1
      AGG=G5+G6+(2.*B4+B3+2.*X1+B3)*Y2/2.
      GO TO 940
  930 AGG=G5+G6+G7+B3*(ADT-G4)
  940 CONTINUE
      FCE=(ADD*FCD+AGG*FCP)/(ADD+AGG)
      BET1=0.85-0.05*(FCE-4000.)/1000.
      IF(BET1.LT.0.65) BET1=0.65
      AD=BET1*CD
C     CALCULATE CF,CBC AND BP
      IF(AD.GT.TD) GO TO 950
      CF=0.85*FCD*AD*BE
      CBC=AD/2.
      BP=BE
      GO TO 990
  950 IF(AD.GT.G2) GO TO 960
      CF1=G8
      CF2=0.85*FCP*(AD-TD)*B1
      CB1=G12
      CB2=(AD-TD)/2.+TD
      CF=CF1+CF2
      CBC=(CF1*CB1+CF2*CB2)/CF
      BP=B1
      GO TO 990
  960 IF(AD.GT.G3) GO TO 970
      Y3=D3
      Y1=G3-AD
      X3=(B1-(2.*B4+B3))/2.
      X1=X3*Y1/Y3
      Y2=D3-Y1
      XX=2.*X1+B3+2.*B4
      CF1=G8
      CF2=G9
      CF3=0.85*FCP*(B1+XX)*Y2/2.
      CB1=G12
      CB2=G13
      CB3=Y2*(B1+2.*XX)/3./(B1+XX)+G2
      CF=CF1+CF2+CF3
      CBC=(CF1*CB1+CF2*CB2+CF3*CB3)/CF
      BP=XX
      GO TO 990
  970 IF(AD.GT.G4) GO TO 980
      X3=B4
      Y3=D4
      Y1=G4-AD
      Y2=D4-Y1
      X1=X3*Y1/Y3
      XX=2.*X1+B3
      CF1=G8
      CF2=G9
      CF3=G10
      CF4=0.85*FCP*(2.*B4+B3+2.*X1+B3)*Y2/2.
      CB1=G12
      CB2=G13
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      CB3=G14
      CB4=Y2*(2.*B4+B3+2.*XX)/3./(2.*B4+B3+XX)+G2
      CF=CF1+CF2+CF3+CF4
      CBC=(CF1*CB1+CF2*CB2+CF3*CB3+CF4*CB4)/CF
      BP=XX
      GO TO 990
  980 CONTINUE
      CF1=G8
      CF2=G9
      CF3=G10
      CF4=G11
      CF5=0.85*FCP*(AD-G4)*B3
      CB1=G12
      CB2=G13
      CB3=G14
      CB4=G15
      CB5=(AD-G4)/2.+G4
      CF=CF1+CF2+CF3+CF4+CF5
      CBC=(CF1*CB1+CF2*CB2+CF3*CB3+CF4*CB4+CF5*CB5)/CF
      BP=B3
  990 CONTINUE
      IF(TF.GT.CF) GO TO 850
C    CHECK WHETHER THE FIRST ITERATION OR THE SECOND
      IF(IPI.EQ.1) GO TO 1000
      IPI=1
      CD=CD-CDI
      CDI=0.1
      GO TO 850
C
C    END OF LOOP (ULTIMATE MOMENT)
C
C---------------------------------------------------------------------
 1000 CONTINUE
C    AFTER THE SECOND ITERATION
      XMNS=(CF+TF)/2.*(DE-CBC)/12./1000.
      XMN = .9*XMNS
      IF(XMN.LT.XMU) GO TO 780
C     CALCULATE CRACKING MOMENT
      XMCR=SBC/12.*(FS/AG+FS*ET/SB-XMD*12./SB+FCR/1000.)
      IF(XMNS.LT.(1.2*XMCR)) GO TO 780
C
C    END OF ITERATION (THE NO. OF STRANDS)
C
C+++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
C    CHECK MAX. REINFORCEMENT INDEX
C     CALCULATE RIX
      IF(AD.GT.TD) GO TO 1010
      FCE=FCD
      PA=AS/BP/DE
      GO TO 1020
 1010 CONTINUE
      PA=AD*0.85*FCE/AFSU/DE
 1020 RIX=PA*AFSU/FCE
      IF(RIX.LE.0.3) GO TO 1040
      WRITE(IW,1030) RIX
 1030 FORMAT(1H0,10X,'*** WARNING ***',// 11X,
     1 'REINFORCEMENT INDEX EXCEEDS 0.3    RIX= ',F6.3)

      GO TO 1290
 1040 CONTINUE
C
      WRITE(IW,1050) NS
 1050 FORMAT(1H0,10X,'NO.OF STRANDS REQUIRED     NS= ',I3)
      WRITE(IW,1060)
 1060 FORMAT(1H0,12X,'ROW',10X,'NO.OF STRANDS PER ROW',/)
      NRR=0
C
      DO 1100 I=1,50
      NRT=NRR+NR(I)
      IF(NS.GT.NRT) GO TO 1080
      NNN=NS-NRR
      WRITE(IW,1070) I,NNN
 1070 FORMAT(1H ,12X,I3,17X,I3)
      GO TO 1110
 1080 WRITE(IW,1090) I,NR(I)
 1090 FORMAT(1H ,12X,I3,17X,I3)
      NRR=NRT
 1100 CONTINUE
C
 1110 CONTINUE
```

```
      WRITE(IW,1120) AS
 1120 FORMAT(1H0,10X,'TOTAL AREA OF STRANDS REQUIRED(IN2)  AS=',F8.3)
      SIT = SIT*1000.
      SIB = SIB*1000.
      SST = SST*1000.
      SSB = SSB*1000.
      WRITE(IW,1130) FI,SIT,SIB,FS,SST,SSB
 1130 FORMAT(1H0,10X,'INITIAL PRESTRESSING FORCE(KIP)               FI=',F8.3
     1 ,/ 12X,         'TOP FIBER STRESS(PSI)        SIT=',F9.1
     2 ,/ 12X,         'BOTTOM FIBER STRESS(PSI)     SIB=',F9.1
     3 ,/ 11X,         'PRESTRESSING FORCE AT SERVICE LOAD(KIP)  FS=',F8.3
     4 ,/ 12X,         'TOP FIBER STRESS(PSI)        SST=',F9.1
     5 ,/ 12X,         'BOTTOM FIBER STRESS(PSI)     SSB=',F9.1)
      WRITE(IW,1140) EP0,EP2
 1140 FORMAT(1H0,10X,'STRAINS AT CENTROID OF STRANDS',// 12X,
     1 'STRAIN DUE TO PRESTRESS ONLY AT SERVICE LOAD    EP0=',F8.5,/ 12X
     2,'TOTAL STRAIN AT ULTIMATE MOMENT                  EP2=',F8.5)
      WRITE(IW,1150)
 1150 FORMAT(1H0,10X,'GEOMETRIC PARAMETERS OF GIRDER SECTION')
      WRITE(IW,1160) ET,CE
 1160 FORMAT(1H0,11X,'DISTANCE FROM CENTROID OF STRANDS TO CENTROID OF G
     1IRDER SECTION-NONCOMPOSITE-(IN)    ET=',F7.2,/ 12X,
     2'DISTANCE FROM CENTROID OF STRANDS TO BOTTOM FIBER(IN)',31X,'CE=',
     3F7.2)
      WRITE(IW,1170) DE,CD,BET1,AD,CBC,BP
 1170 FORMAT(1H ,11X,'DISTANCE FROM TOP FIBER(COMPOSITE SECTION) TO CENT
     1ROID OF STRANDS(IN)',15X,'DE=',F7.2,/ 12X,
     2'DISTANCE FROM TOP FIBER(COMPOSITE SECTION) TO NEUTRAL AXIS(IN)',
     322X,'CD=',F7.2,/ 12X,'RATIO OF STRESS BLOCK DEPTH TO CD',49X,
     4'BET1=',F6.3,/ 12X,'COMPRESSION STRESS BLOCK DEPTH(IN)',50X,'AD='
     5,F7.2, / 12X,'DISTANCE FROM TOP FIBER(COMPOSITE SECT.)  TO CENTROI
     6D OF STRESS BLOCK(IN)',10X,'CBC=',F7.2, / 12X,'EQUIVALENT WIDTH OF
     7 WEB(COMPOSITE SECTION)(IN)',38X,'BP=',F7.2)
      WRITE(IW,1180) AFSU,FCE,PA,RIX
 1180 FORMAT(1H0,10X,'AVERAGE STRESS IN STRANDS AT ULTIMATE MOMENT(PSI)
     1  AFSU=',F9.1,/ 11X,'WEIGHT AVERAGED CONCRETE STRENGTH(PSI)',15X,
     2'FCE=',F7.1, / 11X,'STRAND RATIO',42X,'PA=',F9.6, / 11X,'REINFORC
     3EMENT INDEX',34X,'RIX=',F6.3)
      WRITE(IW,1190) TF,CF,XMN,XMCR
 1190 FORMAT(1H0,10X,'TENSILE FORCE IN STRANDS(LBS)              TF=',F10.1
     1,/ 11X,         'COMPRESSIVE FORCE IN STRESS BLOCK(LBS)  CF=',F10.1
     2,/ 11X,       'FLEXURAL DESIGN STRENGTH OF GIRDER(COMPOSITE SECTIO
     3N(FT-KIP)    XMN=',F8.1,/ 11X,'CRACKING MOMENT OF GIRDER(FT-KIP)',
     429X,'XMCR=',F8.1)
C*********************************************************************
C
C **STEP(7) DEFLECTIONS AT MIDSPAN **
C   SEE P.I/1.101 OF STATE OF ILLINOIS DESIGN MANUAL
C  * INPUT *
C      FROM STEP(1)
C        SRF,SL
C      FROM STEP(2)
C        EC,ECI
C      FROM STEP(4)
C        XIG
C      FROM STEP(5)
C        WUG,WUD
C      FROM SUBSTEP(6-A)
C        FI,ET
C  *OUTPUT *
C    DUP=UPWARD DEFLECTION DUE TO PRESTRESSING(STRAIGHT STRANDS)(INCH)
C    DDG=DOWNWARD DEFLECTION DUE TO GIRDER WEIGHT(IN)
C    CAMB=RESULTANT CAMBER AT ERECTION(IN)
C    DDD=DOWNWARD DEFLECTION DUE TO DECK WEIGHT(IN)
C
C   CALCULATION OF DEFLECTIONS
      DUP=0.125*FI*ET*SL**2*12**2*1000./SRF/ECI/XIG
      DDG=5.*WUG*SL**4*12**3*1000./SRF/384./ECI/XIG
      CAMB=DUP-DDG
      DDD=5.*WUD*SL**4*12**3*1000./384./EC/XIG
C
      WRITE(IW,1200)
 1200 FORMAT(1H0,4X,'**(7) DEFLECTIONS AT MIDSPAN **')
      WRITE(IW,1210) DUP,DDG,CAMB,DDD
 1210 FORMAT(1H0,10X,'UPWARD DEFLECTION DUE TO PRESTRESSING FORCE(STRAIG
     1HT STRANDS)(IN)     DUP=',F7.3,/11X,'DOWNWARD DEFLECTION DUE TO GIR
     2DER WEIGHT(IN)',25X,'DDG=',F7.3,/11X,'RESULTANT CAMBER AT ERECTION
     3(IN)',36X,'CAMB=',F7.3,/11X,'DOWNWARD DEFLECTION DUE TO DECK WEIGH
     4T(IN)',27X,'DDD=',F7.3)
C*********************************************************************
```

```
C
C **STEP(8) COST PER UNIT SURFACE AREA OF BRIDGE **
C  * INPUT *
C      FROM STEP(1)
C        SL,GS,WC,WCD,RUCG,RUCD,RUS,RUR,RUE,UST,UR5,UR6
C      FROM STEP(3)
C        TD,BS
C      FROM STEP(4)
C        AG
C      FROM SUBSTEP(6-A)
C        NS
C  *OUTPUT *
C    TWS=TOTAL WEIGHT OF STRANDS PER GIRDER(LBS)
C    TWR=TOTAL WEIGHT OF DECK REINFORCEMENT PER GIRDER(LBS)
C    TWCG=TOTAL WEIGHT OF GIRDER CONCRETE PER GIRDER(LBS)
C    TWCD=TOTAL WEIGHT OF DECK CONCRETE PER GIRDER(LBS)
C    WUS=STRAND WEIGHT PER UNIT SURFACE AREA(LBS/FT2)
C    WUR=DECK REINFORCEMENT WEIGHT PER UNIT SURFACE AREA(LBS/FT2)
C    WUCG=GIRDER CONCRETE WEIGHT PER UNIT SURFACE AREA(LBS/FT2)
C    WUCD=DECK CONCRETE WEIGHT PER UNIT SURFACE AREA(LBS/FT2)
C    CUS=COST OF STRAND PER UNIT SURFACE AREA(UNIT/FT2)
C    CUR=COST OF DECK REINFORCEMENT PER UNIT SURFACE AREA(UNIT/FT2)
C    CUCG=COST OF GIRDER CONCRETE PER UNIT SURFACE AREA(UNIT/FT2)
C    CUCD=COST OF DECK CONCRETE PER UNIT SURFACE AREA(UNIT/FT2
C    TUT=TOTAL COST INDEX PER UNIT SURFACE AREA BASED ON RELATIVE
C        UNIT COSTS(UNIT/FT2)
C
C   COST CALCULATION
      XNS=NS
      TWS=XNS*SL*UST
      IF(IBAR.EQ.2) GO TO 1220
      TWR=SL*12.*GS*2.*UR5/BS(IBAR)
      GO TO 1230
 1220 TWR=SL*12.*GS*2.*UR6/BS(IBAR)
 1230 CONTINUE
      TWCG=AG*SL*WC/12**2
      TWCD=TD*GS*SL*WCD/12.
      WUS=TWS/SL/GS
      WUR=TWR/SL/GS
      WUCG=TWCG/SL/GS
      WUCD=TWCD/SL/GS
      CUS=RUS*WUS
      CUR=(RUR+RUE)/2.*WUR
      CUCG=RUCG*WUCG
      CUCD=RUCD*WUCD
      TUT=CUS+CUR+CUCG+CUCD
C
      WRITE(IW,1240)
 1240 FORMAT(1H0,4X,'**(8) COST INDEX PER UNIT SURFACE AREA OF BRIDGE **
     1 ')
      WRITE(IW,1250) TWS,TWR,TWCG,TWCD
 1250 FORMAT(1H0,10X,'TOTAL WEIGHT OF EACH MATERIAL PER GIRDER(LBS)',
     1  // 13X,'STRAND                     TWS=',F10.1
     2  / 13X,'DECK REINFORCEMENT         TWR=',F10.1
     3  / 13X,'GIRDER CONCRETE           TWCG=',F10.1
     4  / 13X,'DECK CONCRETE             TWCD=',F10.1)
      WRITE(IW,1260) WUS,WUR,WUCG,WUCD
 1260 FORMAT(1H0,10X,'WEIGHT OF EACH MATERIAL PER UNIT SURFACE AREA(LBS/
     1FT2)',
     2  // 13X,'STRAND                     WUS=',F10.3,
     3  / 13X,'DECK REINFORCEMENT         WUR=',F10.3,
     4  / 13X,'GIRDER CONCRETE           WUCG=',F10.3
     5  / 13X,'DECK CONCRETE             WUCD=',F10.3)
      WRITE(IW,1270) CUS,CUR,CUCG,CUCD
 1270 FORMAT(1H0,10X,'COST INDEX OF EACH MATERIAL PER UNIT SURFACE AREA(
     1UNIT/FT2)',
     2  // 13X,'STRAND                     CUS=',F10.2,
     3  / 13X,'DECK REINFORCEMENT         CUR=',F10.2,
     4  / 13X,'GIRDER CONCRETE           CUCG=',F10.2,
     5  / 13X,'DECK CONCRETE             CUCD=',F10.2)
      WRITE(IW,1280) TUT
 1280 FORMAT(1H0,10X,'TOTAL COST INDEX PER UNIT SURFACE AREA BASED ON RE
     1LATIVE UNIT COSTS(UNIT/FT2)',// 50X,'TUT=',F10.2)
 1290 READ(IR,20) (TITLE(J),J=1,20)
      IF (TITLE(1).EQ.CONT) GO TO 10
      STOP
      END
```

Sample Problems

Data input and information output for two sample problems is presented in the following pages. Example 1 is the design of an AASHTO Type VI girder, while Example 2 is for a Washington Series 14 girder. Both examples are for girders with a span of 130 ft (39.6 m) and spacing of 6.0 ft (1.83 m).

Data input on punched cards is reproduced in Table 10. These data were prepared according to "User's Input Instructions" discussed earlier in this appendix, and summarized in Table 9. In Example 1, default options, where available, were selected. For this reason cards 7 through 11 are blank cards. A sample of data output for both examples is given in the next pages. This output is self-explanatory.

TABLE 10 - SAMPLE OF DATA INPUT

Example	Data Punched on Cards						Card No.
1	SAMPLE PROBLEM 1						CARD 1
	AASHTO TYPE VI						CARD 2
	MARCH 11,1981						CARD 3
	130.	6.					CARD 4
	42.	28.	8.	4.			CARD 5
	72.	5.	3.	4.	10.	8.	CARD 6
							CARD 7
							CARD 8
							CARD 9
							CARD 10
							CARD 11
	CONTINUE						CARD 12
2	SAMPLE PROBLEM 2						CARD 1
	WASHINGTON SERIES 14						CARD 2
	MARCH 11,1981						CARD 3
	130.	6.0					CARD 4
	42.0	14.0	5.0	2.0			CARD 5
	73.5	2.875	2.625	2.0	3.0	6.0	CARD 6
	2.0	2.0	2.0				CARD 7
	6000.0	4500.0	145.0	145.0	150.0	150.0	CARD 8
	280000.0	28000000.0	45000.0				CARD 9
	0.55						CARD 10
	1.0	1.0	8.0	9.0	12.0		CARD 11
	END						CARD 12

Sample of Data Output for Example 1

```
**SAMPLE PROBLEM 1                                                                           CARD  1 **
**AASHTO TYPE VI                                                                             CARD  2 **
**MARCH 11,1981                                                                              CARD  3 **

**(1) INPUT DATA **

 GEOMETRIC PROPERTIES

      GIRDER SPAN LENGTH(FT)    SL=     130.00
      GIRDER SPACING(FT)        GS=       6.00

      HORIZONTAL DIMENSIONS OF GIRDER CROSS SECTION
       B1=     42.00  B2=     28.00  B3=      8.00  B4=       4.00

      VERTICAL DIMENSIONS OF GIRDER CROSS SECTION
       D1=     72.00  D2=      5.00  D3=      3.00  D4=       4.00  D5=     10.00  D6=      5.00

      CENTER TO CENTER SPACING OF STRANDS(IN        CTC=  2.00
      CONCRETE SURFACE TO CENTER OF STRANDS(IN)     CSL=  2.00
      NUMBER OF STRANDS WITHIN WEB WIDTH            SWW=  2.00

 MATERIAL PROPERTIES    (*=INTERNALLY ASSIGNED VALUES)

   CONCRETE

      SPECIFIED GIRDER CONCRETE STRENGTH(PSI)       FCP = 6000.0
      CONCRETE STRENGTH AT PRESTRESS TRANSFER(PSI)  FCPI= 4500.0
      SPECIFIED DECK CONCRETE STRENGTH(PSI)         FCD = 4000.0*

      UNIT WEIGHT OF GIRDER CONCRETE(PCF)           WC = 145.0
      UNIT WEIGHT OF DECK CONCRETE(PCF)             WCD= 145.0
      UNIT WEIGHT OF GIRDER(PCF)                    WG = 150.0
      UNIT WEIGHT OF DECK(PCF)                      WD = 150.0

   STRAND

      STRESS-STRAIN CURVE OF STRAND
                                        STRESS(PSI)       STRAIN
      SPECIFIED ULTIMATE STRENGTH FPU= 270000.0*  SU= 0.0400*
      INTERMEDIATE STRESS         FPM= 255000.0*  SM= 0.0120*
      SPECIFIED YIELD STRESS      FPY= 230000.0   SY= 0.0082

      MODULUS OF ELASTICITY OF STRAND(PSI)    EPS= 28000000.0
      TOTAL PRESTRESS LOSSES (PSI)            XLS=    45000.0

 GIRDER STIFFNESS REDUCTION FACTOR           SRF= 0.550

 RELATIVE COST INDEX(UNIT/LB)
      GIRDER CONCRETE                   PUCG=  1.00
      DECK CONCRETE                     PUCD=  1.00
      STRAND                            PUS =  8.00
      DECK REINFORCEMENT                RUR =  9.00
      EPOXY COATED DECK REINFORCEMENT   RUE = 12.00

**(2) CALCULATED MATERIAL PROPERTIES **



 CONCRETE

   ALLOWABLE GIRDER CONCRETE STRESSES(PSI)

     AT PRESTRESS TRANSFER
        COMPRESSIVE STRESS       FCCI= 2700.0
        TENSILE STRESS           FCTI=    0.0
     AT SERVICE LOAD
        COMPRESSIVE STRESS       FCC = 2400.0
        TENSILE STRESS           FCT =  464.7

   MODULUS OF RUPTURE OF GIRDER CONCRETE PSI)   FCR=  580.9
```

```
  MODULUS OF ELASTICITY(PSI
     GIRDER CONCRETE                                    EC = 4463146.0
     GIRDER CONCRETE AT PRESTRESS TRANSFER              ECI= 3865196.0
     DECK CONCRETE                                      ECD= 3644143.5

  MODULAR RATIO,DECK TO GIRDER     XNE= 0.816

 STRAND
     ALLOWABLE STRESS AT SERVICE LOAD(PSI)      FPS= 184000.0
     EFFECTIVE PRESTRESS AT SERVICE LOAD(PSI)   FSE= 144000.0

**(3) THICKNESS AND REINFORCEMENT OF DECK **

     EFFECTIVE SPAN OF DECK(FT)     SE=  5.33
     THICKNESS OF DECK(IN)          TD=  7.00
     BAR SPACING(IN) OF NO.5 BAR    BS=  8.50

**(4) SECTIONAL PROPERTIES OF GIRDER **

     EFFECTIVE TOP FLANGE(DECK) WIDTH(IN)  BE=   72.00

     AREA OF NONCOMPOSITE SECTION(IN2)      AG= 1095.00
     AREA OF COMPOSITE SECTION(IN2)         AC= 1496.51

     DISTANCE FROM CENTROID TO TOP FIBER(IN)        YT=  35.61  -NONCOMPOSITE SECTION-
     DISTANCE FROM CENTROID TO BOTTOM FIBER(IN)     YB=  36.38  -NONCOMPOSITE SECTION-
     YT FOR COMPOSITE SECTION(IN)                  YTC=  24.86
     YB FOR COMPOSITE SECTION(IN)                  YBC=  47.13

     MOMENT OF INERTIA OF NONCOMPOSITE SECTION(IN4)      XIG=    733320.3
     MOMENT OF INERTIA OF COMPOSITE SECTION(IN4)        XIGC=   1191531.0

     SECTION MODULUS FOR TOP FIBER(IN3)      ST=    20587.6  -NONCOMPOSITE SECTION-
     SECTION MODULUS FOR BOTTOM FIBER(IN3)   SB=    20156.8  -NONCOMPOSITE SECTION-
     ST FOR COMPOSITE SECTION(IN3)          STC=    47927.3
     SB FOR COMPOSITE SECTION(IN3)          SBC=    25278.6

**(5) DESIGN LOADS AND MOMENTS **

     IMPACT LOAD FACTOR                                  XIP= 0.196
     UNIFORM LOAD DUE TO GIRDER WEIGHT(KIP/FT)           WUG=  1.130
     UNIFORM LOAD DUE TO DECK WEIGHT(KIP/FT)             WUD=  0.525

     MOMENT DUE TO GIRDER WEIGHT(FT-KIP)                XMDG= 2387.56
     MOMENT DUE TO DECK PLUS GIRDER WEIGHT(FT-KIP)       XMD= 3496.62
     MOMENT DUE TO LIVE LOAD(FT-KIP)                     XML= 1237.86
     FACTORED MOMENT(FT-KIP)                             XMU= 7759.94

**(6) REQUIRED AMOUNT OF STRAND **



     NO.OF STRANDS REQUIRED      NS= 42

       ROW           NO.OF STRANDS PER ROW

        1                    13
        2                    13
        3                    13
        4                     3

     TOTAL AREA OF STRANDS REQUIRED(IN2)  AS=    6.426

     INITIAL PRESTRESSING FORCE(KIP)            FI=1214.514
      TOP FIBER STRESS(PSI)       SIT=    617.6
      BOTTOM FIBER STRESS(PSI)    SIB=   1631.7
     PRESTRESSING FORCE AT SERVICE LOAD(KIP)    FS= 925.343
      TOP FIBER STRESS(PSI)       SST=   1819.0
      BOTTOM FIBER STRESS(PSI)    SSB=   -458.2

     STRAINS AT CENTROID OF STRANDS

      STRAIN DUE TO PRESTRESS ONLY AT SERVICE LOAD     EPO= 0.00514
      TOTAL STRAIN AT ULTIMATE MOMENT                  EP2= 0.02947
```

GEOMETRIC PARAMETERS OF GIRDER SECTION

DISTANCE FROM CENTROID OF STRANDS TO CENTROID OF GIRDER SECTION-NONCOMPOSITE-(IN) ET= 32.0[illegible]
DISTANCE FROM CENTROID OF STRANDS TO BOTTOM FIBER(IN) DE= 4.28
DISTANCE FROM TOP FIBER(COMPOSITE SECTION) TO CENTROID OF STRANDS(IN) DE= 74.71
DISTANCE FROM TOP FIBER(COMPOSITE SECTION) TO NEUTRAL AXIS(IN) CD= [illegible]
RATIO OF STRESS BLOCK DEPTH TO CD BET1= .85[illegible]
COMPRESSION STRESS BLOCK DEPTH(IN) AD= [illegible]
DISTANCE FROM TOP FIBER(COMPOSITE SECT.) TO CENTROID OF STRESS BLOCK(IN) CSC= [illegible]
EQUIVALENT WIDTH OF WEB(COMPOSITE SECTION)(IN) BF= [illegible]

AVERAGE STRESS IN STRANDS AT ULTIMATE MOMENT(PSI) AFSU= 264362.9
WEIGHT AVERAGED CONCRETE STRENGTH(PSI) FCE= 4000.0
STRAND RATIO PA= 0.001194
REINFORCEMENT INDEX RIX= 0.[illegible]73

TENSILE FORCE IN STRANDS(LBS) TF= 1698796.[illegible]
COMPRESSIVE FORCE IN STRESS BLOCK(LBS) CF= 1706256.0
FLEXURAL DESIGN STRENGTH OF GIRDER(COMPOSITE SECTION)(FT-KIP) AMN= [illegible]
CRACKING MOMENT OF GIRDER(FT-KIP) AMCR= [illegible]

** 7) DEFLECTIONS AT MIDSPAN **

UPWARD DEFLECTION DUE TO PRESTRESSING FORCE(STRAIGHT STRANDS)(IN) DUP= [illegible]
DOWNWARD DEFLECTION DUE TO GIRDER WEIGHT(IN) DDG= 4.[illegible]
RESULTANT CAMBER AT ERECTION(IN) CAMB= [illegible]
DOWNWARD DEFLECTION DUE TO DECK WEIGHT(IN) DDD= [illegible]

**(8) COST INDEX PER UNIT SURFACE AREA OF BRIDGE **

TOTAL WEIGHT OF EACH MATERIAL PER GIRDER(LBS)

STRAND	TWS=	2893.6
DECK REINFORCEMENT	TWR=	2297.0
GIRDER CONCRETE	TWCG=	142029.5
DECK CONCRETE	TWCD=	65[illegible]75.0

WEIGHT OF EACH MATERIAL PER UNIT SURFACE AREA(LBS/FT2)

STRAND	WUS=	3.710
DECK REINFORCEMENT	WUR=	2.944
GIRDER CONCRETE	WUCG=	182.089
DECK CONCRETE	WUCD=	84.583

COST INDEX OF EACH MATERIAL PER UNIT SURFACE AREA(UNIT/FT2)

STRAND	CUS=	29.68
DECK REINFORCEMENT	CUR=	30.92
GIRDER CONCRETE	CUCG=	182.09
DECK CONCRETE	CUCD=	84.58

TOTAL COST INDEX PER UNIT SURFACE AREA BASED ON RELATIVE UNIT COSTS(UNIT/FT2)

TUT= 327.27

Sample of Data Output for Example 2

```
 **SAMPLE PROBLEM 2                                                                   CARD  1 **
 **WASHINGTON SERIES 14                                                               CARD  2 **
 **MARCH 11,1981                                                                      CARD  3 **

**(1) INPUT DATA **

 GEOMETRIC PROPERTIES

      GIRDER SPAN LENGTH(FT)     SL=     130.00
      GIRDER SPACING(FT)         GS=       6.00

      HORIZONTAL DIMENSIONS OF GIRDER CROSS SECTION
       B1=     42.00  B2=     24.00  B3=       5.00  B4=       2.00

      VERTICAL DIMENSIONS OF GIRDER CROSS SECTION
       D1=     73.50  D2=      1.87  D3=       1.62  D4=       2.00  D5=       3.00  D6=       2.00

      CENTER TO CENTER SPACING OF STRANDS IN        CTC=  2.00
      CONCRETE SURFACE TO CENTER OF STRANDS(IN      CSC=  2.00
      NUMBER OF STRANDS WITHIN WEB WIDTH            SWW=  2.00

 MATERIAL PROPERTIES   (*=INTERNALLY ASSIGNED VALUES)

   CONCRETE

      SPECIFIED GIRDER CONCRETE STRENGTH(PSI)       FCP = 6000.0
      CONCRETE STRENGTH AT PRESTRESS TRANSFER(PSI) FCPI= 4500.0
      SPECIFIED DECK CONCRETE STRENGTH(PSI)         FCD = 4000.0*

      UNIT WEIGHT OF GIRDER CONCRETE(PCF)           WC = 145.0
      UNIT WEIGHT OF DECK CONCRETE(PCF)             WCD= 145.0
      UNIT WEIGHT OF GIRDER(PCF)                    WG = 150.0
      UNIT WEIGHT OF DECK(PCF)                      WD = 150.0

   STRAND

      STRESS-STRAIN CURVE OF STRAND
                                         STRESS(PSI)       STRAIN
      SPECIFIED ULTIMATE STRENGTH FPU= 270000.0*  SU= 0.0400*
      INTERMEDIATE STRESS         FPM= 255000.0*  SM= 0.0120*
      SPECIFIED YIELD STRESS      FPY= 230000.0   SY= 0.0082

      MODULUS OF ELASTICITY OF STRAND(PSI)   EPS= 28000000.0
      TOTAL PRESTRESS LOSSES (PSI)           YLS=    45000.0



 GIRDER STIFFNESS REDUCTION FACTOR            SRF= 0.550

 RELATIVE COST INDEX(UNIT/LB)
      GIRDER CONCRETE                    RUCG=  1.00
      DECK CONCRETE                      RUCD=  1.00
      STRAND                             RUS =  8.00
      DECK REINFORCEMENT                 RUR =  9.00
      EPOXY COATED DECK REINFORCEMENT    RUE = 12.00

**(2) CALCULATED MATERIAL PROPERTIES **

 CONCRETE

   ALLOWABLE GIRDER CONCRETE STRESSES(PSI)

     AT PRESTRESS TRANSFER
        COMPRESSIVE STRESS      FCCI= 2700.0
        TENSILE STRESS          FCTI=    0.0
     AT SERVICE LOAD
        COMPRESSIVE STRESS      FCC = 2400.0
        TENSILE STRESS          FCT =  464.7

   MODULUS OF RUPTURE OF GIRDER CONCRETE(PSI)   FCR=  580.9
```

```
  MODULUS OF ELASTICITY PSI
     GIRDER CONCRETE                                    EI = [illegible]
     GIRDER CONCRETE AT PRESTRESS TRANSFER             EII= [illegible]
     DECK CONCRETE                                      EIC= 3644143.5

  MODULAR RATIO,DECK TO GIRDER      XNE= 0.816

 STRAND
     ALLOWABLE STRESS AT SERVICE LOAD(PSI)       FPS= 184000.0
     EFFECTIVE PRESTRESS AT SERVICE LOAD(PSI)   PSE= 144000.0

**(3) THICKNESS AND REINFORCEMENT OF DECK **

     EFFECTIVE SPAN OF DECK(FT)      SE=  3.33
     THICKNESS OF DECK(IN)           TD=  7.00
     BAR SPACING(IN) OF NO.5 BAR     BS=  8.50

** 4. SECTIONAL PROPERTIES OF GIRDER **

     EFFECTIVE TOP FLANGE(DECK) WIDTH(IN)   BE=   [illegible]

     AREA OF NONCOMPOSITE SECTION(IN2)       AG=  [illegible]4.18
     AREA OF COMPOSITE SECTION(IN2)          AC= 10[illegible]

     DISTANCE FROM CENTROID TO TOP FIBER(IN)      YT=  35.[illegible]  -NONCOMPOSITE SECTION-
     DISTANCE FROM CENTROID TO BOTTOM FIBER(IN)   YB=  38.1[illegible]  -NONCOMPOSITE SECTION-
     YT FOR COMPOSITE SECTION(IN)                YTC=  20.[illegible]1
     YB FOR COMPOSITE SECTION(IN)                YBC=  52.[illegible]

     MOMENT OF INERTIA OF NONCOMPOSITE SECTION(IN4)     XIG=    514311.[illegible]
     MOMENT OF INERTIA OF COMPOSITE SECTION(IN4)       XIGC=    [illegible]

     SECTION MODULUS FOR TOP FIBER(IN3)       ST=   14555.6  -NONCOMPOSITE SECTION-
     SECTION MODULUS FOR BOTTOM FIBER(IN3)    SB=   13475.7  -NONCOMPOSITE SECTION-
     ST FOR COMPOSITE SECTION(IN3)           STC=   43724.2
     SB FOR COMPOSITE SECTION(IN3)           SBC=   17043.8

**(5) DESIGN LOADS AND MOMENTS **



     IMPACT LOAD FACTOR                                      XIF= 0.1[illegible]
     UNIFORM LOAD DUE TO GIRDER WEIGHT(KIP/FT)               WUG=  0.702
     UNIFORM LOAD DUE TO DECK WEIGHT(KIP/FT                  WUD=  0.525

     MOMENT DUE TO GIRDER WEIGHT(FT-KIP)                    XMDG= 1483.56
     MOMENT DUE TO DECK PLUS GIRDER WEIGHT(FT-KIP)           XMD= 25[illegible]
     MOMENT DUE TO LIVE LOAD(FT-KIP)                         XML= 1237.[illegible]
     FACTORED MOMENT(FT-KIP)                                 XMU= 6584.74

**(6) REQUIRED AMOUNT OF STRAND **

     NO.OF STRANDS REQUIRED      NS= 33

       ROW            NO.OF STRANDS PER ROW

         1                     11
         2                     11
         3                      6
         4                      2
         5                      2
         6                      1

     TOTAL AREA OF STRANDS REQUIRED(IN2)   AS=    5.049

     INITIAL PRESTRESSING FORCE(KIP)              FI= 954.260
      TOP FIBER STRESS(PSI)        SIT=     414.3
      BOTTOM FIBER STRESS(PSI)     SIB=    2475.0
     PRESTRESSING FORCE AT SERVICE LOAD(KIP)   FS= 727.055
      TOP FIBER STRESS(PSI)        SST=    1941.8
      BOTTOM FIBER STRESS(PSI)     SSB=    -458.7

     STRAINS AT CENTROID OF STRANDS

      STRAIN DUE TO PRESTRESS ONLY AT SERVICE LOAD    EPO= 0.00[illegible]
      TOTAL STRAIN AT ULTIMATE MOMENT                 EPS= [illegible]
```

GEOMETRIC PARAMETERS OF GIRDER SECTION

DISTANCE FROM CENTROID OF STRANDS TO CENTROID OF GIRDER SECTION-NONCOMPOSITE-(IN) E1= [illegible]
DISTANCE FROM CENTROID OF STRANDS TO BOTTOM FIBER(IN) CE= 4.54
DISTANCE FROM TOP FIBER(COMPOSITE SECTION) TO CENTROID OF STRANDS(IN) DE= 75.[illegible]
DISTANCE FROM TOP FIBER(COMPOSITE SECTION) TO NEUTRAL AXIS(IN) CC= [illegible]
RATIO OF STRESS BLOCK DEPTH TO CD BET1= [illegible]
COMPRESSION STRESS BLOCK DEPTH(IN) AD= [illegible]
DISTANCE FROM TOP FIBER(COMPOSITE SECT.) TO CENTROID OF STRESS BLOCK(IN) DSC= [illegible]
EQUIVALENT WIDTH OF WEB(COMPOSITE SECTION)(IN) BA= [illegible]

AVERAGE STRESS IN STRANDS AT ULTIMATE MOMENT(PSI) AFSU= 268214.8
WEIGHT AVERAGED CONCRETE STRENGTH(PSI) FCE= 4000.0
STRAND RATIO PA= 0.[illegible]
REINFORCEMENT INDEX PIX= [illegible]

TENSILE FORCE IN STRANDS(LBS) TF= 1354[illegible].7
COMPRESSIVE FORCE IN STRESS BLOCK(LBS) CF= 137[illegible].9
FLEXURAL DESIGN STRENGTH OF GIRDER(COMPOSITE SECTION)(FT-KIP) FMN= [illegible]
CRACKING MOMENT OF GIRDER(FT-KIP) FMCR= [illegible]

**(7) DEFLECTIONS AT MIDSPAN **

UPWARD DEFLECTION DUE TO PRESTRESSING FORCE(STRAIGHT STRANDS)(IN) DUP= [illegible]
DOWNWARD DEFLECTION DUE TO GIRDER WEIGHT(IN) DDG= [illegible]
RESULTANT CAMBER AT ERECTION(IN) CAMB= 4.[illegible]

DOWNWARD DEFLECTION DUE TO DECK WEIGHT(IN) DDD= 1.4[illegible]

**(8) COST INDEX PER UNIT SURFACE AREA OF BRIDGE **

TOTAL WEIGHT OF EACH MATERIAL PER GIRDER(LBS)

STRAND	TWS=	2273.7
DECK REINFORCEMENT	TWR=	2297.0
GIRDER CONCRETE	TWCG=	88253.0
DECK CONCRETE	TWCD=	65975.0

WEIGHT OF EACH MATERIAL PER UNIT SURFACE AREA(LBS/FT2)

STRAND	WUS=	2.914
DECK REINFORCEMENT	WUR=	2.944
GIRDER CONCRETE	WUCG=	113.144
DECK CONCRETE	WUCD=	84.583

COST INDEX OF EACH MATERIAL PER UNIT SURFACE AREA(UNIT/FT2)

STRAND	CUS=	23.31
DECK REINFORCEMENT	CUR=	30.[illegible]
GIRDER CONCRETE	CUCG=	113.14
DECK CONCRETE	CUCD=	84.58

TOTAL COST INDEX PER UNIT SURFACE AREA BASED ON RELATIVE UNIT COSTS(UNIT/FT2)

TUT= 2[illegible]1.[illegible]

APPENDIX F - COST CHARTS

This appendix contains cost charts for the following girders:

1. AASHTO Type IV
2. AASHTO Type V
3. AASHTO Type VI
4. Modified AASHTO Type IV
5. Modified AASHTO Type V
6. Modified AASHTO Type VI
7. Colorado G54
8. Colorado G68
9. Modified Colorado G68/6
10. Washington Series 80
11. Washington Series 100
12. Washington Series 120
13. Washington Series 14
14. Modified Washington 80/6
15. Modified Washington 100/6
16. Modified Washington 120/6
17. Modified Washington 14/6
18. Bulb-T BT48
19. Bulb-T BT60
20. Bulb-T BT72
21. Modified BT48/6
22. Modified BT60/6
23. Modified BT72/6

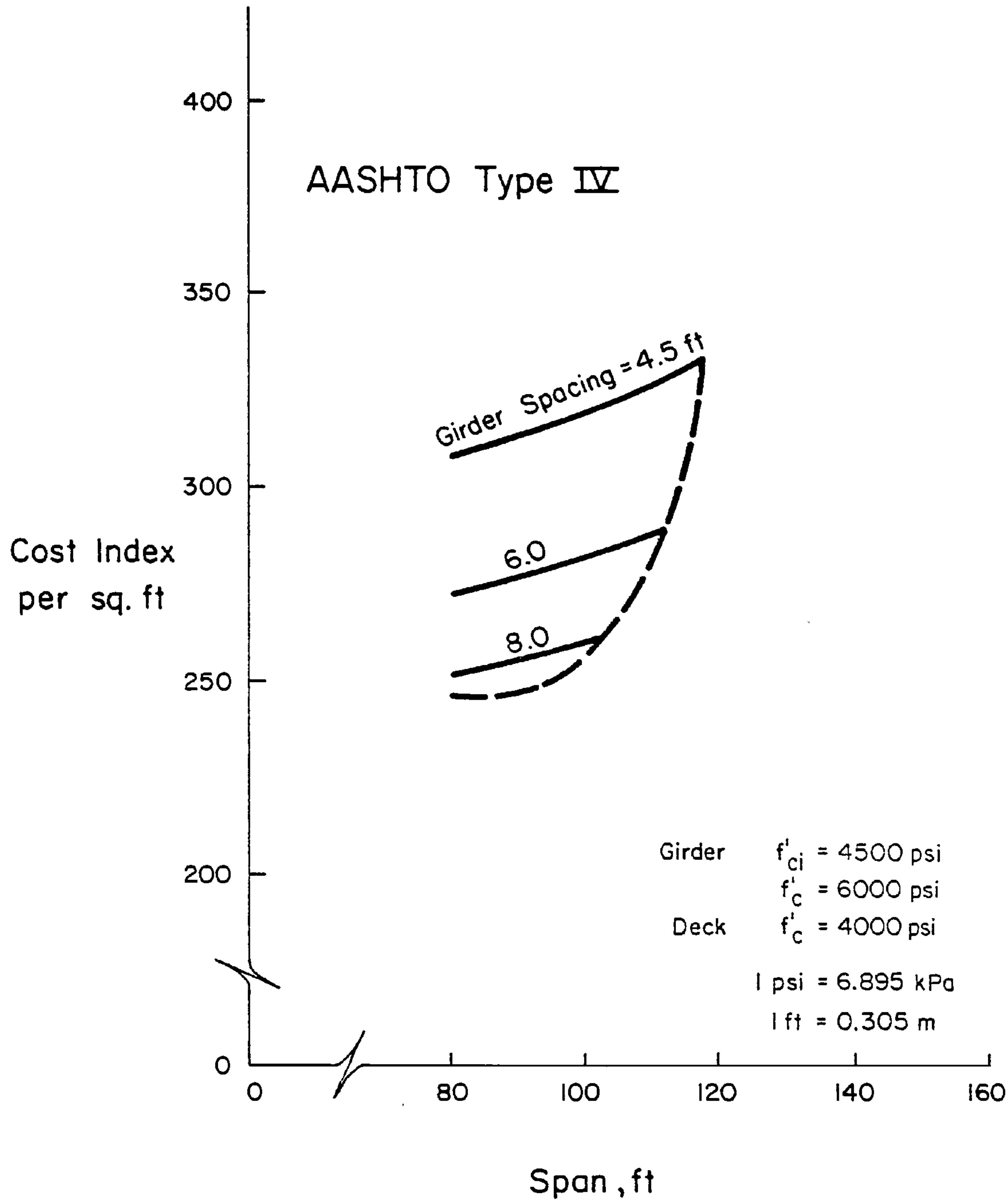

Figure 35 Cost Chart for AASHTO Type IV Girder

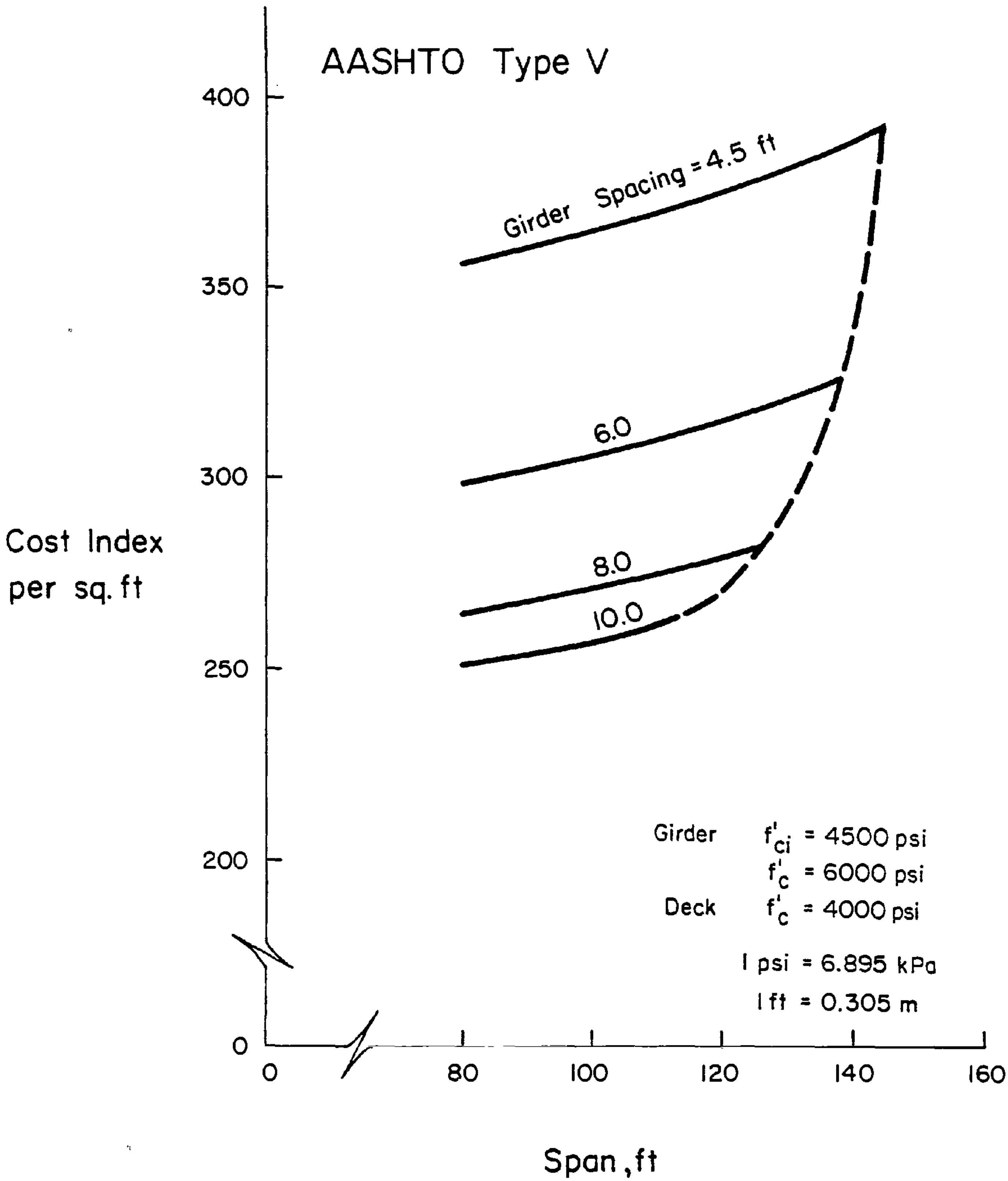

Figure 36 Cost Chart for AASHTO Type V Girder

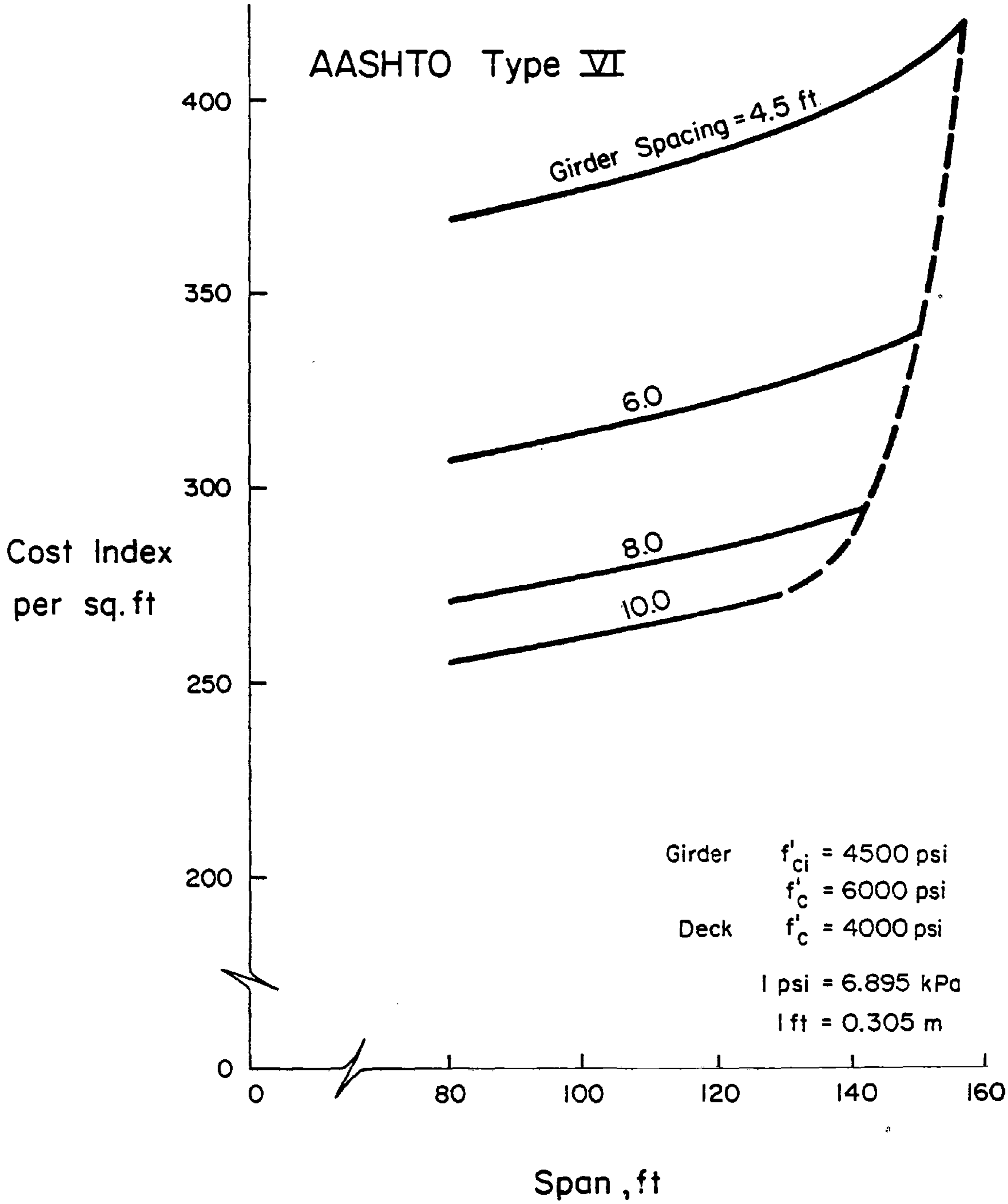

Figure 37 Cost Chart for AASHTO Type VI Girder

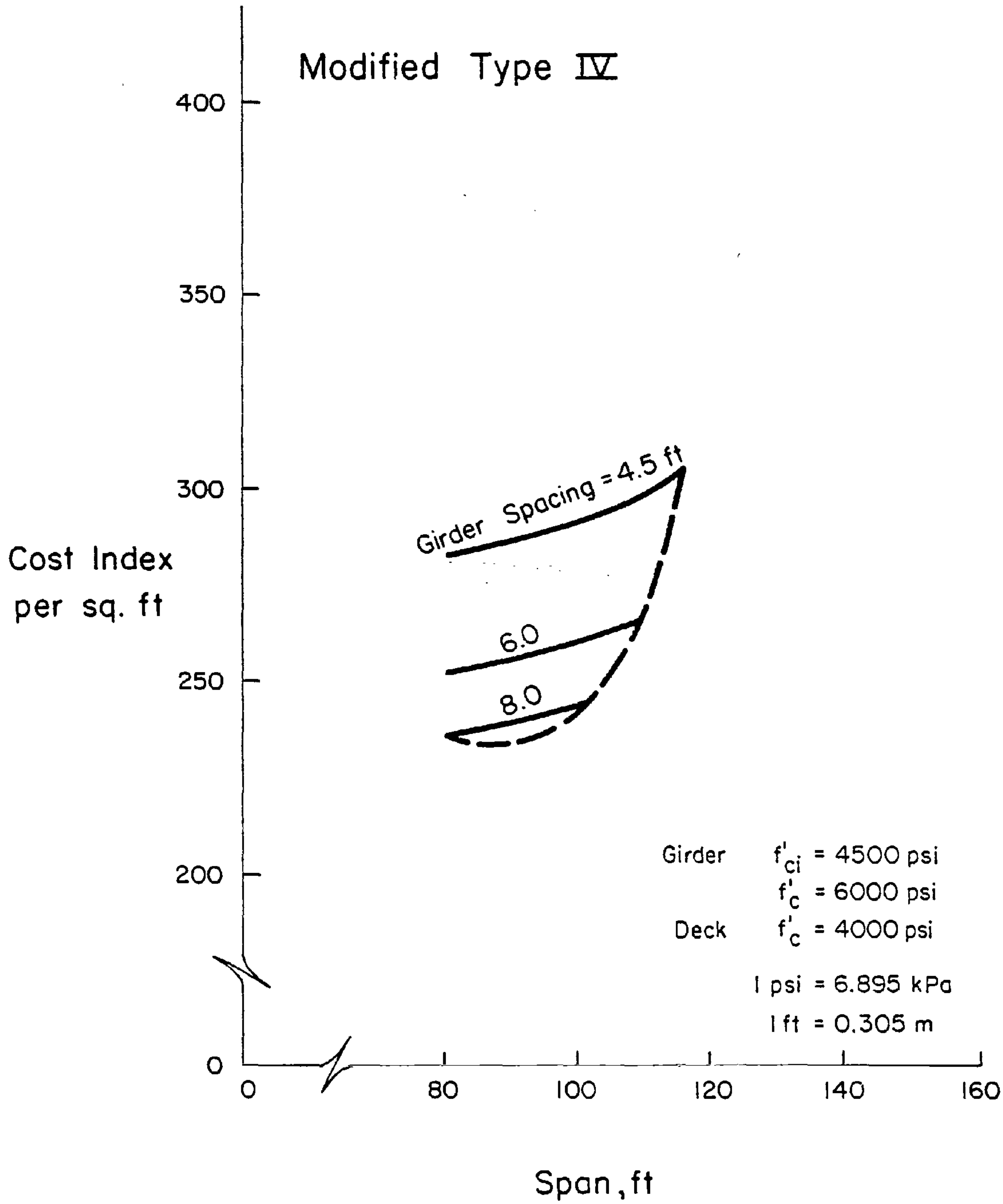

Figure 38 Cost Chart for Modified Type IV Girder

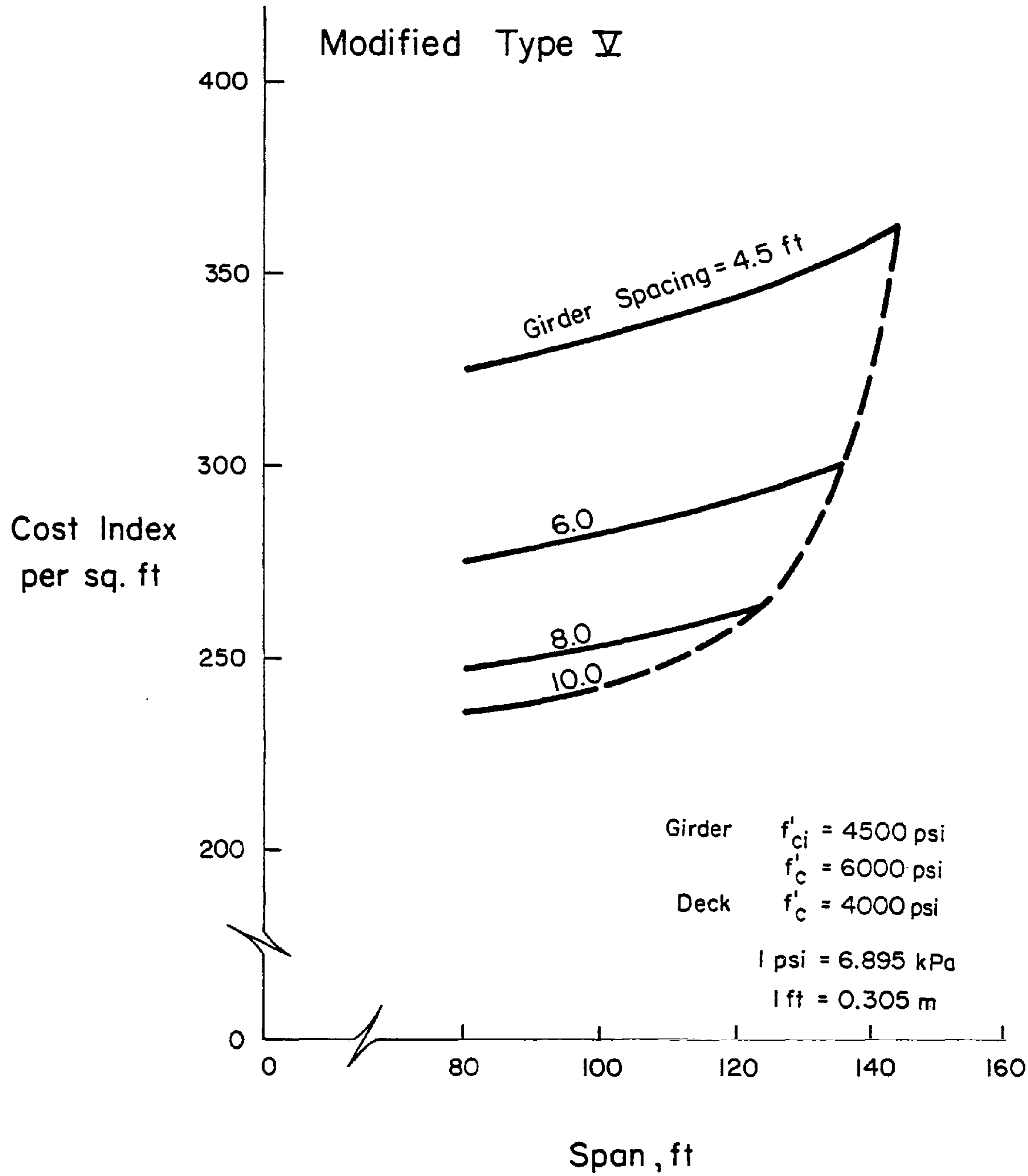

Figure 39 Cost Chart for Modified Type V Girder

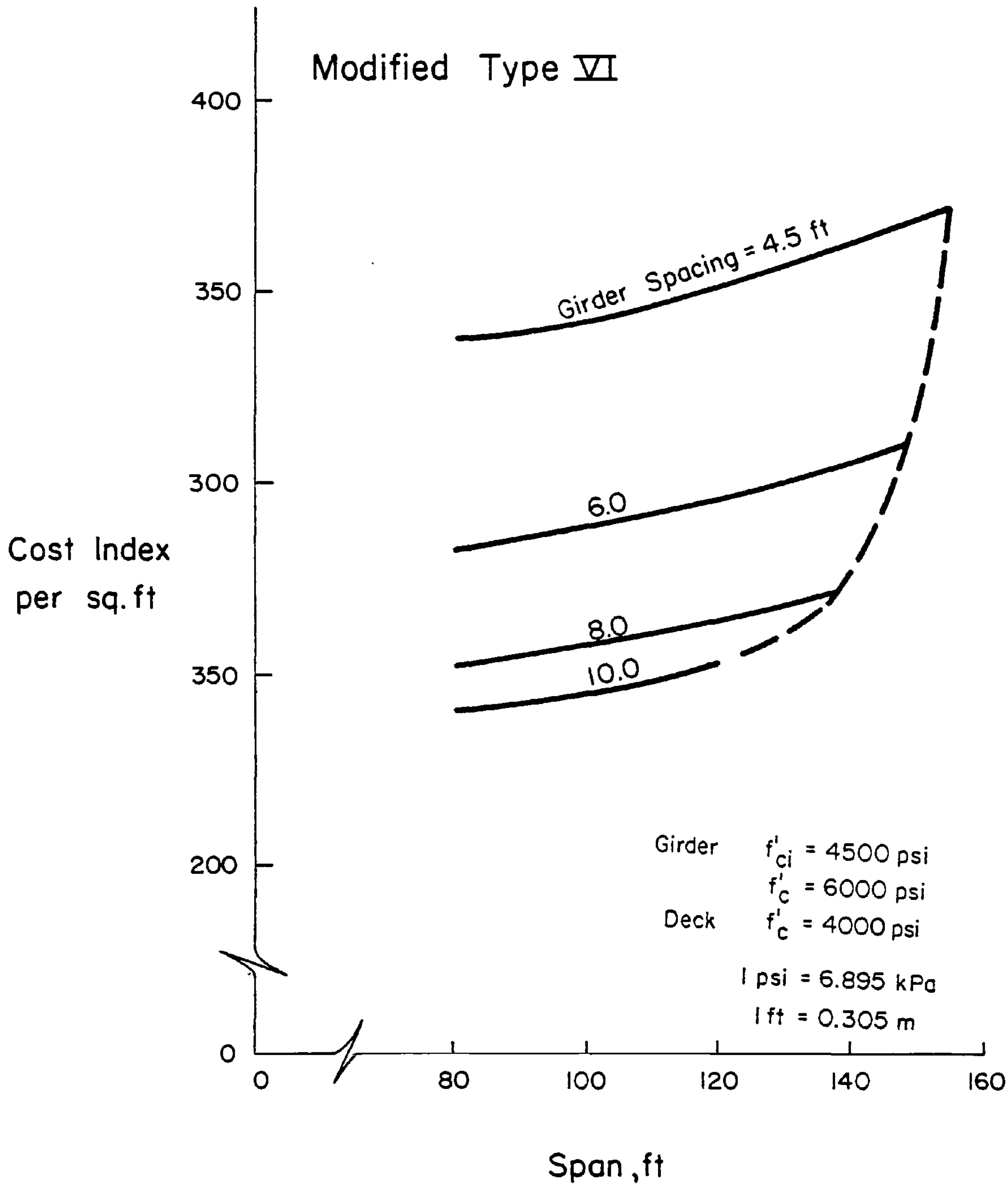

Figure 40 Cost Chart for Modified Type VI Girder

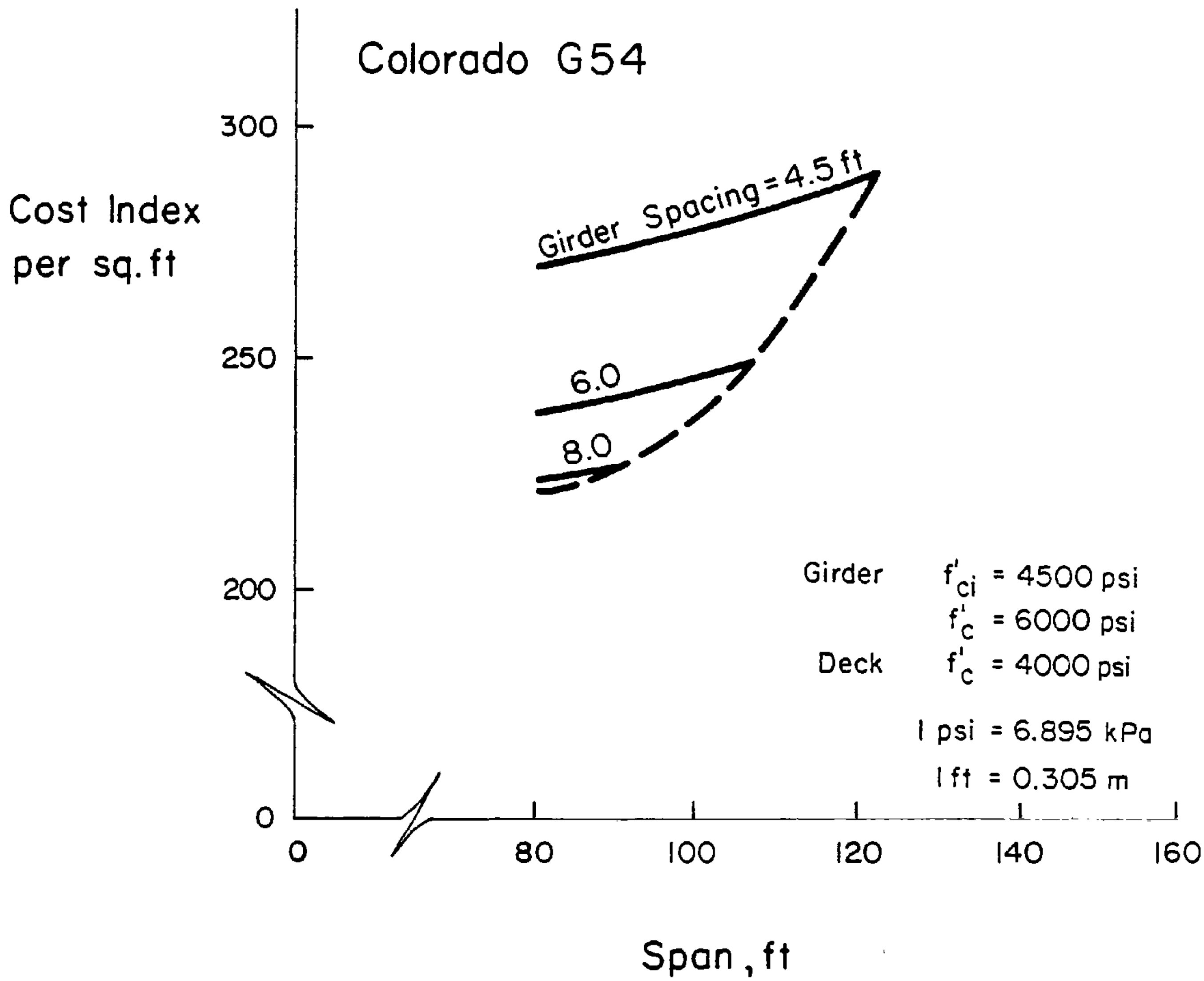

Figure 41 Cost Chart for Colorado G54 Girder

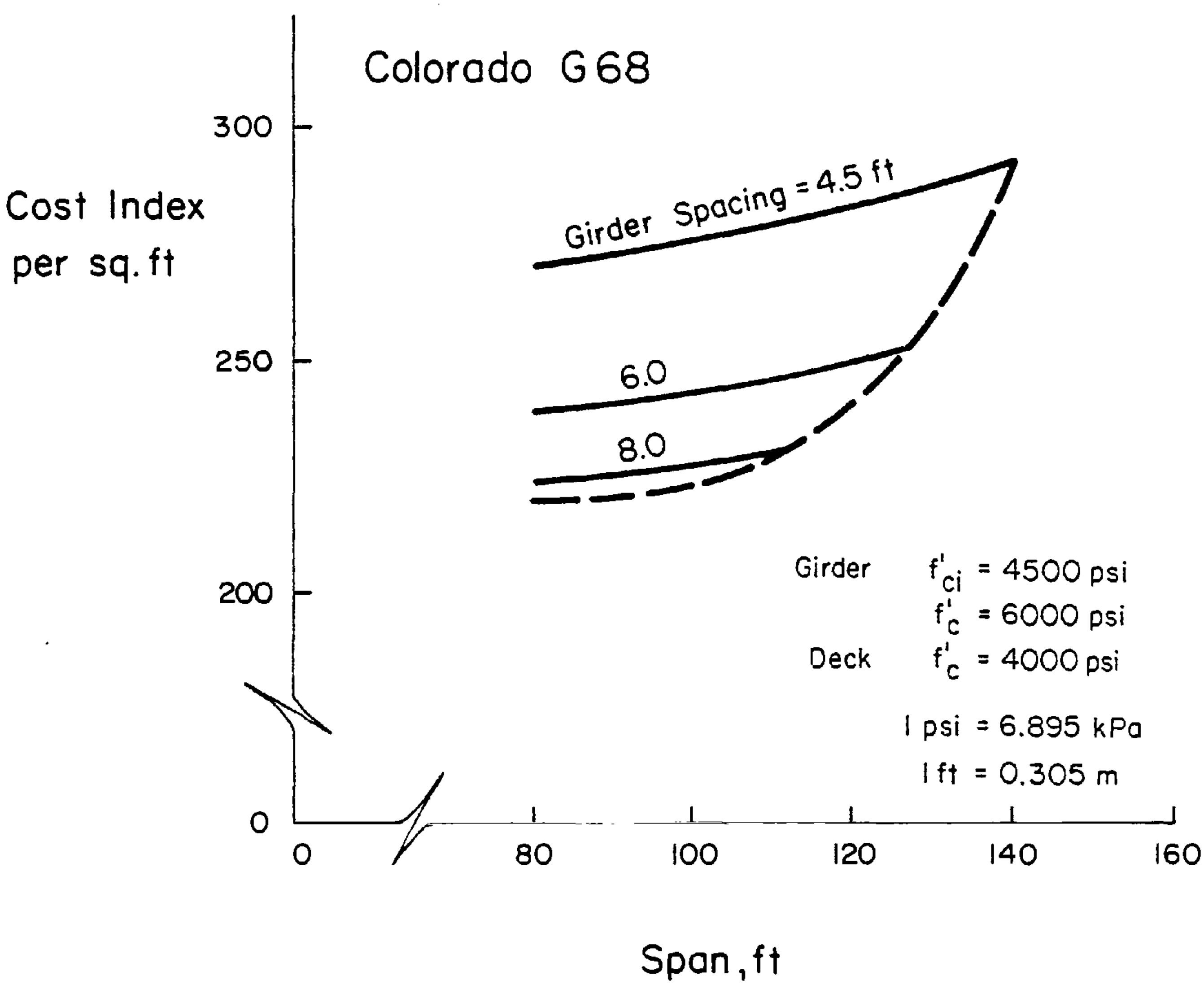

Figure 42 Cost Chart for Colorado G68 Girder

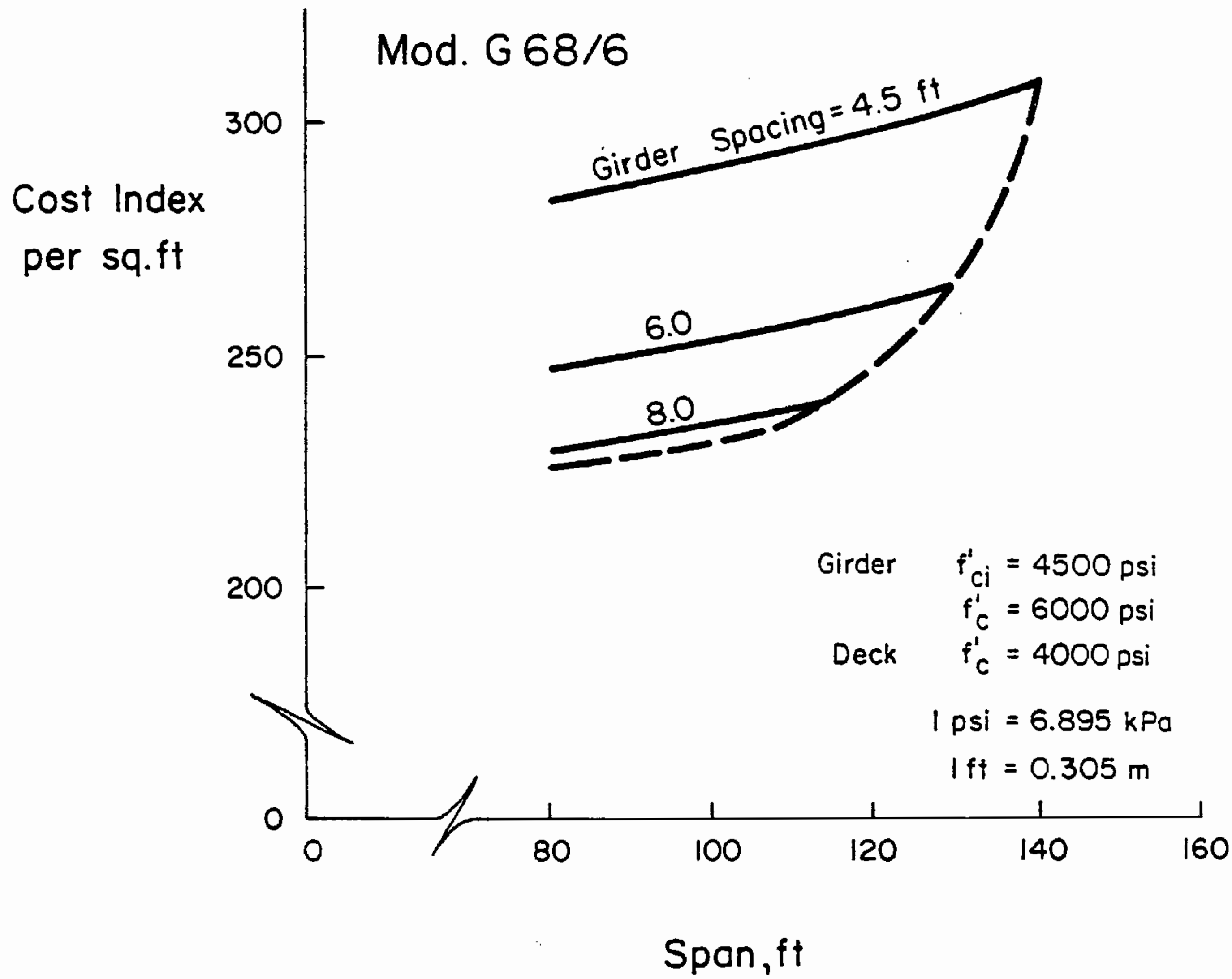

Figure 43 Cost Chart for Modified G68/6 Girder

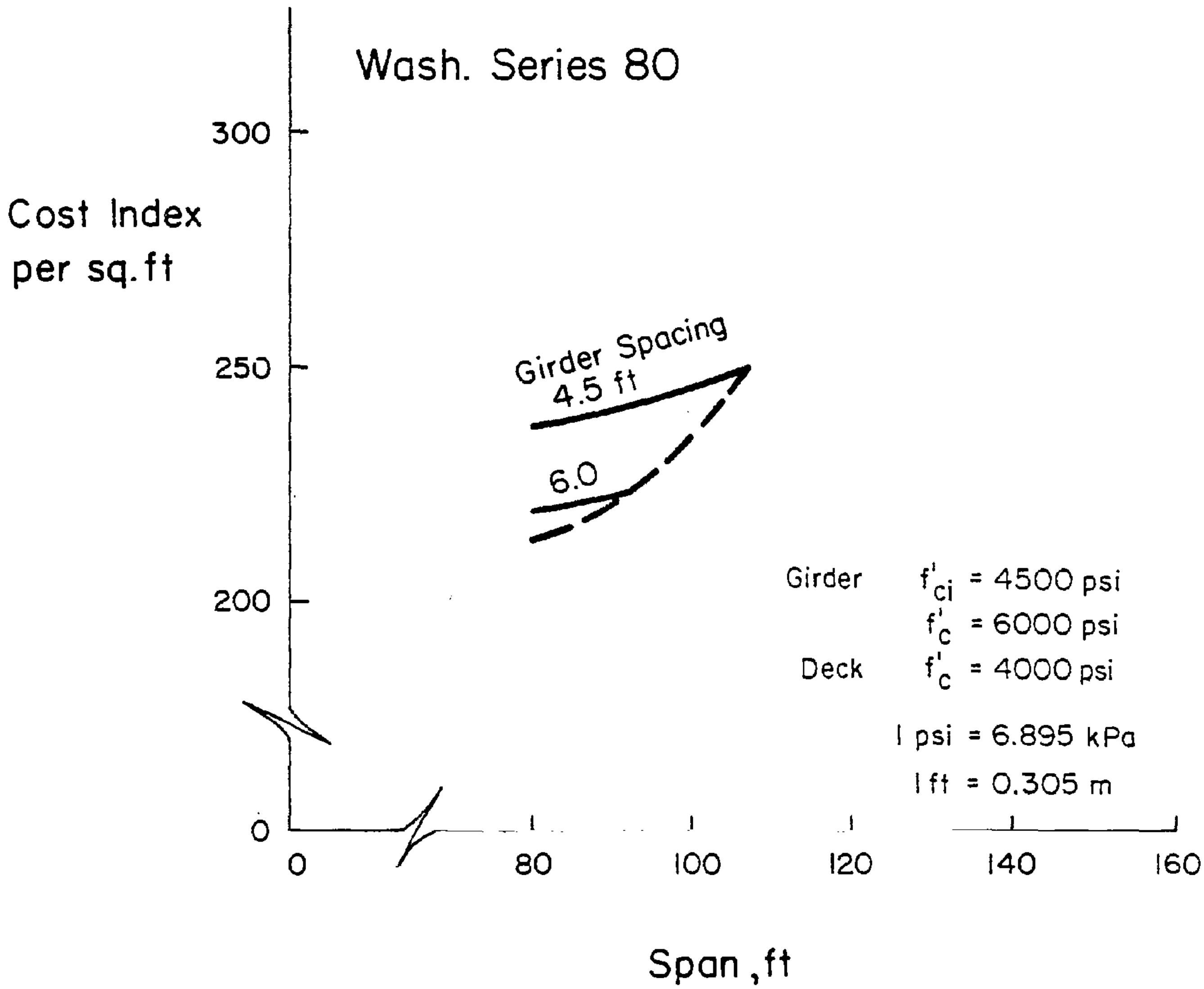

Figure 44 Cost Chart for Washington Series 80

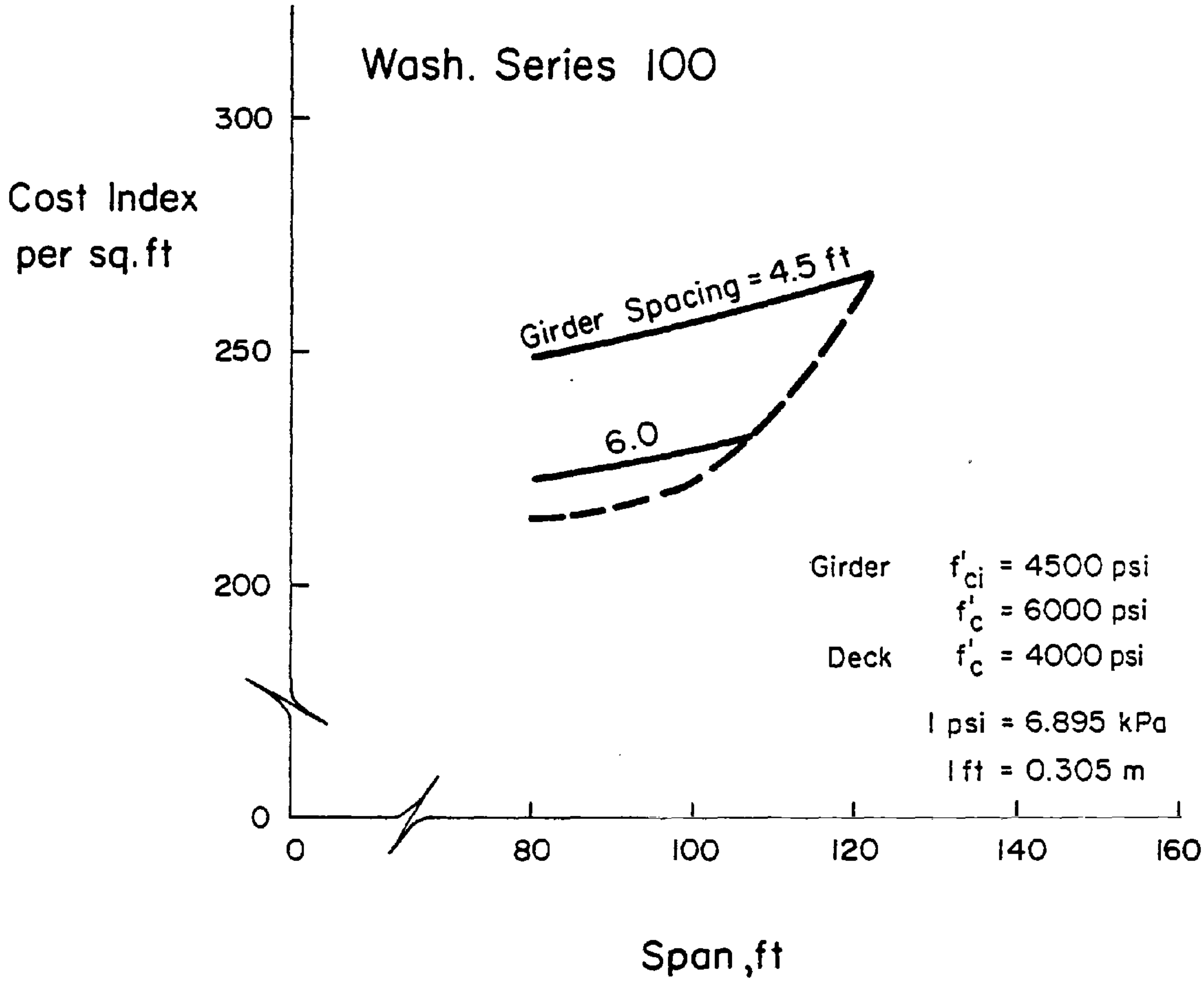

Figure 45 Cost Chart for Washington Series 100

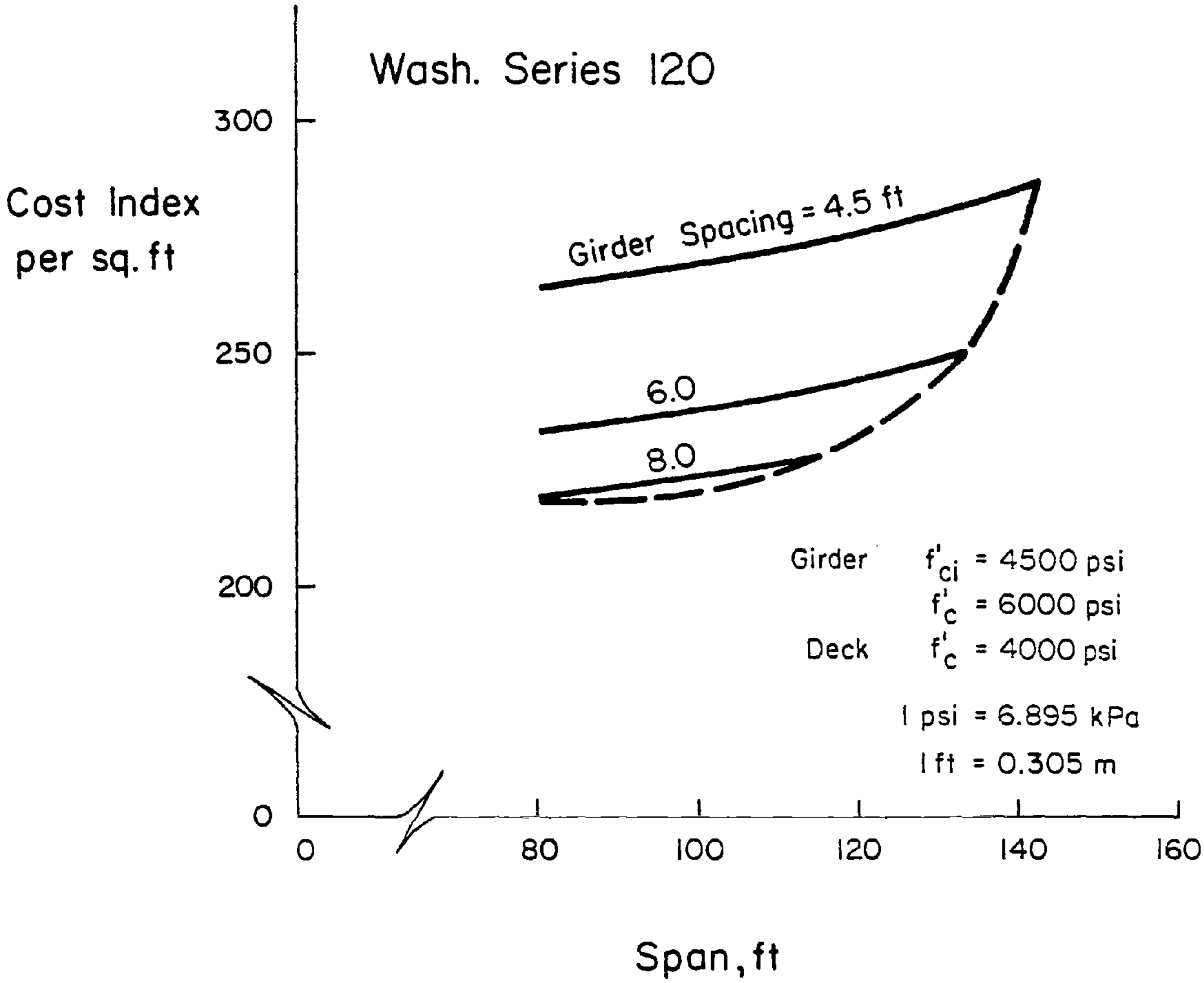

Figure 46 Cost Chart for Washington Series 120

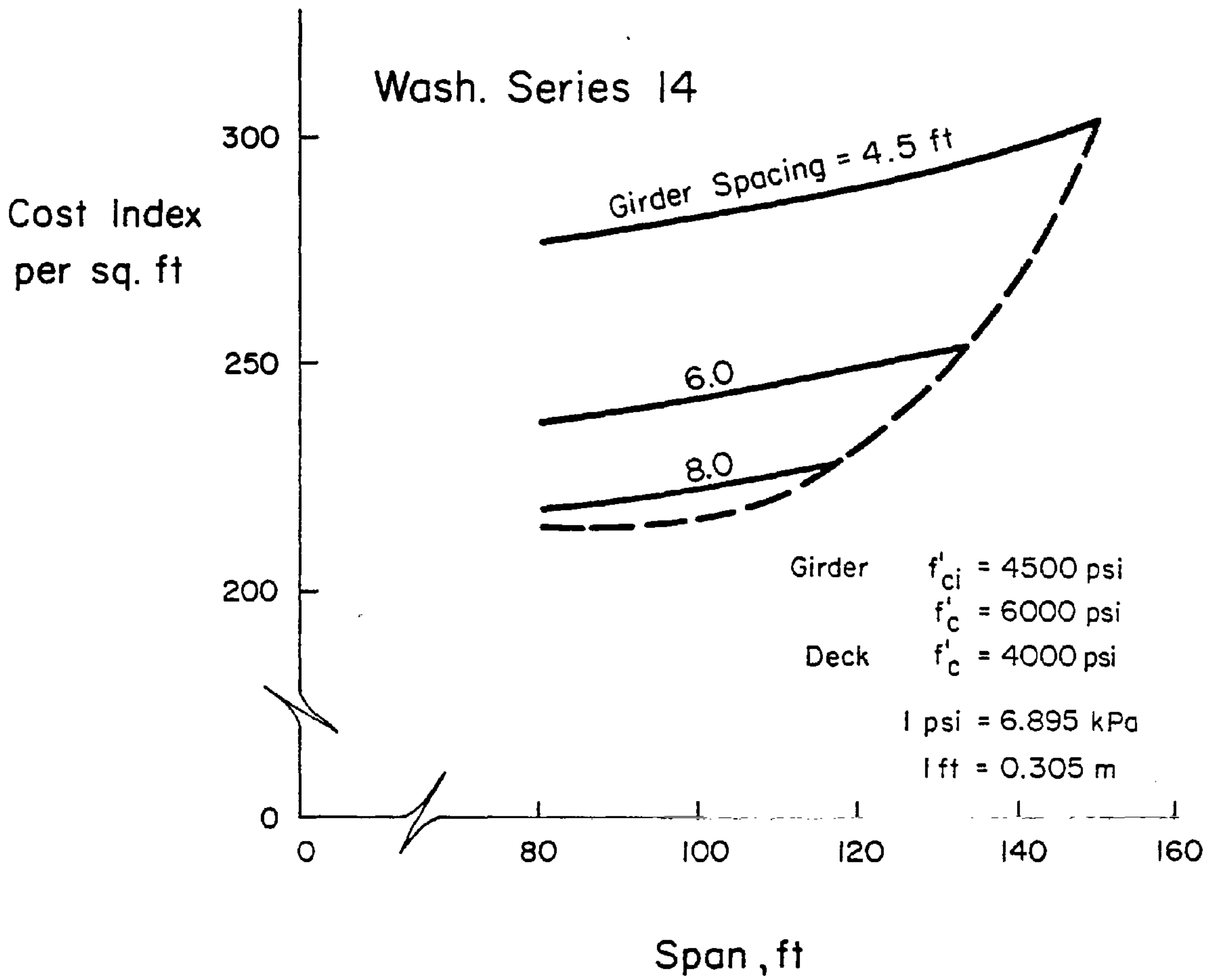

Figure 47 Cost Chart for Washington Series 14

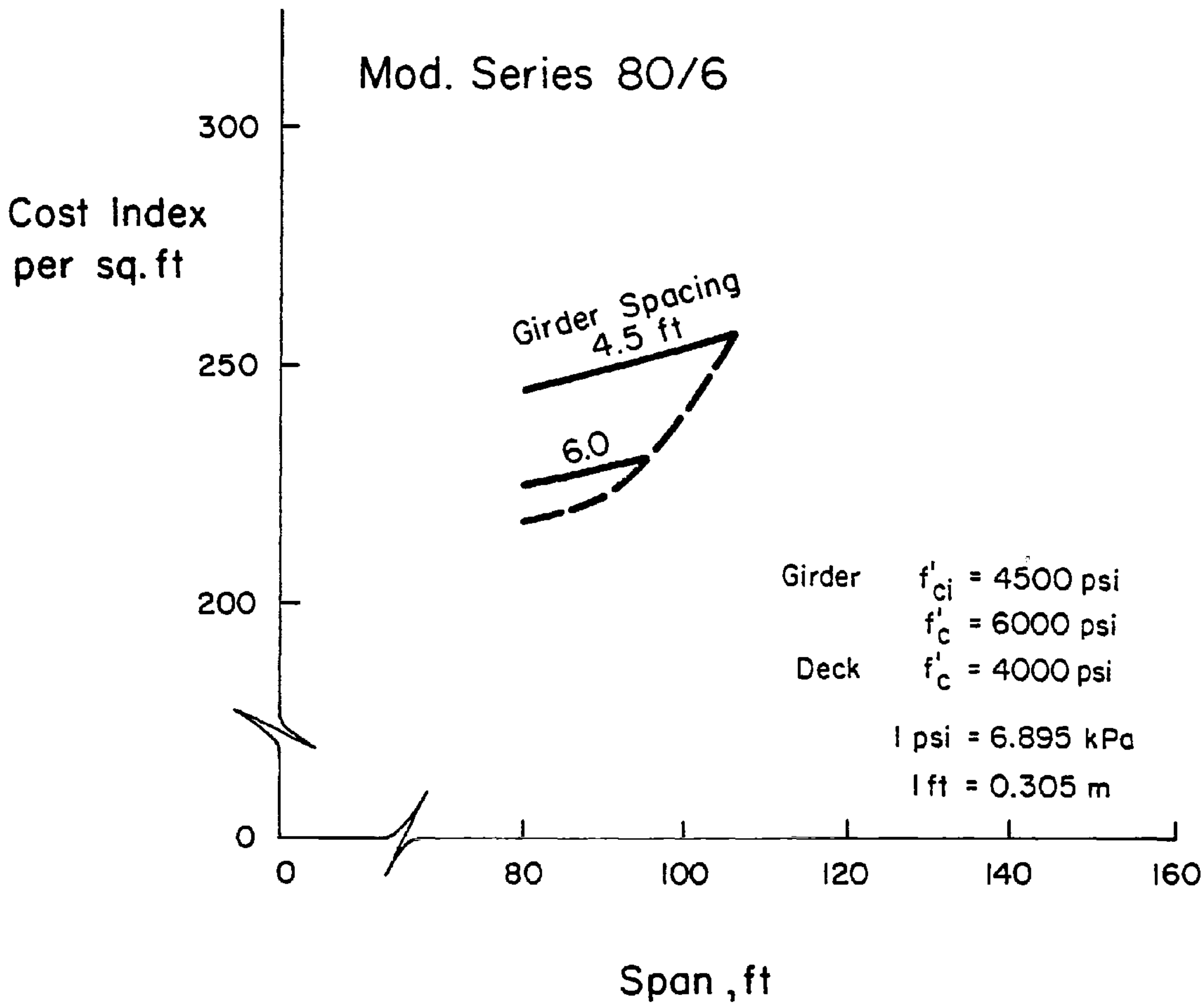

Figure 48 Cost Chart for Modified Series 80/6

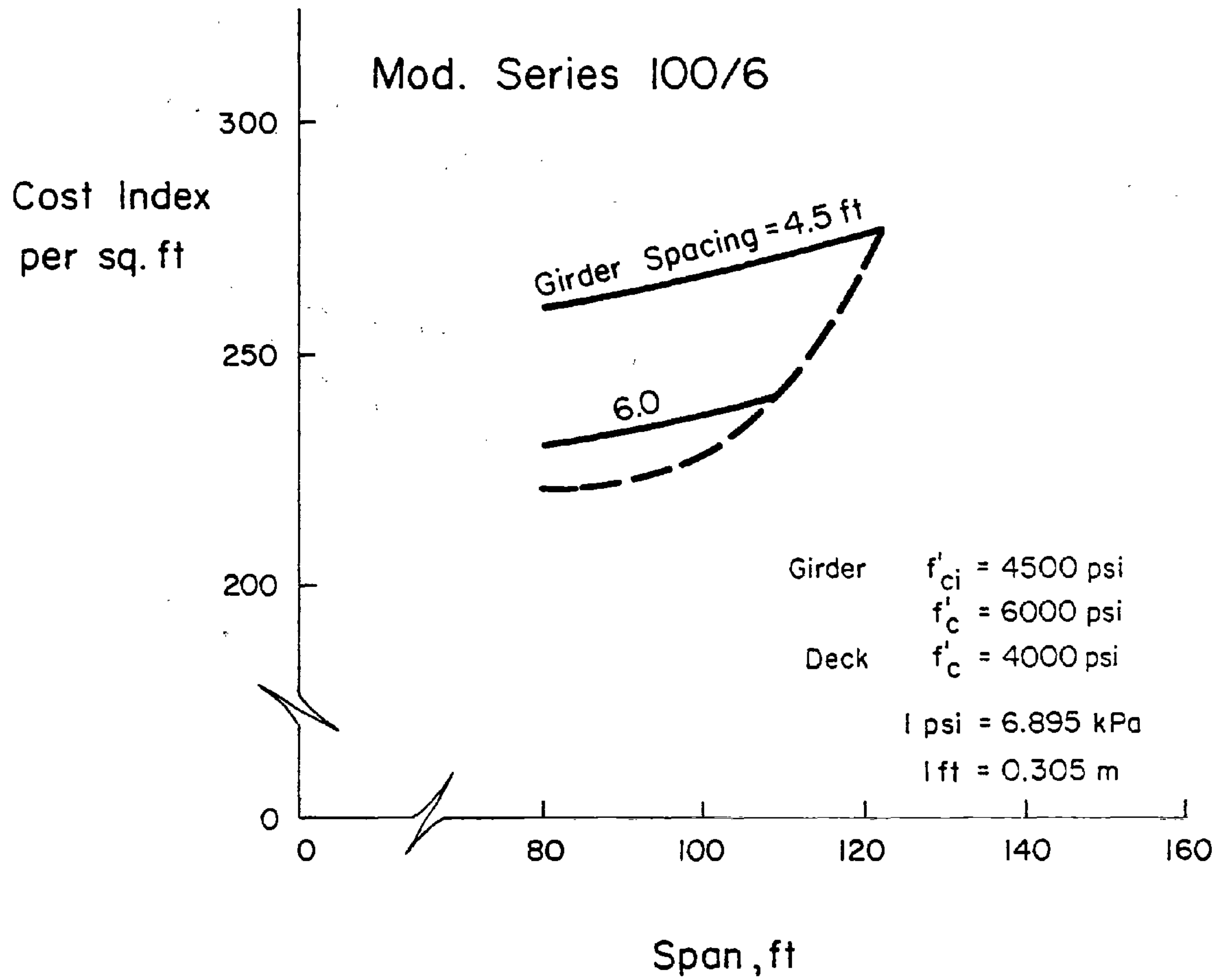

Figure 49 Cost Chart for Modified Series 100/6

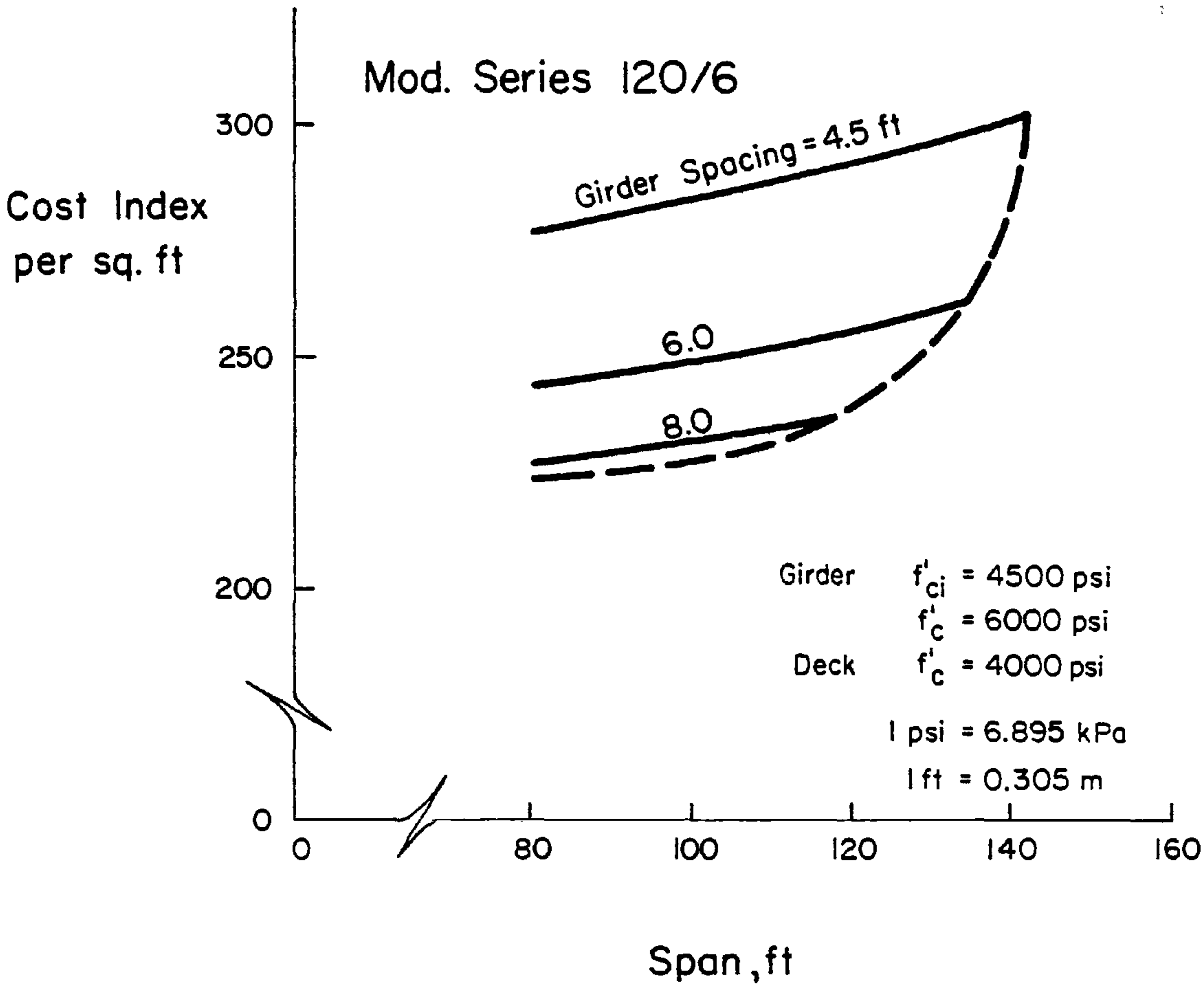

Figure 50 Cost Chart for Modified Series 120/6

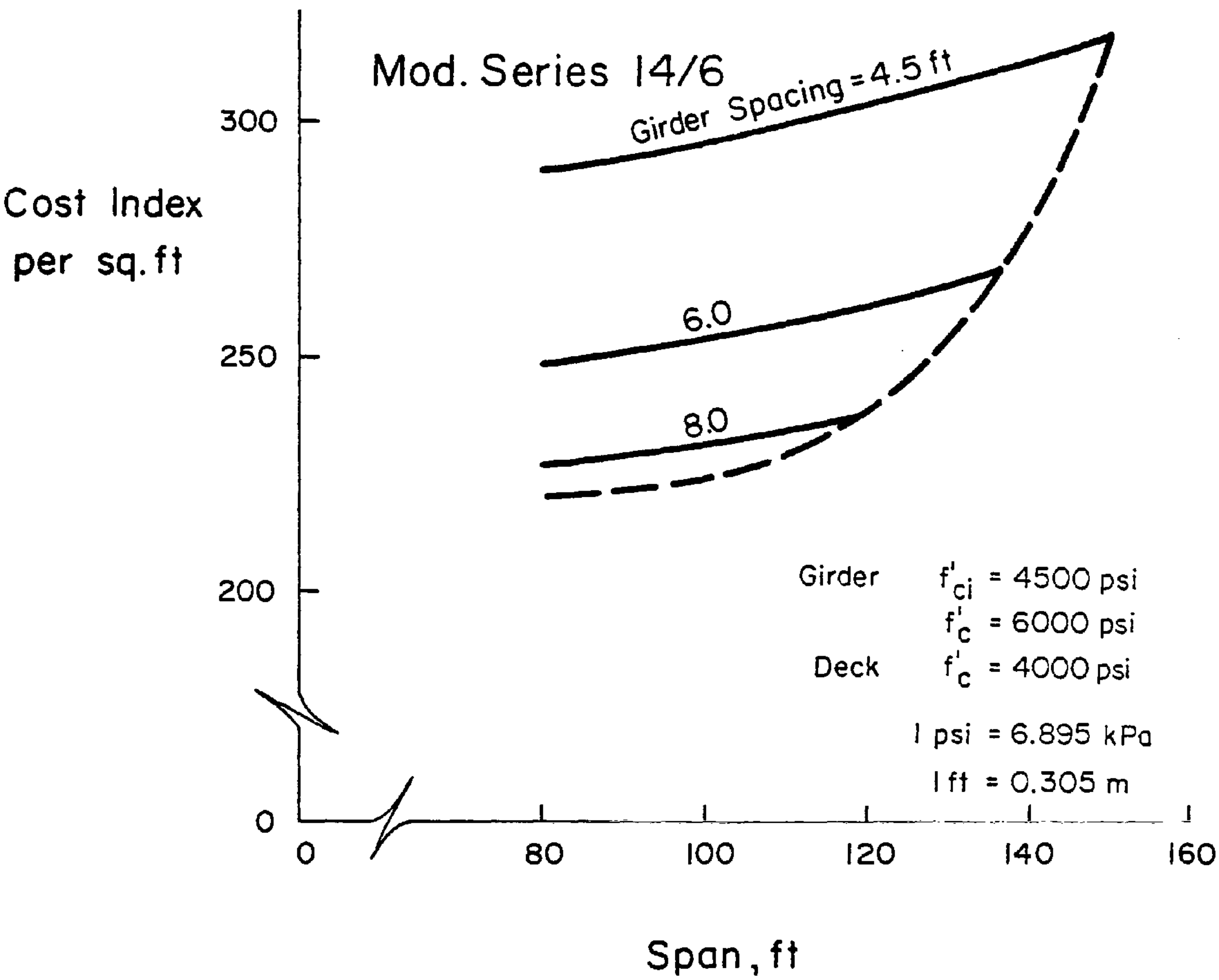

Figure 51 Cost Chart for Modified Series 14/6

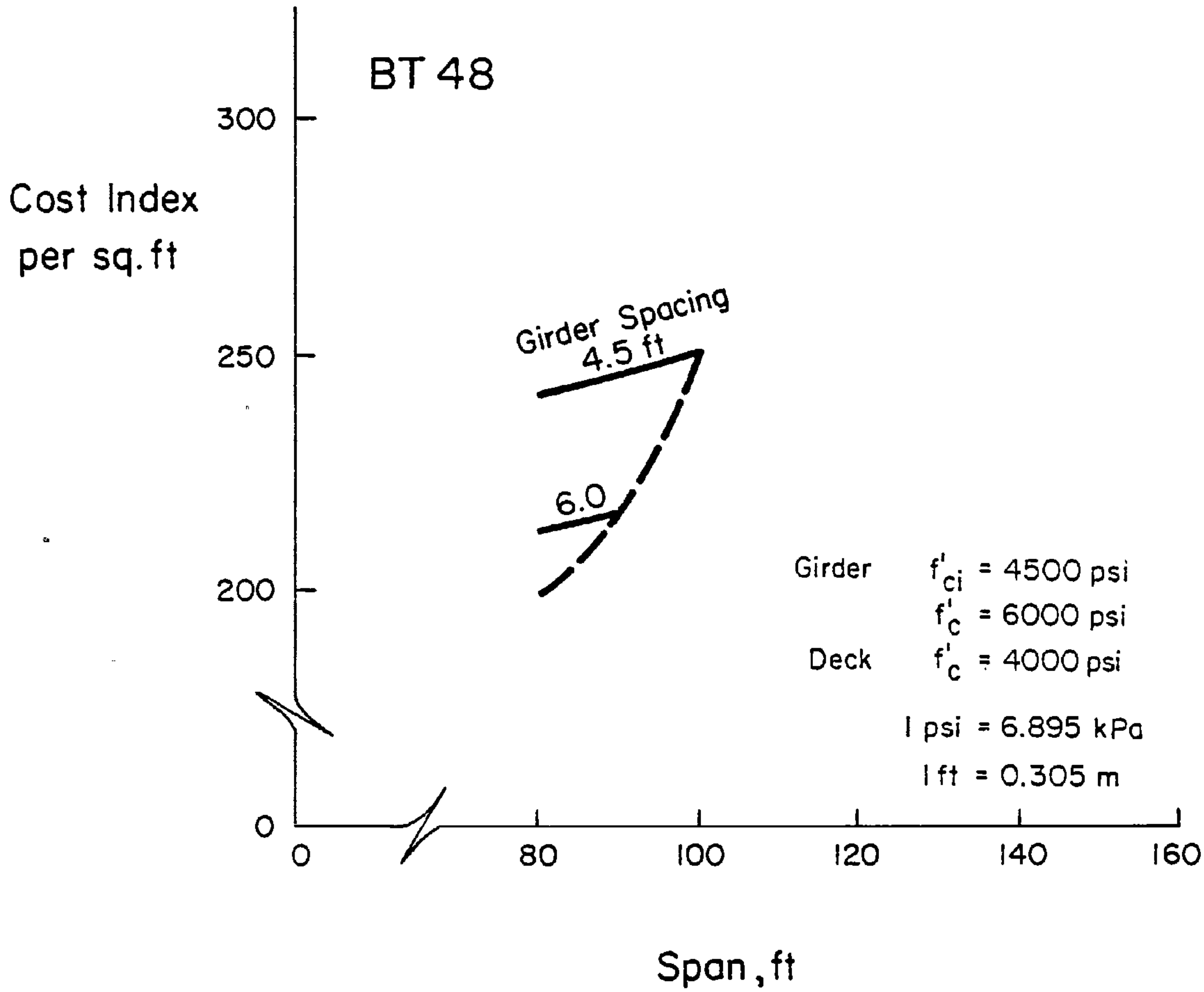

Figure 52 Cost Chart for BT48

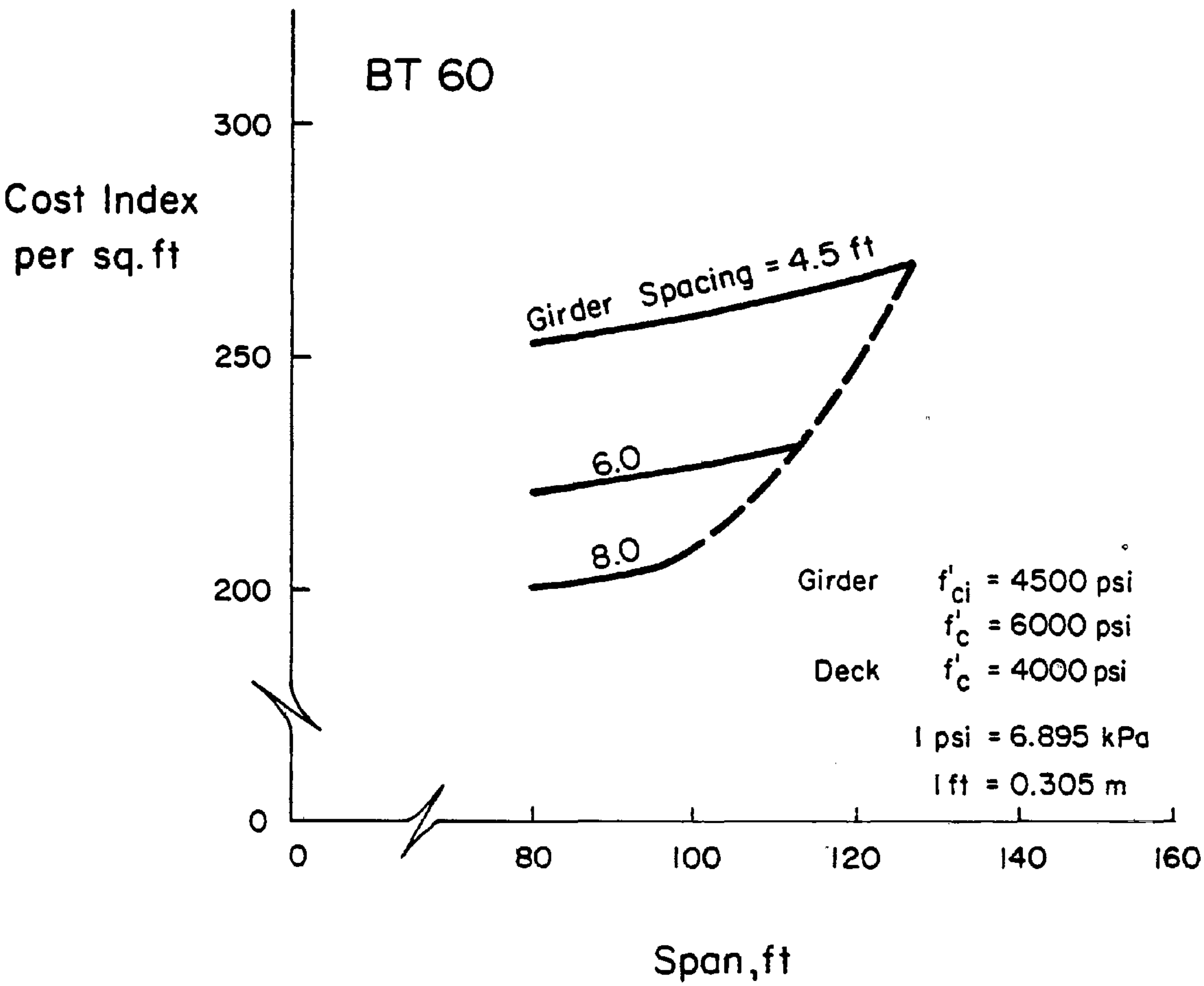

Figure 53 Cost Chart for BT60

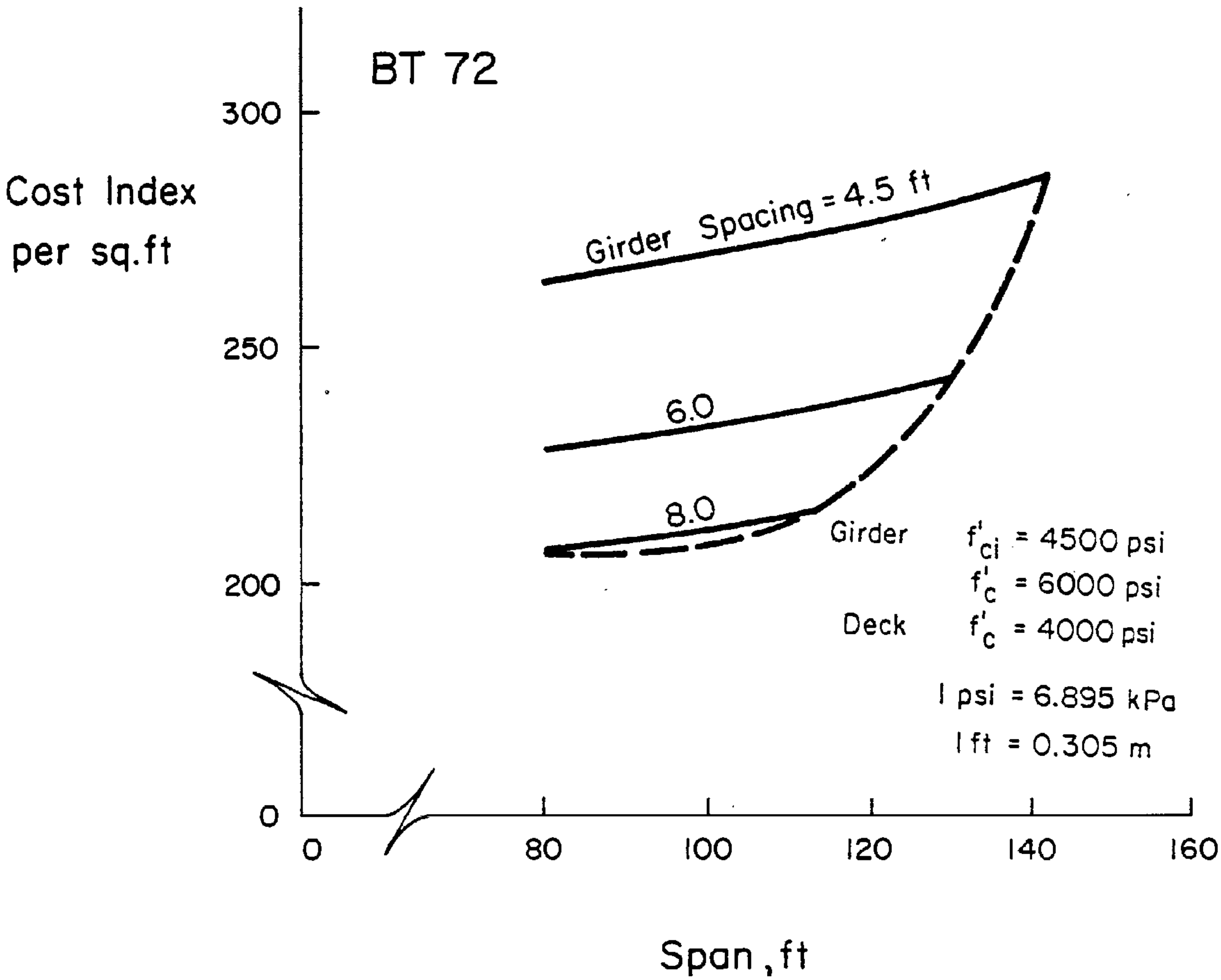

Figure 54 Cost Chart for BT72

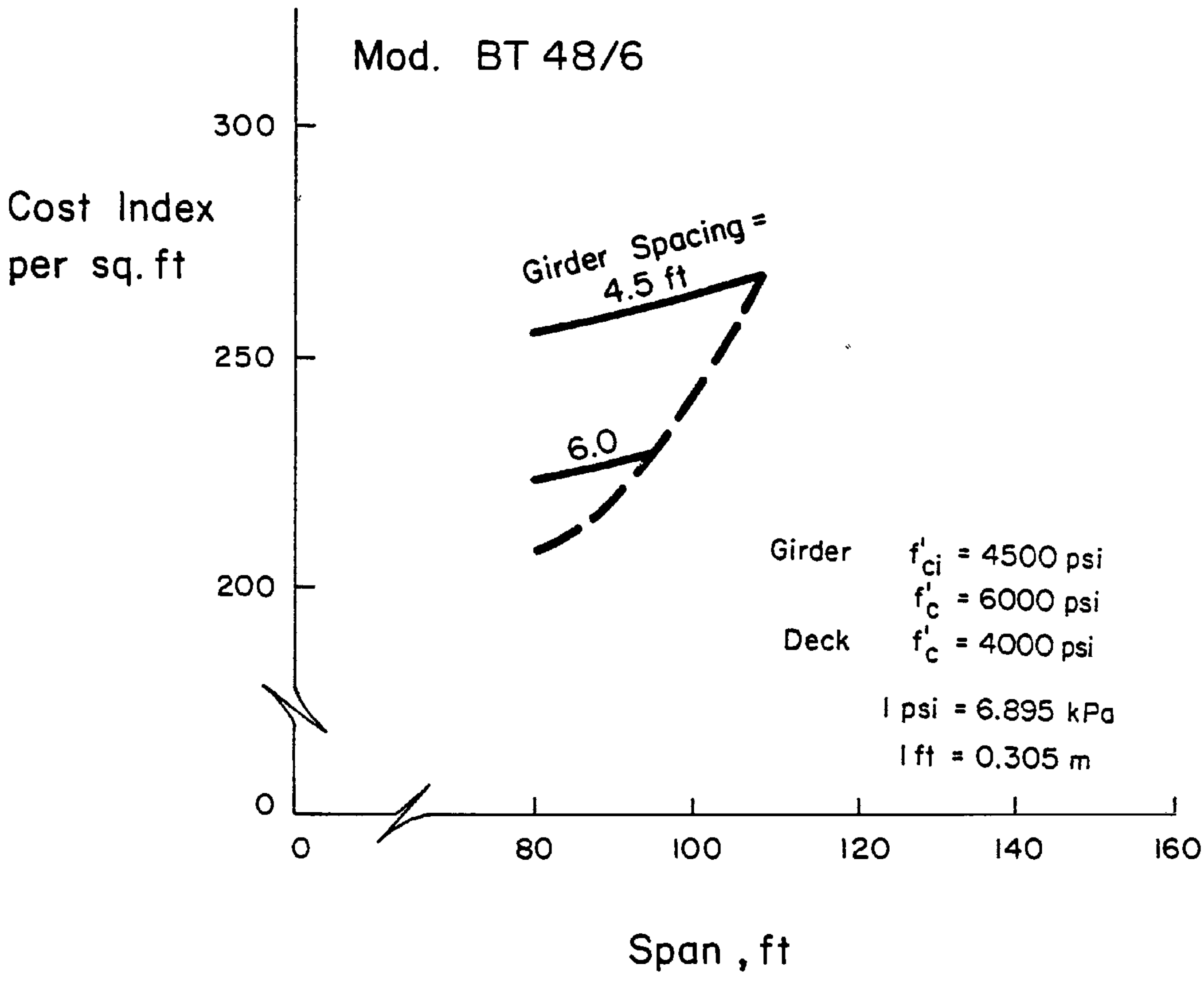

Figure 55 Cost Chart for Modified BT48/6

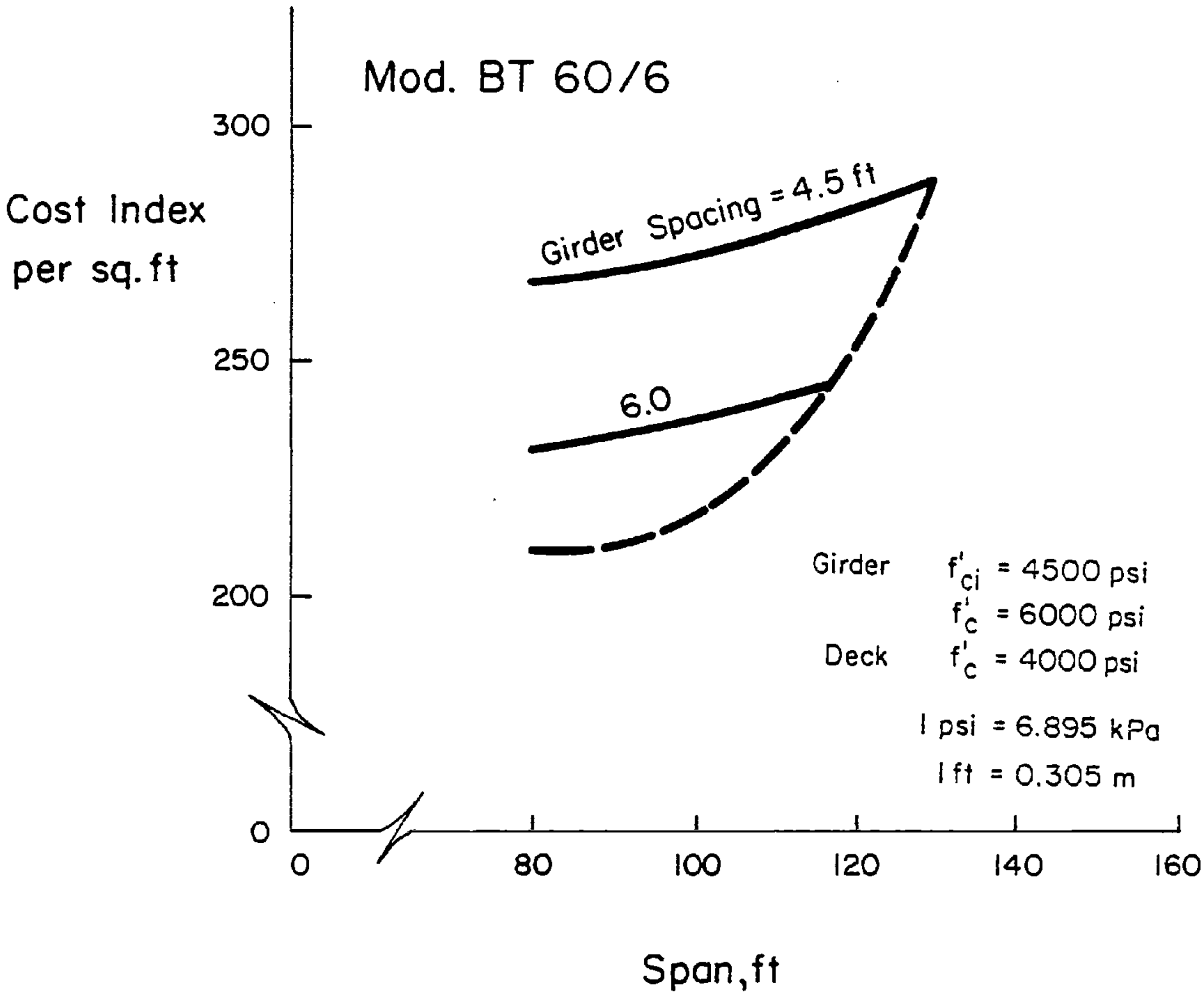

Figure 56 Cost Chart for Modified BT60/6

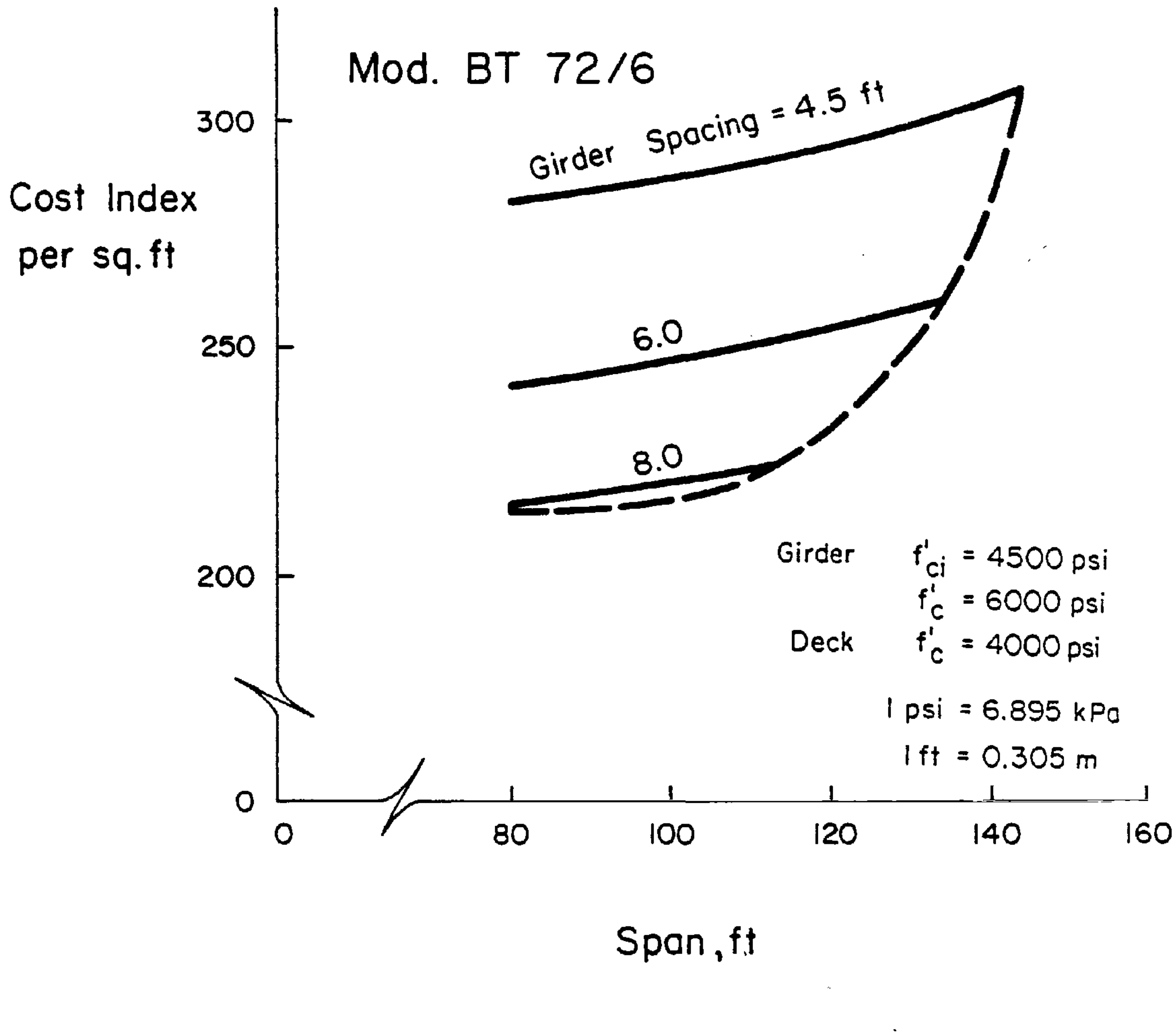

Figure 57 Cost Chart for Modified BT72/6

* U.S. GOVERNMENT PRINTING OFFICE: 1982 361-428/2022

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge and technology of materials properties, using available natural materials, improving structural foundation materials, recycling highway materials, converting industrial wastes into useful highway products, developing extender or substitute materials for those in short supply, and developing more rapid and reliable testing procedures. The goals are lower highway construction costs and extended maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highways at reasonable costs.

6. Improved Technology for Highway Construction

This category is concerned with the research, development, and implementation of highway construction technology to increase productivity, reduce energy consumption, conserve dwindling resources, and reduce costs while improving the quality and methods of construction.

7. Improved Technology for Highway Maintenance

This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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