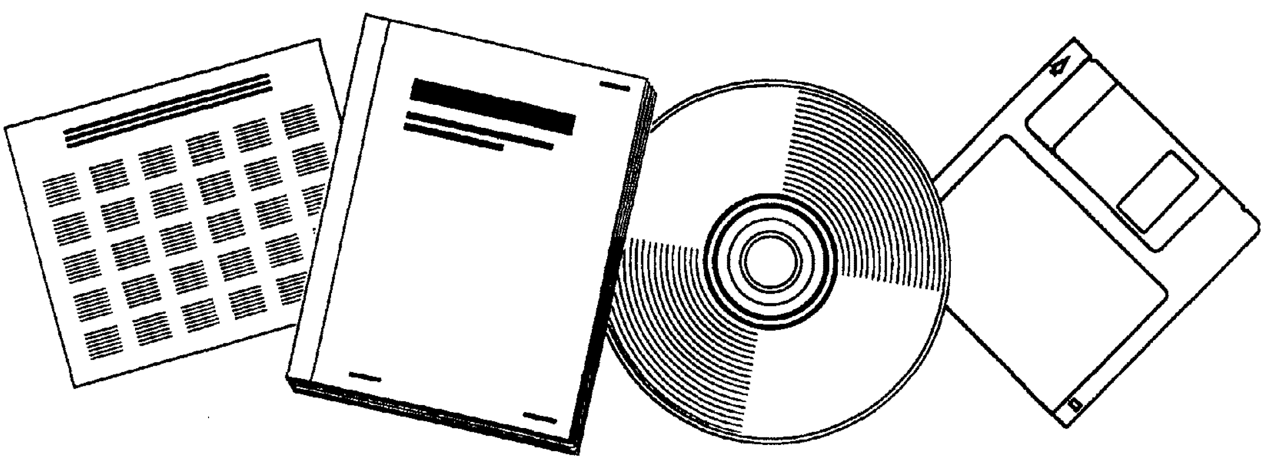
PB98-107071

OPTIMIZED SECTIONS FOR HIGH STRENGTH CONCRETE BRIDGE GIRDERS

AUGUST 1997



U.S. DEPARTMENT OF COMMERCE
National Technical Information Service

Optimized Sections for High-Strength Concrete Bridge Girders

PUBLICATION NO. FHWA-RD-95-180

PB98-107071

AUGUST 1997

U.S. Department of Transportation
Federal Highway Administration

Research and Development
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, VA 22101-2296

FOREWORD

This report, Volume I of a two-volume report on optimized sections for high-strength concrete bridge girders, presents the results of research conducted for the Federal Highway Administration (FHWA). This report will be of interest to bridge design engineers and structural research engineers because it investigates alternative bridge cross sections.

The report documents all phases of the research. It examines the feasibility of using high-performance concrete in correlation with existing and modified prestressed concrete girder cross sections to take advantage of the high-strength concretes. It also determines the factors that limit the application of high-strength concrete. The analysis indicated that the use of existing girder cross sections with concrete compressive strengths up to 10,000 psi (69 MPa) allows longer span lengths and more economical structures. The study concludes that at a minimum, all highway departments should adopt 8,000-psi (53-MPa) compressive strength concrete as the normal design strength for longer span girders.

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Director, Office of Engineering
Research and Development

NOTICE

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-RD-95-180		

4. Title and Subtitle	5. Report Date
OPTIMIZED SECTIONS FOR HIGH-STRENGTH CONCRETE BRIDGE GIRDERS	August 1997
	6. Performing Organization Code 480017
7. Author(s) H.G. Russell, J.S. Volz, and R.N. Bruce	8. Performing Organization Report No.
9. Performing Organization Name and Address Construction Technology Laboratories, Inc. 5420 Old Orchard Road Skokie, IL 60077	10. Work Unit No. (TRAIS) D9A 11. Contract or Grant No. DTFH61-93-X-00038
12. Sponsoring Agency Name and Address Office of Advanced Research, Federal Highway Administration 6300 Georgetown Pike McLean, Virginia 22101-2296	13. Type of Report and Period Covered Final Report July 1993 - June 1997
	14. Sponsoring Agency Code

15. Supplementary Notes

Agreement Officer's Technical Representative (AOTR): Susan Lane (HNR-10)

16. Abstract

The objectives of this investigation were: to identify the limitations of existing prestressed concrete girder cross sections relative to the use of high-strength concrete; to examine the feasibility of using modified cross sections to take advantage of the high-strength concretes; to investigate the use of alternative construction systems; and to define factors that limit the application of high-strength concrete.

The research was performed using the computer program "BRIDGE" to determine relative unit costs and maximum span lengths for different simple-span prestressed concrete bridge designs.

The analyses indicated that the use of existing girder cross sections with concrete compressive strengths up to 10,000 psi (69 MPa) allow longer span lengths and more economical structures. To effectively utilize higher strength concretes, additional prestressing force must be applied to the cross section in the form of smaller strand spacings, larger strand sizes, higher strength strands, or post-tensioning. The Bulb-Tee should continue to be considered as a national standard for span lengths from 80 to 200 ft (24 to 61 m). However, the Washington and Colorado sections are equivalent up to span lengths of 120 ft (37 m), and the Florida and University of Nebraska sections are slightly more economical for span lengths greater than 150 ft (46 m).

17. Key Words	18. Distribution Statement
Bridges, costs, girders, high-strength concrete, long spans, precast concrete, prestressed concrete, post-tensioned concrete.	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	164	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

Preface

For more than 25 years, concretes with specified compressive strengths in excess of 6,000 psi (41 MPa) have been used in the construction of columns of high-rise buildings. Initially, the availability of the high-strength concretes was limited to a few geographic locations. However, over the years, opportunities have developed to utilize these concretes at more locations across the United States. As the opportunities have developed, the materials producers and contractors have accepted the challenge to produce concretes with higher compressive strengths.

In the precast, prestressed concrete bridge field, a specified compressive strength of 6,000 psi (41 MPa) has been used for many years. However, strengths at release have often controlled the concrete mix design so that actual strengths at 28 days were often in excess of 6,000 psi (41 MPa). It is only in recent years that a strong interest in the utilization of concrete with higher compressive strengths has emerged. This interest has developed at a few geographic locations in a similar manner to the development in the building industry. Several research studies have addressed the application of high-strength concrete in bridge girders. These studies have suggested that there may be a limit at which the higher strength concretes can no longer be effectively utilized.

This report examines the use of high-strength concrete in precast, prestressed solid section girders. The objective of the research is to define the limits at which the utilization of higher strength concretes may no longer be structurally beneficial or cost-effective. The report then describes some solutions to overcome the limitations so that higher strength concretes can be effectively utilized in bridge construction.

This report contains several recommendations about the usage of high-strength concrete. Areas where research is needed are also identified. In some cases, the research must be performed before the recommendations are implemented. This is necessary so that the designer will have the engineering information available in order to design with the higher strength concretes.

The research described in this report was sponsored by the Federal Highway Administration, Office of Advanced Research. The office is responsible for the planning, administering, conducting, and coordinating of fundamental research and innovative adaptations for emerging and advanced technologies that have potential for long-range application in the highway program. The authors believe that high-strength concrete represents a technology with great potential for use in the highway program.

Acknowledgments

The authors would like to express their appreciation to the following individuals and organizations who provided information relative to this project:

Joseph S. Balik, W. R. Grace & Co.

W. Vincent Campbell, Bayshore Concrete Products Corporation

Reid W. Castrodale, Portland Cement Association

James E. Cook, Gifford-Hill, Inc.

Z. T. George, Texas Concrete Company

Howard W. Knapp, Rocky Mountain Prestress, Inc.

Robert P. McCrossen, Florida Wire and Cable, Inc.

Walter Podolny, Jr., Federal Highway Administration

Basile G. Rabbat, Portland Cement Association

Bradley K. Violetta, Master Builders Technology

Max J. Williams, Gulf Coast Pre-Stress, Inc.

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

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1. INTRODUCTION

BACKGROUND

Ever since the introduction of prestressed concrete into North America, the use of prestressed concrete has expanded as an efficient, economical, functional and versatile bridge construction material. In the early applications, designers developed their own ideas of which was the "best" girder cross section to use. As a result, each bridge utilized a different girder shape. Consequently, the reuse of girder formwork on subsequent contracts was not possible. As a result, girder shapes had to be standardized in the interest of improving economy of construction. This lead to the development of the standard American Association of State Highway and Transportation Officials - Prestressed Concrete Institute (AASHTO-PCI) sections for bridge girders. Types I through IV were developed in the late 1950s and Types V and VI were developed in the 1960s.

Adoption of the AASHTO standard bridge girders simplified design practice and lead to wider use of prestressed concrete for bridges. Standardization resulted in considerable cost savings in the construction of bridges. However, following the original adoption of the standard AASHTO-PCI shapes, a number of significant developments in the technology of prestressed concrete design and construction took place. As a result, individual States again developed their own standard sections for improved efficiency and economy. In 1980, the Federal Highway Administration (FHWA) initiated an investigation to identify new optimized sections for major prestressed concrete girders. The objective of the investigation was to determine which existing girder cross sections represented optimum designs that could be promoted as national or regional standards.

OPTIMIZED SECTIONS

The research on optimized sections was performed in two phases.[1,2] In Phase 1, information was collected throughout the United States on the types of girders being utilized. Advantages and disadvantages of the concepts were assessed. In Phase 2, the structural efficiency and cost-effectiveness of the best existing designs, as well as some modified ones, were evaluated relative to the efficiency and cost-effectiveness of the standard AASHTO sections. A parametric investigation was conducted to evaluate the effects of girder spacing, span length, concrete strength and deck thickness on relative costs. A computer program called BRIDGE was developed for use in the parametric studies.

Based on these studies, the most structurally efficient sections were the Bulb-Tee, Washington, and Colorado girders.[1,2] Based on the analysis for cost-effectiveness, the Bulb-Tee girder with a 6-in (152-mm) web was recommended for use as a national standard for precast, prestressed concrete bridge girders in the United States for span lengths from 80 to 140 ft (24 to 43 m). The recommended cross section is identified as the CTL Bulb-Tee in the current report.

Subsequently, the PCI Committee on Concrete Bridges developed a modified section for use as a national standard. The modifications resulted in a slightly heavier section that was easier to produce and handle. This cross section was subsequently adopted by several States and is identified as the PCI Bulb-Tee in the current report. Another version of the Bulb-Tee was also adopted by the Florida Department of Transportation.(3) A comparison of the cross sections is shown in figure 1.

In an investigation at the University of Nebraska, Lincoln, Geren and Tadros developed an optimized precast/prestressed bridge I-girder for use in continuous span bridges.(4) The primary thrust of their investigation was to develop a girder that could be post-tensioned. They proposed a series of girders with a web width of 175 mm. For pretensioned concrete applications only, they proposed that the web be reduced to 150 mm. Their proposed sections had depths ranging from 0.75 m to 2.4 m. Their structural optimization was achieved as follows:

1. Minimize girder depth to achieve a given span length.
2. Provide a bottom flange with a vertical thickness to accommodate two full rows of pretensioned strands and as wide as possible, but still fitting existing prestressing beds.
3. Provide a minimum web thickness to accommodate post-tensioning ducts.
4. Provide a wide top flange to reduce the effective span length of the deck in the transverse direction and at the same time minimize the top flange cross-sectional area.

The resultant cross section is shown in figure 1.

As a means of improving the aesthetic appearance of long span bridges and providing an economic design solution, the Texas Department of Transportation developed a new shape referred to as the Texas U-beam.(5) The U-beam is larger than any other precast prestressed concrete girder used in Texas, with a bottom flange width of 1400 mm and the ability to accommodate two or three horizontal layers of strands as well as strands in the webs. A total of 99 strands can be accommodated in the larger section. Strands are straight and are debonded in the end regions as required. Although other States have used U-beams, the Texas section is included in this report because it will be used with high-strength concrete in the Louetta Road Overpass project.(5)

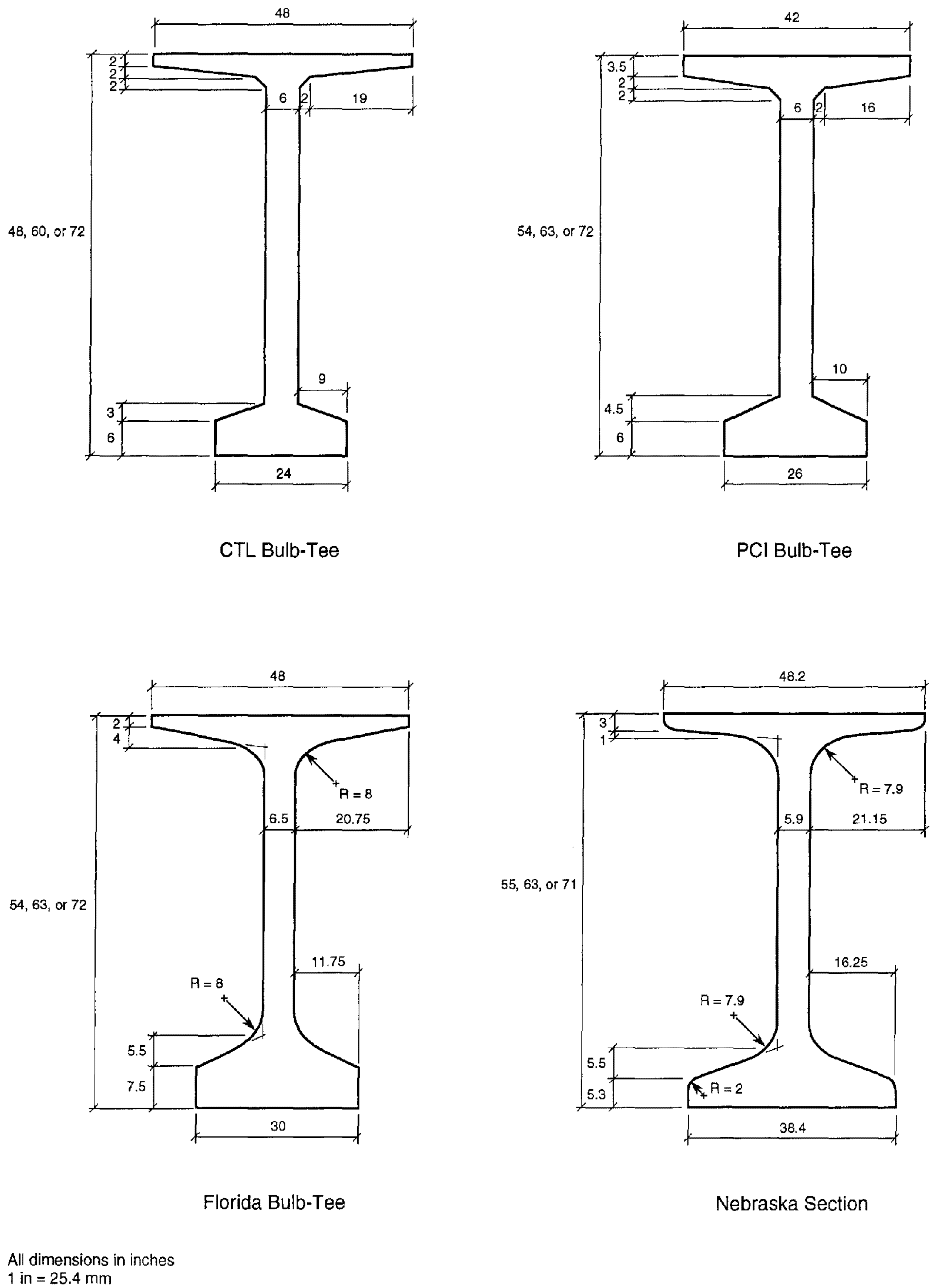


Figure 1. Comparison of Bulb-Tee Cross Sections.

HIGH-STRENGTH CONCRETE

In the previous CTL project on optimized sections, a limited investigation of the effect of concrete strength was made.(1,2) Comparisons showed that by increasing the girder concrete compressive strength from 5,000 to 7,000 psi (35 to 48 MPa), the maximum span capability of AASHTO girders was increased by about 15 percent.

In another project, the advantages of utilizing high-strength concrete in highway bridges were identified.(6) Results indicated that the span capabilities of various girder cross sections could be increased through the utilization of higher strength concretes. Alternatively, for the same span length, the number of girders in a cross section could be reduced by utilizing a higher strength concrete. Concretes with strengths ranging from 6,000 to 10,000 psi (41 to 69 MPa) were investigated. It was found that at the higher concrete strength levels, the maximum available prestressing force limited the advantages of high-strength concrete.

The use of high-strength concrete in long span, simply supported, precast, prestressed, concrete beams was investigated in a series of parametric studies by Zia et al.(7) Their investigations included normal-weight concretes with strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa) at 28 days with current standardized AASHTO girders, PCI Bulb-Tee girders and AASHTO box beams. Their results indicated that longer span lengths can be achieved with higher strength concretes. However, when the compressive strength was increased beyond a certain strength level, there was little or no benefit to be gained. The strength level at which the use of higher strength concrete was not beneficial varied between 8,000 and 12,000 psi (55 and 83 MPa), depending on the cross-sectional configuration and the prestressing force. It should be noted that the study did not consider concrete compressive strengths above 12,000 psi (83 MPa).

Zia also found that smaller sections with higher strength concretes could be used in place of larger sections with lower strength concretes. For example, the maximum span length of an AASHTO Type IV with 12,000-psi (83-MPa) concrete was found to be similar to that of an AASHTO Type VI with 6,000-psi (41-MPa) concrete. The benefits of using 0.6-in (15.2-mm) diameter strands with higher levels of concrete compressive strength were found to vary with member cross section. Increases in span length were possible when 0.6-in (15.2-mm) diameter strands were used in a modified PCI Bulb-Tee section with a 7-in (178-mm) thick web. In the studies of cost, Zia found that for all girder sections investigated, a girder spacing of 8 ft (2.4 m) was the most cost-effective design for about 60 percent of the span range of a girder section. For the longest span lengths, a 6-ft (1.8-m) spacing was the most cost-effective.

In a study at the University of Texas, Castrodale determined the maximum span lengths that could be used with different girder types, girder spacings and girder concrete compressive strengths.[8] He showed that maximum span lengths can be increased through the use of higher strength concretes. However, for all cross sections analyzed, the rate of increase in span length decreased as concrete compressive strength increased. Consequently, reduced benefits were achieved at the higher strength levels. Castrodale also proposed a new section with an efficiency close to that of the Bulb-Tee. This new section was suitable for longer span lengths, but even the new section showed limited benefits at the higher strength levels. Castrodale's study was limited to a concrete compressive strength less than 15,000 psi (103 MPa) and a maximum girder spacing of 10 ft (3.1 m). Cost comparisons were not included.

In work for the Louisiana Transportation Research Center, CTL and Tulane University evaluated the feasibility of using high-strength concrete in prestressed concrete girders.[9] An experimental program consisting of the construction and testing of full-size, prestressed concrete girders was performed. The girders utilized a 54-in (1370-mm) deep Bulb-Tee cross section with a 6-in (152-mm) thick web. Design concrete compressive strength at 28 days was 10,000 psi (69 MPa). The investigation concluded that structural members utilizing concrete with a compressive strength up to 10,000 psi (69 MPa) can be designed conservatively using the AASHTO Standard Specifications.[10] The program did not consider concrete with compressive strengths in excess of 10,000 psi (69 MPa).

Concretes with compressive strengths in excess of 10,000 psi (69 MPa) have been produced commercially utilizing ready-mixed concrete at many geographic locations around the United States. These concretes have been produced with a high degree of workability and pumpability. Concretes of these strengths have been produced in Illinois, New York, Ohio, South Carolina, Texas, and Washington. However, in bridge design, a design strength in excess of 6,000 psi (41 MPa) at 28 days is rarely utilized. Specific applications of concrete with strengths in excess of 6,000 psi (41 MPa) are East Huntington across the Ohio River, Annacis Bridge across the Fraser River in British Columbia, and the Toutle River Bridge in Washington State.[11] Concrete with a specified strength in excess of 10,000 psi (69 MPa) has rarely been utilized in a highway bridge structure. Consequently, there is a need to seek ways in which this material can be effectively utilized.

In addition to providing a higher compressive strength, high-strength concrete provides a higher modulus of elasticity, a higher tensile strength, reduced creep and greater durability. For the same cross section and span length, a high-strength concrete will result in less axial shortening and less short-term and long-term deflections. The higher tensile strength is advantageous where the allowable stress in tension controls the design. High tensile strength may be beneficial in reducing transfer length at the ends of girders. This is particularly important when larger diameter strands are used. In an investigation of transfer and development length, Cousins et al. found that increasing the concrete strength from 6,000 to 8,000 psi (41 to 55 MPa) up to 10,000 to 11,000 psi (69 to 76 MPa) significantly reduced transfer and development lengths.(12) They also found that reducing the spacing of 0.5-in (12.7-mm) diameter strand from 2 to 1.75 in (51 to 44 mm) did not have a significant effect on transfer length, development length or moment capacity. The reduced creep will result in less prestress losses, which can be beneficial in reducing the number of strands and reducing the change in camber. Improved durability, particularly when silica fume is used, can result in a longer life for bridge girders.

OBJECTIVES AND SCOPE

The objectives of the current research were as follows:

1. Identify the limitations of existing girder cross sections relative to the use of high-strength concrete in simple span structures.
2. Examine the feasibility of modified cross sections that can be used to take advantage of the higher strength concretes that are currently available.
3. Investigate the use of alternative construction systems that can be used with high-strength concrete.
4. Define existing factors that serve to limit the applications of high-strength concrete in bridge girders.

The objectives were accomplished with the following scope of activities:

1. Analyses of existing cross sections.
2. Analyses of modified cross sections and strand properties.
3. Analyses of post-tensioned girders.
4. Preparation of a report.

RESEARCH APPROACH

The majority of the research was performed using the computer program BRIDGE. BRIDGE was written as part of the previous investigation for the *Optimized Sections for Precast, Prestressed Bridge Girders* report.(1) The required input of BRIDGE consists of girder span, spacing and cross section; concrete and strand characteristics; and relative costs of materials. The program determines deck thickness and deck reinforcement, required number of prestressing strands, and cost index per unit surface area of bridge deck. The program also provides section properties, moments, stress levels, and deflections. Comparisons were made on the basis of relative costs for simply supported spans.

The computer program BRIDGE was originally written in Fortran IV with printed card input. For the present investigation, the program was modified to run on a personal computer with keyboard input. Several modifications to the program were necessary to extend the range of its applications. These modifications included:

1. Modification of the effective slab span to current AASHTO Specifications.(10)
2. Extension of the deck design table for effective slab spans greater than 10 ft (3.05 m).
3. Extension of the HS 20-44 Moments Table for span lengths up to 300 ft (91.5 m).

A complete description of the revised computer program is given in Appendix C.

2. ANALYSES OF EXISTING CROSS SECTIONS

In previous research, several cross sections were identified as having a high degree of structural efficiency and as being cost-effective.(1-8) These sections are:

1. CTL Bulb-Tee
2. PCI Bulb-Tee
3. Florida Bulb-Tee
4. AASHTO Type VI
5. Washington Series
6. Colorado Series
7. Texas Box U54
8. Nebraska Sections

Prior to performing the analyses for cost-effectiveness, the cross sections were compared on the basis of structural efficiency. With the exception of the Texas U-beam and the Florida Bulb-Tee, the web widths were taken as 6 in (152 mm). Cross-sectional dimensions of the above girders are given in Appendix A.

STRUCTURAL PARAMETERS

An efficiency factor for prestressed sections has been derived by Guyon.(13) It is based on minimizing the area of the section for a given section modulus. This efficiency factor ρ is defined as:

$$\rho = \frac{r^2}{y_t y_b} \qquad (1)$$

where

r = radius of gyration of section

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

The efficiency factors for the various sections listed above with respect to depth of section are plotted in figure 2. The efficiency factors are tabulated in Appendix A. Figure 2 indicates that the Nebraska sections and the three Bulb-Tees have the highest efficiency factor.

As reported in reference no. 1, Aswad has suggested another way of judging the efficiency of I-sections used in bridge superstructures. He proposed an efficiency ratio α defined as:

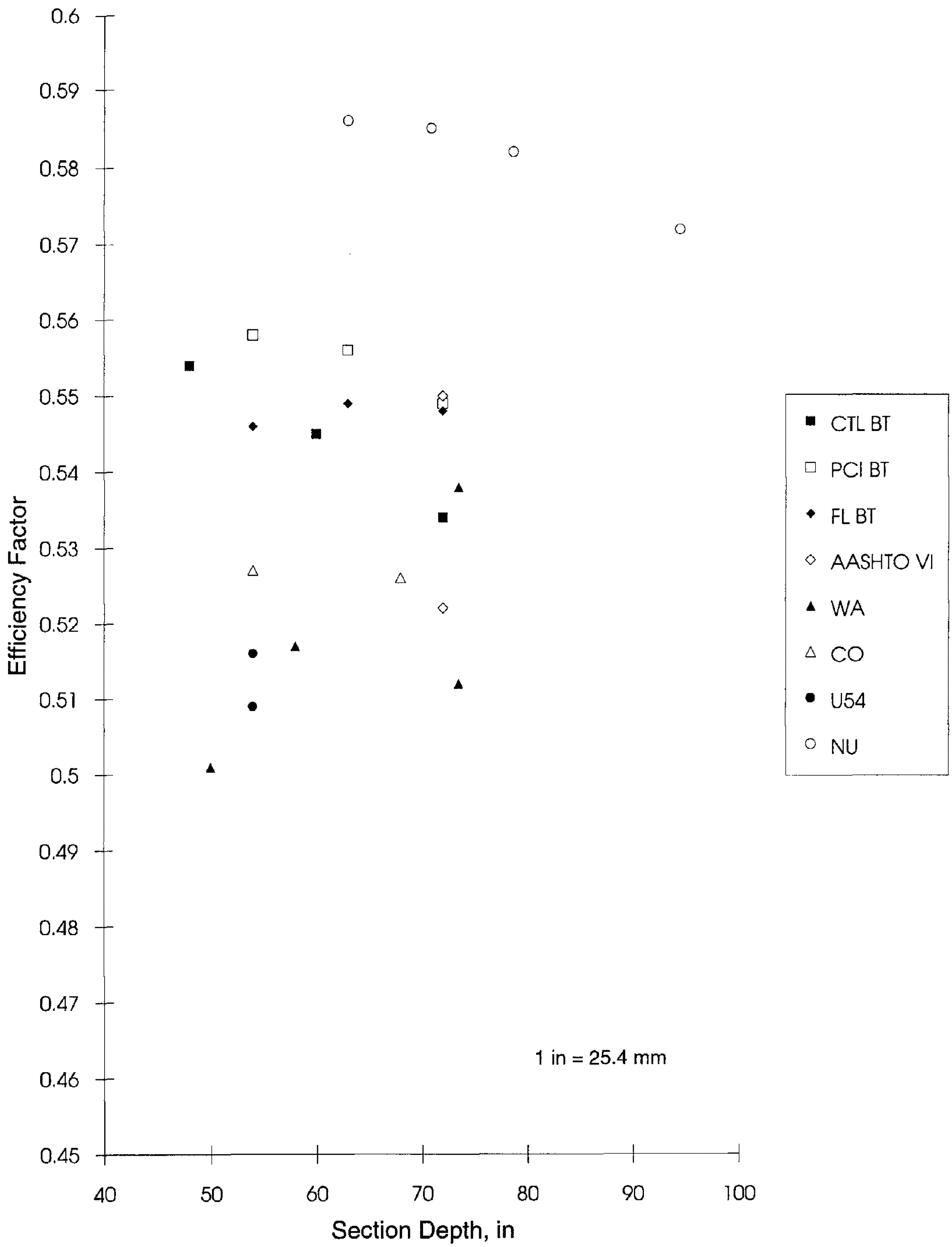


Figure 2. Variation of Efficiency Factor with Depth of Section.

$$\alpha = \frac{3.46\ S_b}{Ah} \qquad (2)$$

where

S_b = section modulus for bottom fibers

A = cross-sectional area

h = depth of section

Efficiency ratios for various sections described above are shown in figure 3 for different section depths. Although the Texas section is not strictly an I-section, it is included for comparison. According to the figure, the Nebraska sections, Texas U-beam and Florida Bulb-Tee sections have the highest degree of efficiency. However, the most structurally efficient section is not necessarily the most cost-effective.

The measures of efficiency factor and efficiency ratio confirmed previous conclusions that the area of the cross section should be concentrated in the two flanges as far as possible from the neutral axis, and the web should be made as thin as possible. Moreover, the haunch between the web and the flanges should be kept as horizontal as possible while still permitting placement of concrete and easy stripping of formwork. Based on previous studies, a minimum web thickness of 6 in (152 mm) still seems desirable for prestressed concrete girders and permits two rows of strands to be draped when the strand spacing is 2 in (51 mm) and the cover to the center of the strand is 2 in (51 mm).

CROSS SECTIONS ANALYZED

The following sections were selected for analysis of their cost-efficiency using the computer program BRIDGE:

1. CTL Bulb-Tee BT-72, identified as CTL BT-72.
2. PCI Bulb-Tee BT-72 and BT-54, identified as BT-72 and BT-54, respectively.
3. Florida Bulb-Tee BT-72, identified as FL BT-72.
4. AASHTO Type VI with a 6-in (152-mm) thick web, identified as Type VI.
5. Washington Series 14/6, which is similar to a Washington Series 14, but with a 6-in (152-mm) thick web; identified as WA 14/6.
6. Colorado Series G68/6, which is a Colorado G68, but with a 6-in (152-mm) thick web; identified as CO G68/6.

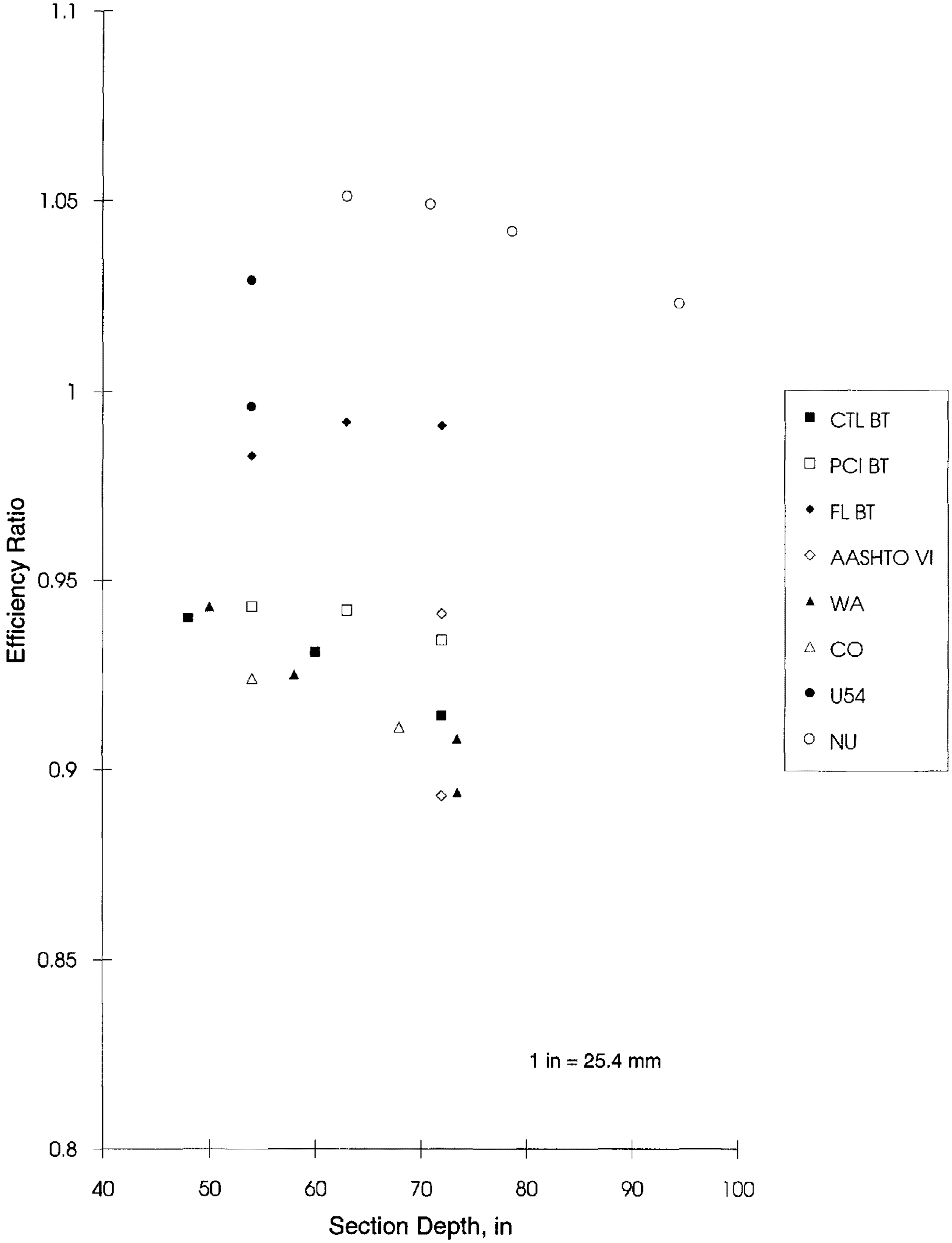


Figure 3. Variation of Efficiency Ratio with Depth of Section.

7. Texas U-beam U54B, which is the Texas section with a bottom flange of sufficient thickness to accommodate three rows of strands. This girder is identified as U54B.

8. Nebraska section with a depth of 1800 mm and a web thickness of 150 mm; identified as NU 1800.

Except for the BT-54, the girder depths were selected on the basis of suitability for similar span lengths. The BT-54 was selected for comparison with the U54B. Dimensions of all sections are shown in figure 4.

The following parameters were considered in the BRIDGE program:

1. Girder spacing. No maximum spacing was placed on the girders. Minimum spacing considered was that which corresponded to the flanges of the two girders touching each other.

2. Span length. Spans in excess of 80 ft (24.4 m) were considered.

3. Deck thickness. Deck thickness varied with girder spacing according to a predetermined design.

4. Concrete strength. Concrete strength of the girders at 28 days was varied from 6,000 psi (41 MPa) upward in increments of 2,000 psi (14 MPa) with no upper limit. Release strength was taken as 75 percent of the 28-day strength.

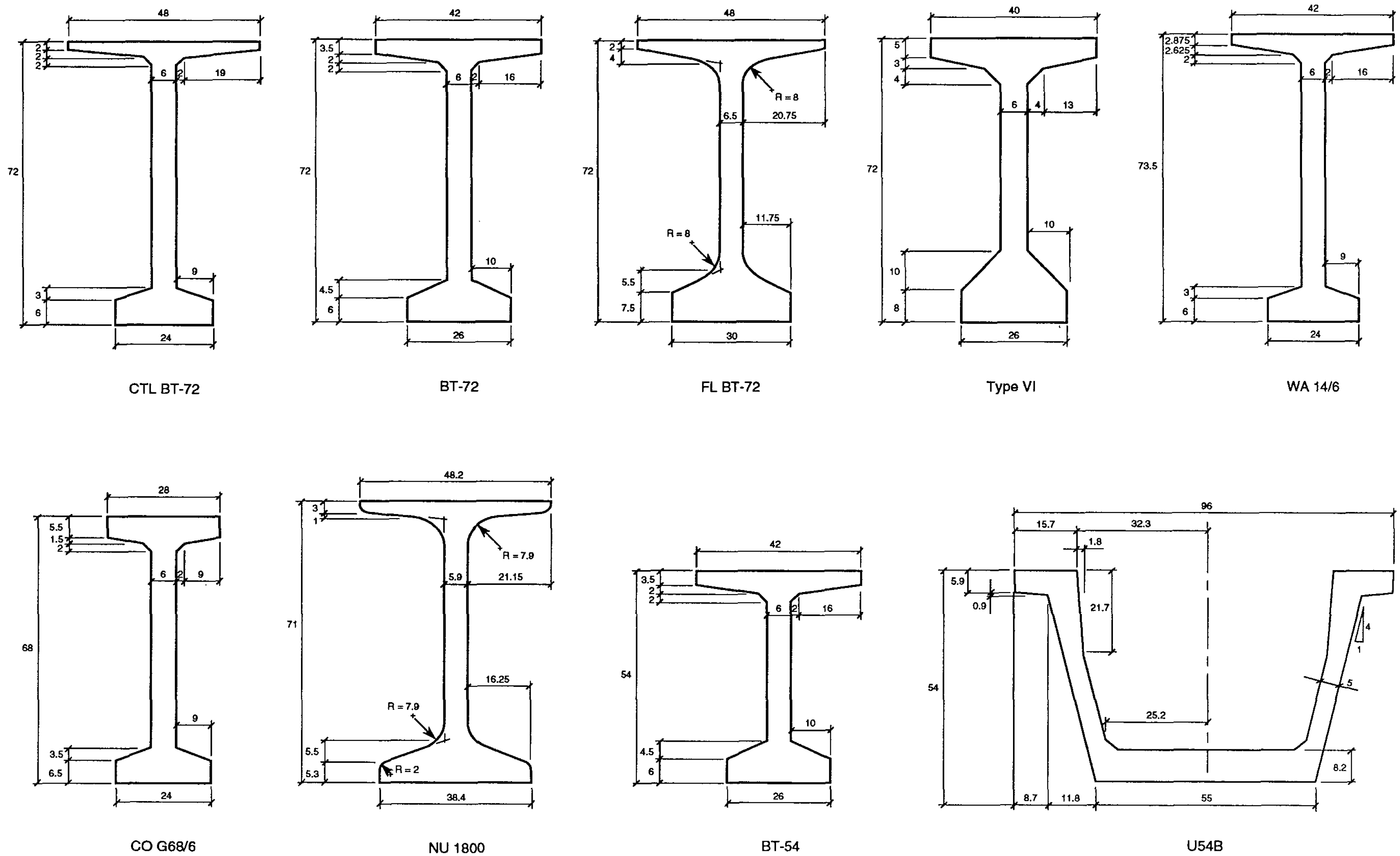
For the purposes of making the cost comparisons for different sections, the relative unit costs for in-place materials were taken as being the same as in the previous report.(1) The effects of premium costs for high-strength concrete are discussed in a later section of this report.

Concrete (girders and deck)	1 unit/unit weight of concrete
Strands	8 units/unit weight of strands
Reinforcing Steel	9 units/unit weight of reinforcing
Epoxy-Coated Reinforcing Steel	12 units/unit weight of epoxy-coated reinforcing

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

The following default assumptions were made in the BRIDGE program:

1. Design conforms to AASHTO Specifications.(10)



All dimensions in inches
1 in = 25.4 mm

Figure 4. Cross Sections of Girders Analyzed.

2. Live load consists of HS 20-44 loading.
3. Girders are simply supported.
4. Design is based on a typical interior girder.
5. Concrete deck is cast-in-place and acts compositely with the girder. Deck formwork is supported on the girder. The transformed area of strands was neglected.
6. Concrete compressive strength of the deck is constant and equal to 4,000 psi (28 MPa) at 28 days. If the compressive strength of the concrete in the deck limited the design, the strength was increased.
7. Strands are Grade 270 with a 0.5-in (12.7-mm) diameter and have an idealized trilinear stress-strain curve.
8. Strands are spaced at 2-in (51-mm) centers with a minimum 2-in (51-mm) concrete surface to center of the strand spacing.
9. Total prestress losses are constant and equal 45,000 psi (310 MPa). However, it is possible that with higher strength concretes, the prestress losses may be lower. This would have a beneficial effect in reducing the number of strands.
10. Relative unit costs of materials and labor are constant for each cost analysis. The effect of increased costs for higher strength materials is investigated in a separate phase of the project.
11. Cost analysis comparisons are for the precast girder and the cast-in-place deck only. Costs of substructure and approach fills are not considered.
12. Design is based on flexural strength at midspan. It is assumed that the compressive and tensile stresses that would develop at the ends of the girders if all strands were straight can be handled by the draping of strands, by additional top strands at the ends of the girders or debonding some strands at the ends of the girders. Selected girder designs were checked for shear and found to have adequate strength based on existing design requirements.

It is recognized that shipping lengths, girder weights, lateral stability of girders, prestressing bed capacities that exist today, and plant capabilities to produce high-strength concretes could limit the type of girders that can be produced. However, these limitations were not used as a means to restrict potential applications. The intent of the project was to look beyond current production capabilities.

OPTIMUM COST INDEX CHARTS

The computer program BRIDGE was used to perform cost-efficiency analyses of the various cross sections. As shown in figure 5, the cost index per unit surface area of the bridge deck can be plotted versus span length for a given cross section. At various girder spacings, different cost curves result, as shown by the solid lines in figure 5.

An "optimum cost curve" is obtained if the end points of each individual cost curve are joined, as shown by the dashed line in figure 5. This optimum cost curve indicates the least cost index for a particular span and varies as a function of girder spacing. As shown in figure 5 and as discussed by Rabbat and Russell, for a given span, cost index per square foot of bridge deck decreases as girder spacing increases.(1)

Optimum cost curves are generated for a constant girder concrete strength. The cost chart in figure 5 is for a 28-day girder concrete strength of 6,000 psi (41 MPa). Additional optimum cost curves can be generated at other girder concrete strengths for the same girder cross section. Figure 6 is a plot of the optimum cost curves for a BT-72 at 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa).

Figure 6 illustrates the benefits and limitations of higher strength concrete for existing cross sections of precast, prestressed bridge girders. Although figure 6 represents one particular cross section (BT-72), the results and relationships are consistent with other sections analyzed within this investigation and will be used as a basis for discussion. To examine the benefits and limitations, the curves must be studied at three separate locations.

The first location is for spans less than 90 ft (27.4 m). For these spans, the higher concrete strength would allow more prestressing and, therefore, greater girder spacings and a resultant reduction in the cost index. However, the controlling condition for these spans is initial prestress transfer. For a given span, there is a point at which additional prestressing will cause tension in the top fibers regardless of the concrete strength. Although this tension would be offset in the service load condition, the dead load at prestress transfer is constant for a given span and cross section, and independent of the final in-place girder spacing. As a result, there is no benefit realized by using concrete compressive strengths greater than 6,000 psi (41 MPa) at these span lengths.

The second location is for spans between 90 and 100 ft (27.4 and 30.5 m) for all strengths of concrete and spans between 90 and 110 ft (27.4 and 33.5 m) for strengths of 8,000 psi (55 MPa) and greater. As previously discussed, Rabbat and Russell found that for a given span, cost index per square foot of bridge deck decreases as girder spacing increases.(1) With the use of higher strength concrete, additional prestressing will allow larger girder spacings for

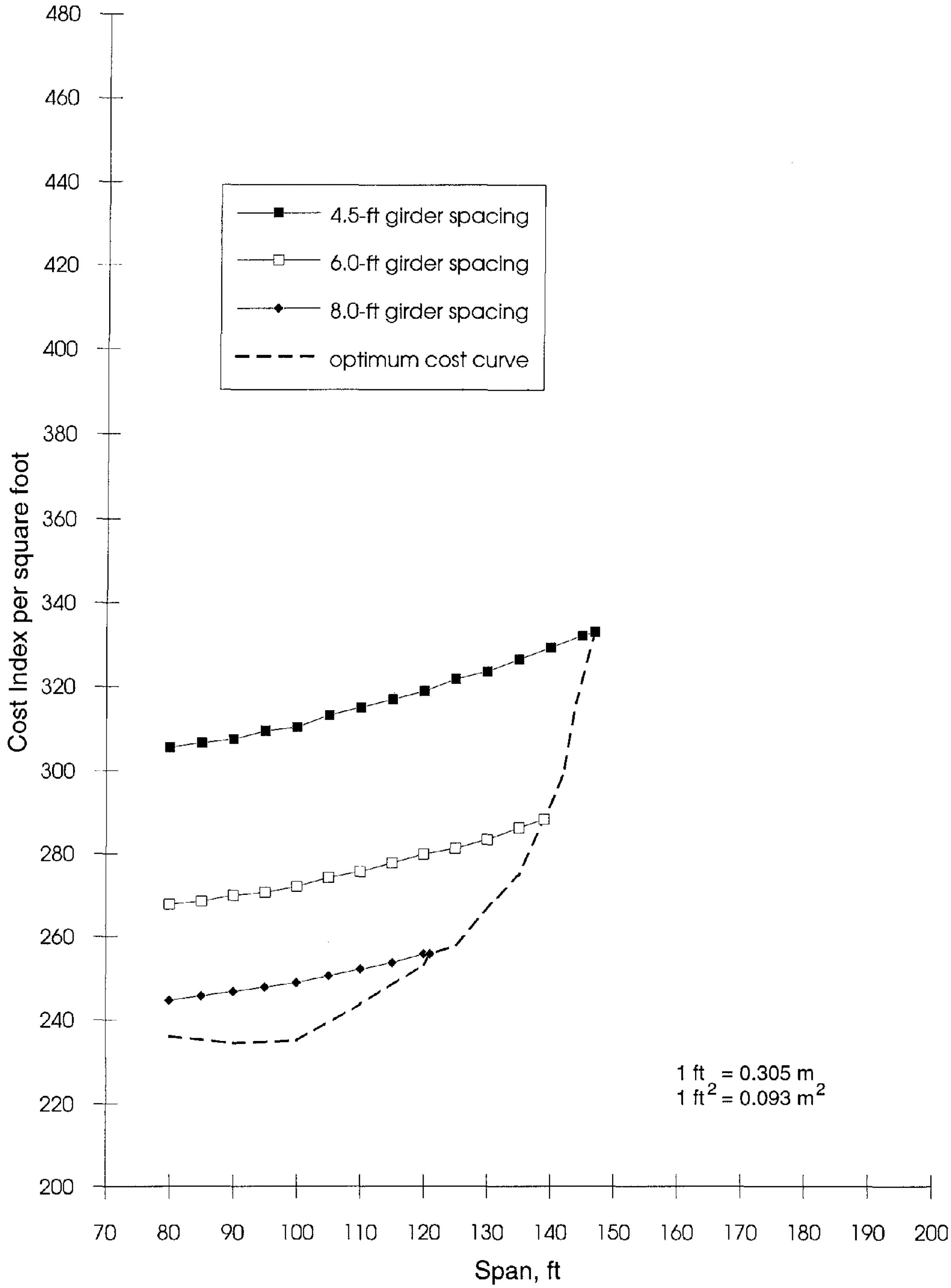


Figure 5. Cost Chart for a BT-72 at 6,000 psi (41 MPa).

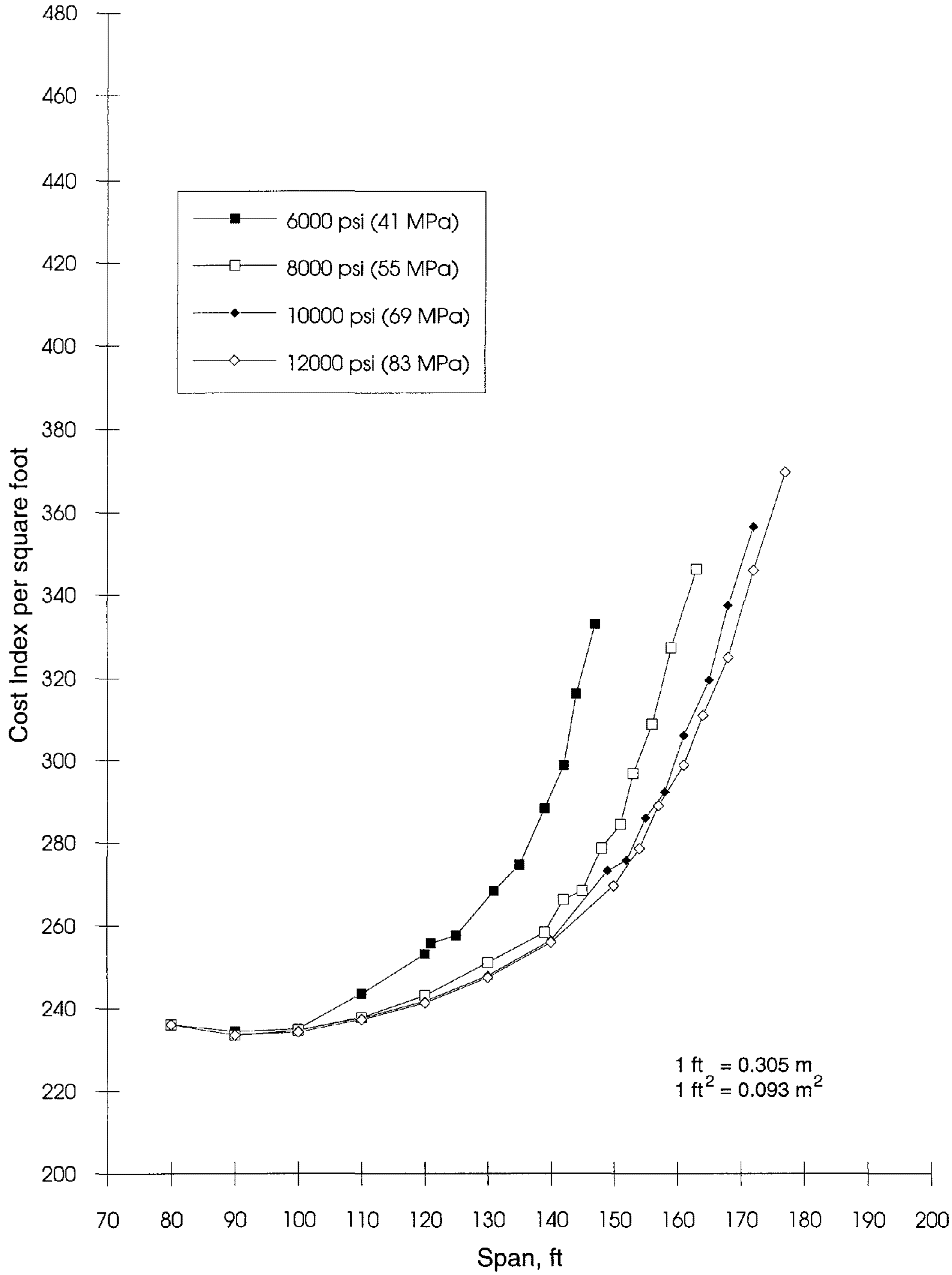


Figure 6. Optimum Cost Curves for a BT-72.

a given cross section and span length. However, there is a point at which unit deck costs begin to offset the savings in unit girder costs associated with larger spacings. This fact is illustrated in figures 7 and 8 for the BT-72.

Both figures are plots of cost index versus girder spacing for a given span length. The small jumps in each curve are associated with changes in deck thickness or reinforcement. As can be seen in figure 7, the cost index decreases as girder spacing increases until the capacity of the section is reached. Figure 8, on the other hand, shows that the savings taper off at about 10-ft (3.0-m) spacing and costs actually increase slightly before the capacity of the section is reached. Therefore, benefits of higher strength concrete are not realized since larger girder spacings do not result in a lower cost of the bridge superstructure. In general, the optimum girder spacings for the sections analyzed ranged between 9 and 14 ft (2.7 and 4.3 m) with larger spacings being more applicable with the higher strength concretes in the girders.

The third location to be examined in figure 6 is for spans exceeding 100 ft (30.5 m) when concrete strengths are between 6,000 and 8,000 psi (41 and 55 MPa), and spans exceeding 110 ft (33.5 m) when concrete strengths exceed 8,000 psi (55 MPa). These areas represent the optimization of benefits of high-strength concrete for the cross sections analyzed. The higher strength concrete allows larger prestressing and, as a result, greater girder spacings for a given span, thus reducing unit cost. For these span lengths, the original conclusion of Rabbat and Russell is confirmed: for a given span, cost index per square foot of bridge deck decreases as girder spacing increases.(1)

At these longer span lengths, the optimum girder spacings that result in the lowest possible cost for a given cross section are not reached. In other words, the cost index as a function of girder spacing is still decreasing when the girder capacity is reached and does not flatten out as shown in figure 8. For example, at a span length of 140 ft (42.7 m) with a 6,000-psi (41-MPa) girder, the maximum spacing is 5.9 ft (1.8 m), while an 8,000-psi (55-MPa) girder can be spaced at 8.3 ft (2.5 m). Although the deck costs will be greater for the 8,000-psi (55-MPa) girder, the savings in girder costs far outweigh increased deck costs and result in a more cost-effective superstructure.

Figure 6 also indicates that cost benefits vary as a function of span length and girder concrete strength. For example, an 8,000-psi (55-MPa) girder has a 3-percent lower cost index than a corresponding 6,000-psi (41-MPa) girder at a span length of 110 ft (33.5 m), but a 10-percent lower unit cost at a span length of 140 ft (42.7 m). These cost benefits continue to increase as the span length increases, reaching a maximum of 18 percent at a span length of 147 ft (44.8 m). At this point, the lower strength girder has reached its maximum span length, while

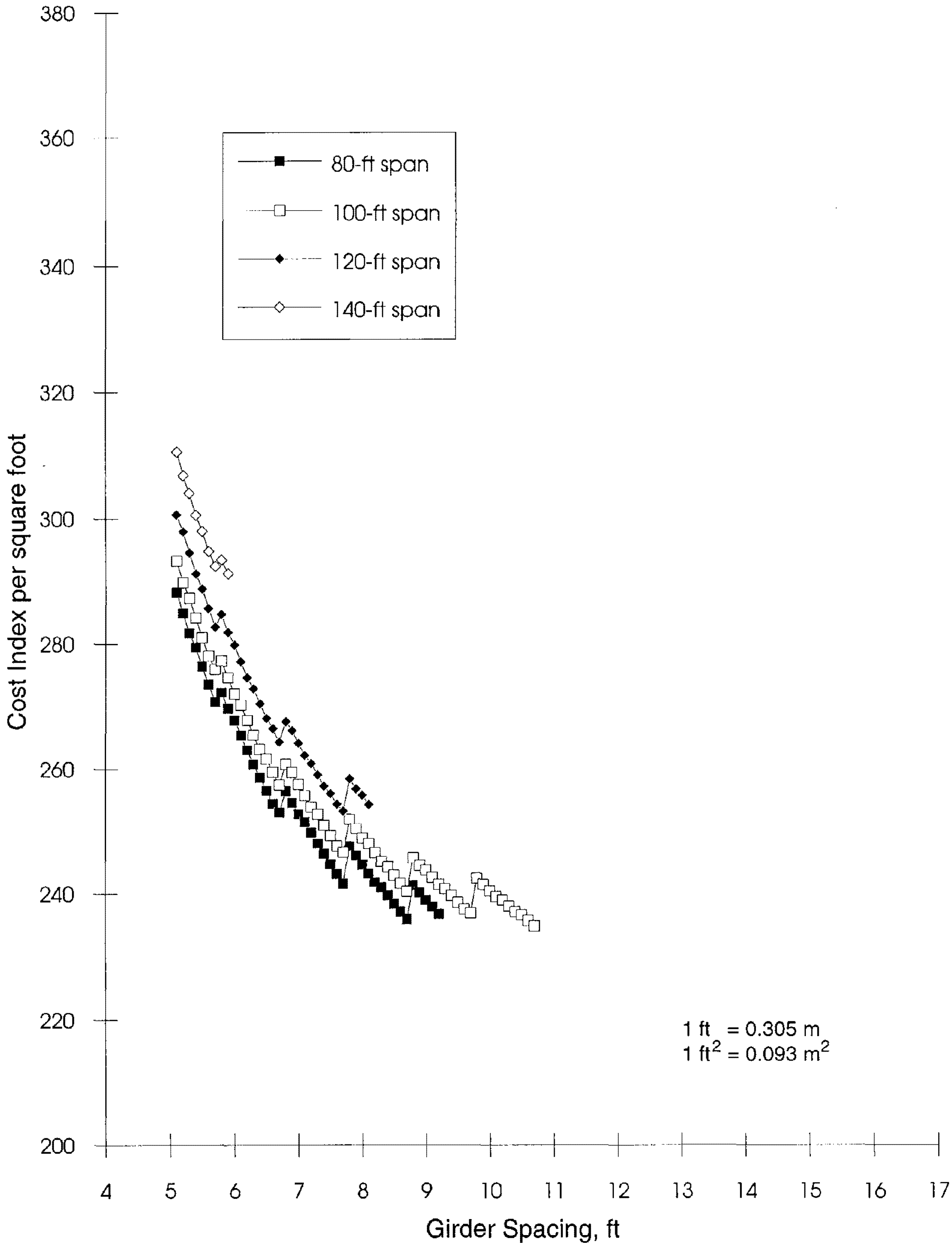


Figure 7. Comparison of Cost Curves for Varying Span and Spacing, BT-72 at 6,000 psi (41 MPa).

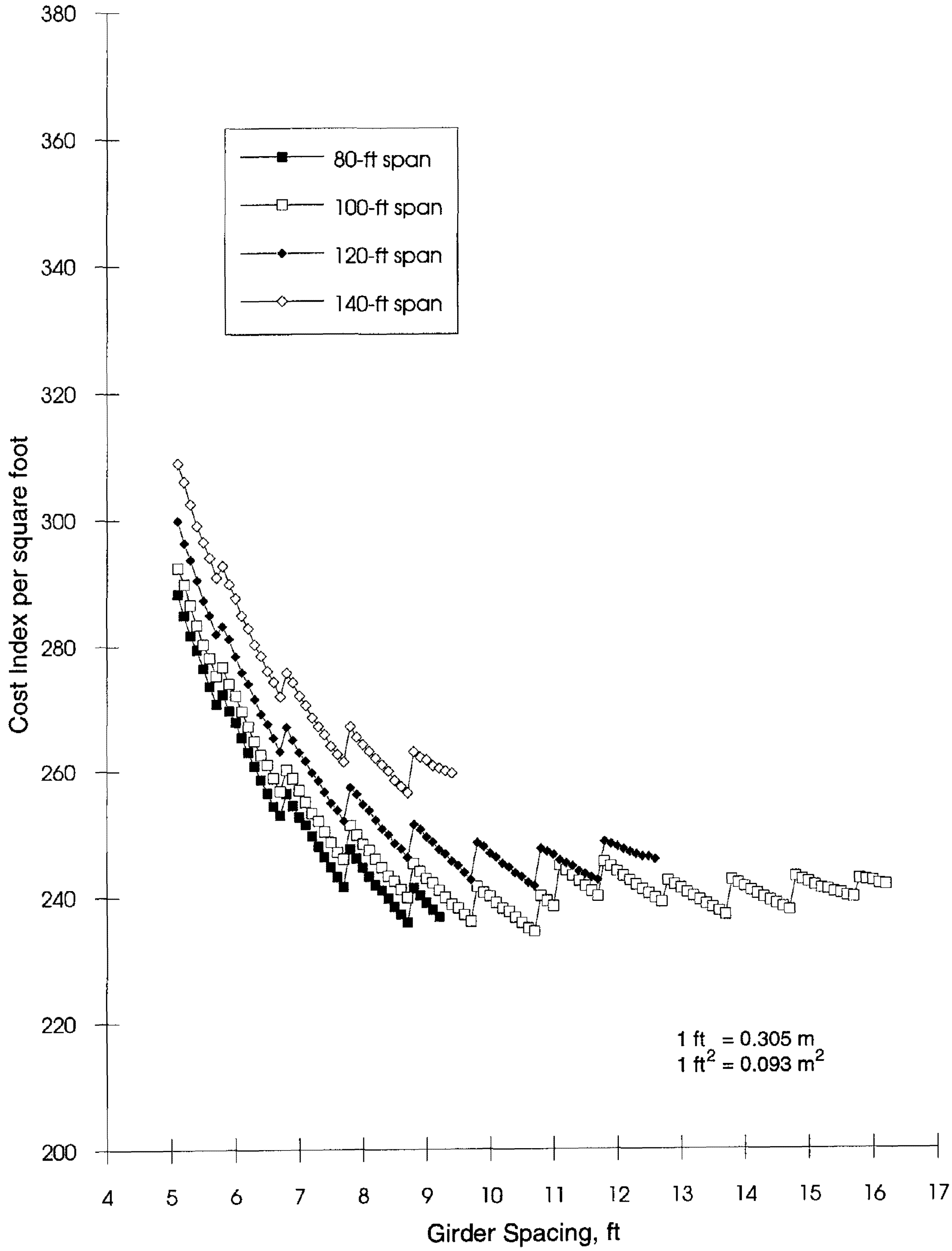


Figure 8. Comparison of Cost Curves for Varying Span and Spacing, BT-72 at 10,000 psi (69 MPa).

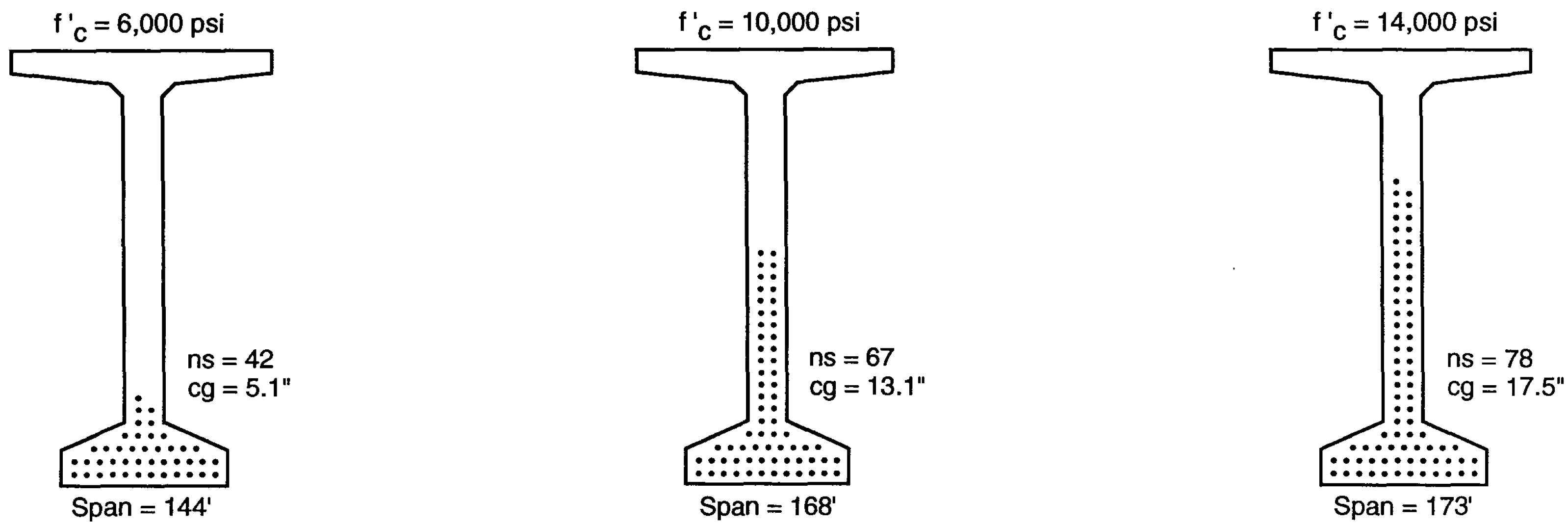
the higher strength girder still has additional capacity. In other words, another benefit of high-strength concrete is the ability to achieve greater span lengths.

Figure 6 also indicates another important point: the diminishing returns realized with the use of high-strength concrete for existing cross sections. The shift in the optimum cost curve decreases for each succeeding 2,000-psi (14-MPa) increase in girder compressive strength. For example, at a girder spacing of 5 ft (1.5 m), the maximum span length increases by 15 ft (4.6 m) when girder compressive strength is increased from 6,000 to 8,000 psi (41 to 55 MPa); however, the maximum span length increases by only 9 ft (2.7 m) when girder compressive strength is increased from 8,000 to 10,000 psi (55 to 69 MPa). Furthermore, the span length increases fall off dramatically at girder compressive strengths exceeding 10,000 psi (69 MPa).

The primary cause of these diminishing returns is the decreasing strand eccentricity. Figure 9 illustrates the inefficient strand layouts that result for increasing girder compressive strengths. Once strands are placed within the web, the efficiency of a particular section begins to decrease rapidly. The incremental benefit of each succeeding strand decreases when sufficient room within the flange does not exist. This result is evident in the stresses that occur at prestress transfer. The ratio of top stress to bottom stress at midspan decreases from 1:2.87 at 6,000 psi (41 MPa) to 1:1.65 at 10,000 psi (69 MPa) and, eventually, to 1:1.24 at 14,000 psi (97 MPa). This almost uniform stress distribution at high girder concrete strengths is extremely inefficient. As a result, the bottom fiber stress at midspan under service load controls the design at both 10,000 and 14,000 psi (69 and 97 MPa); however, at 6,000 psi (41 MPa), both top and bottom stresses at midspan control the design and the result is an efficient use of the material strengths available. Since additional prestressing force cannot be induced in the girder, the beneficial effects are limited to the increase in concrete tensile strength, which only increases as the square root of compressive strength.[11]

A secondary cause that contributes to the diminishing returns is the deck concrete compressive strength. In calculating the composite section properties and service load stresses, the BRIDGE program employs a transformed girder/deck section. As girder strength increases and deck strength remains constant, the composite section properties decrease, with a corresponding increase in service load stresses for the same span and girder spacing. However, as shown in figure 9, the decrease in composite moment of inertia for a 14,000-psi (97-MPa) girder versus a 6,000-psi (41-MPa) girder is only 10 percent, and the corresponding stress increase applies only to the live load portion of the loading.

BT-72 Girder Design, 5-ft (1.5-m) Spacing, Maximum Span, 4000-psi (28-MPa) Deck Concrete
(ns = number of prestressing strands, cg = centroid of strands to bottom fiber)



f'c = 6,000 psi	f'c = 10,000 psi	f'c = 14,000 psi
STRESSES AT TRANSFER*:	STRESSES AT TRANSFER*:	STRESSES AT TRANSFER*:
Top = 825 psi (0)	Top = 1913 psi (0)	Top = 2633 psi (0)
Bottom = 2368 psi (2700 psi)	Bottom = 3160 psi (4500 psi)	Bottom = 3259 psi (6300 psi)
Ratio = 1:2.87	Ratio = 1:1.64	Ratio = 1:1.24
STRESSES AT SERVICE LOAD*:	STRESSES AT SERVICE LOAD*:	STRESSES AT SERVICE LOAD*:
Top = 2376 psi (2400 psi)	Top = 3887 psi (4000 psi)	Top = 4638 psi (5600 psi)
Bottom = -433 psi (-465 psi)	Bottom = -591 psi (-600 psi)	Bottom = -706 psi (-710 psi)
$I_{composite}$ = 881,365 in^4	$I_{composite}$ = 823,718 in^4	$I_{composite}$ = 789,672 in^4

NOTES: * "( )" indicates AASHTO Specification allowable stress, "-" indicates tension

1 in = 25.4 mm
1 ft = 0.305 m
1000 psi = 6.89 MPa

Figure 9. BT-72 Girder Design at Midspan for Various Concrete Strengths.

In general, increases in the girder concrete strength result in the following:

1. A shift in the optimum cost curve to the right for each succeeding increase in girder concrete strength.
2. Decreasing incremental benefits for each incremental increase in concrete strength.
3. Minimal benefits beyond a girder concrete strength of about 10,000 psi (69 MPa).
4. No benefit from higher concrete strength for the horizontal portion (shorter span lengths) of the optimum cost curve.

COMPARISON OF CROSS SECTIONS

General

As discussed in the previous section, a cost-efficiency analysis yields an "optimum cost curve" for a particular cross section. This curve indicates the least cost index for a particular span and varies as a function of girder spacing. Each curve is generated for a constant girder concrete strength; however, a series of curves can be generated for a particular section based on various girder concrete strengths as shown in figure 6.

Optimum cost curves allow variables to be compared on the basis of cost-effectiveness. The previous section contained a discussion of the effect of high-strength concrete on the optimum cost curve. This discussion highlighted some of the general benefits and limitations of high-strength concrete in precast, prestressed bridge girder design. The following section, on the other hand, will compare various specific sections in order to identify which characteristics of existing girder cross sections are more advantageous with the use of high-strength concrete. Cost charts for each of the sections analyzed are included in Appendix D.

Comparison No. 1 - BT-54 and BT-72

Figure 10 displays the optimum cost curves for a BT-54 at concrete strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa). The behavior is very similar to that shown in figure 6 for a 72-in (1830-mm) deep version of the same cross section. It is noted that since this study begins at a span length of 80 ft (24.4 m), a portion of the horizontal segment of the optimum cost curve is omitted for the BT-54 section.

A comparison of figures 6 and 10 reveals one of the benefits of high-strength concrete with existing precast, prestressed bridge girders: shallower sections with higher strength concretes can be used in place of deeper sections with lower strength concretes. This conclusion is more evident in figure 11, which shows the 6,000-psi (41-MPa) optimum cost curve for a BT-72 superimposed on figure 10. As can be seen in this figure, up to a span length of approximately

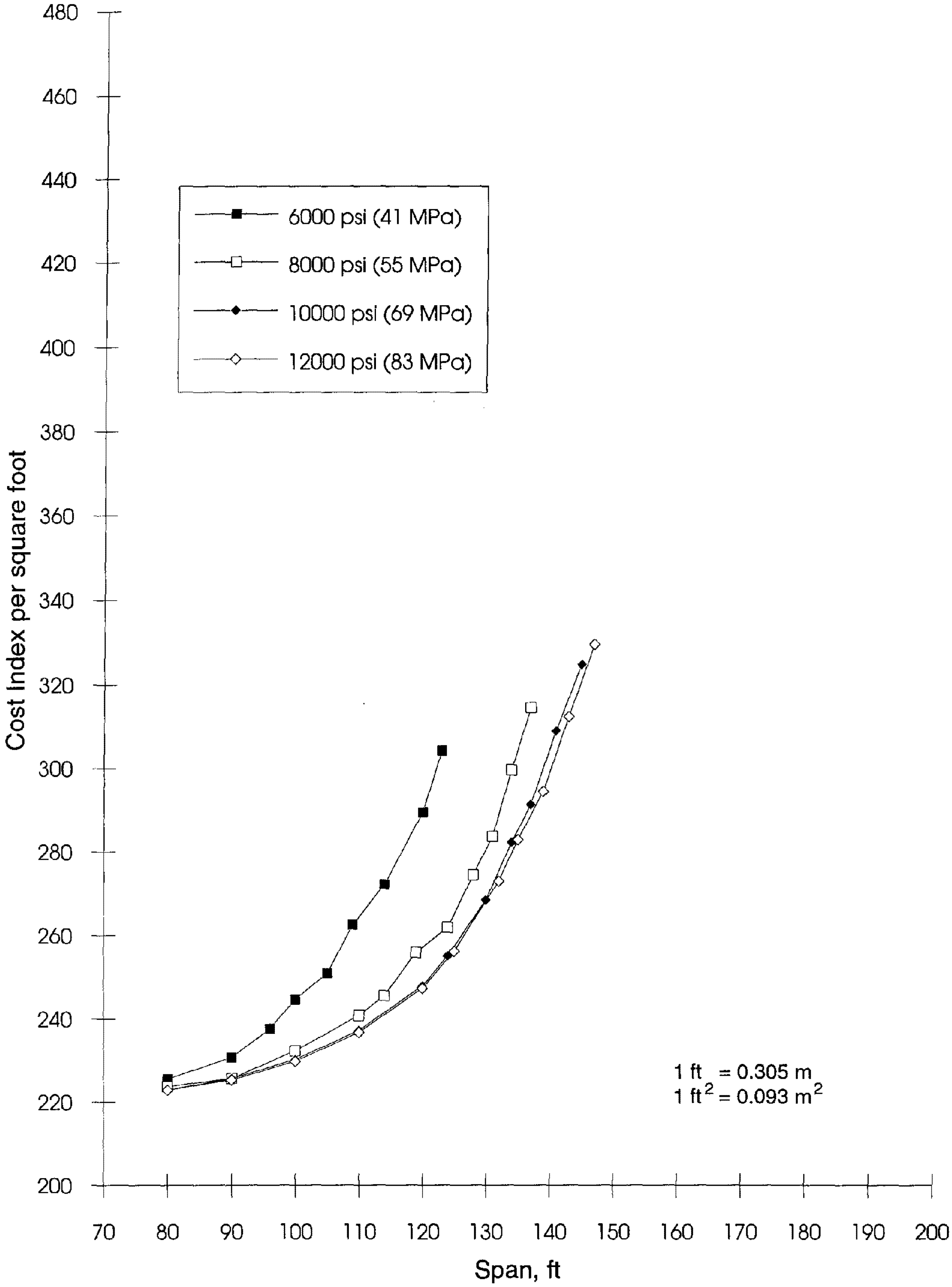


Figure 10. Optimum Cost Curves for a BT-54.

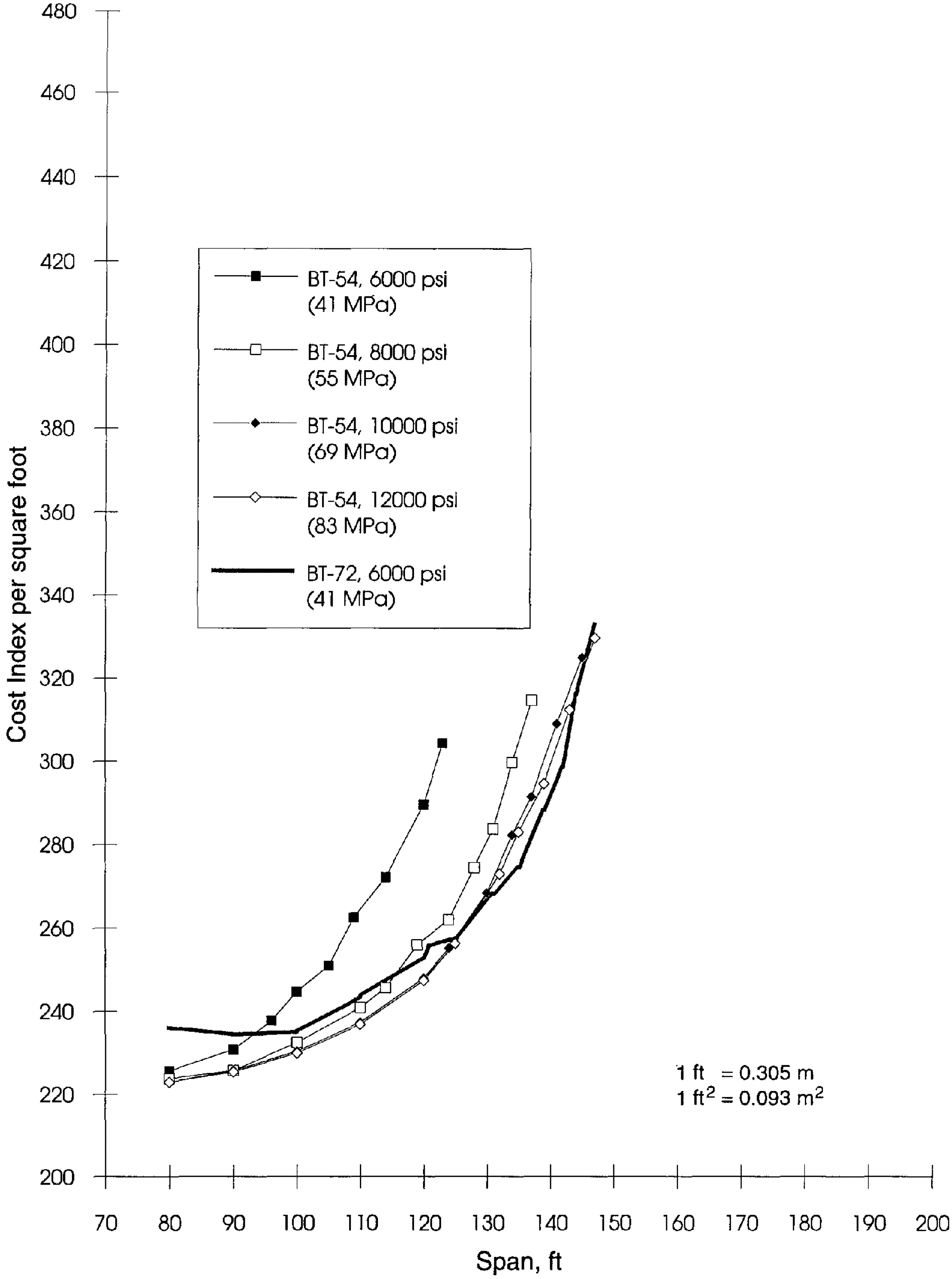


Figure 11. Comparison of a BT-54 of Varying Concrete Strength With a BT-72.

115 ft (35.0 m), a BT-54 with 8,000-psi (55-MPa) concrete is a cost-effective replacement over a BT-72 with 6,000-psi (41-MPa) concrete. Furthermore, a BT-54 with 10,000-psi (69-MPa) concrete is a cost-effective replacement up to a span length of approximately 130 ft (39.6 m). Although the exact benefits vary from section to section, the above conclusion is consistent for all of the cross sections analyzed. It should be noted that the cost index includes only superstructure costs. It does not consider reduced costs for lower approach embankments with shallower sections or the non-quantifiable aesthetic benefits of the shallower section.

Comparison No. 2 - CTL BT-72 and BT-72

Rabbat and Russell recommended that a Bulb-Tee girder with a 6-in (152-mm) web be used as a national standard precast, prestressed concrete bridge girder in the United States for spans from 80 to 140 ft (24.4 to 42.7 m).(1) However, the PCI Committee on Concrete Bridges developed a modified Bulb-Tee section for use as a national standard. The modifications resulted in a slightly heavier section that was easier to produce and handle.

For the sake of completeness, a cost comparison was performed between the originally proposed national standard of Rabbat and Russell (CTL Bulb-Tee) and that which resulted from the PCI Committee (PCI Bulb-Tee). The comparison, shown in figure 12, was performed for a 72-in (1830-mm) deep section. The optimum cost curves are very similar, with the CTL BT-72 exhibiting a slightly more cost-effective response at span lengths less than approximately 135 ft (41.1 m). However, this cost advantage is only 3.5 percent. The BT-72, on the other hand, exhibits a slightly greater maximum span capacity by virtue of its slightly larger bottom flange. However, this span length advantage is only 3.5 percent. Based on the above comparison and since the PCI Bulb-Tee is the accepted national standard, it will be used as the basis for the comparisons in this report.

Comparison No. 3 - Group 1 - BT-72, WA 14/6, and CO G68/6

The 72-in (1830-mm) PCI Bulb-Tee, the Washington Series 14 modified with a 6-in (152-mm) web, and the Colorado Series G68 modified with a 6-in (152-mm) web possess very similar cross sections. All three sections have a 6-in (152-mm) web thickness and an almost identical bottom flange that can accommodate 30 to 39 strands. Although the BT-72 and WA 14/6 share a very similar top flange, the CO G68/6 has a much narrower top flange in comparison. This narrower top flange for the CO G68/6 is the only appreciable difference between the three sections.

Optimum cost curves for the three sections are shown in figures 13 through 16 at concrete strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa), respectively. Important observations from these figures consist of the following:

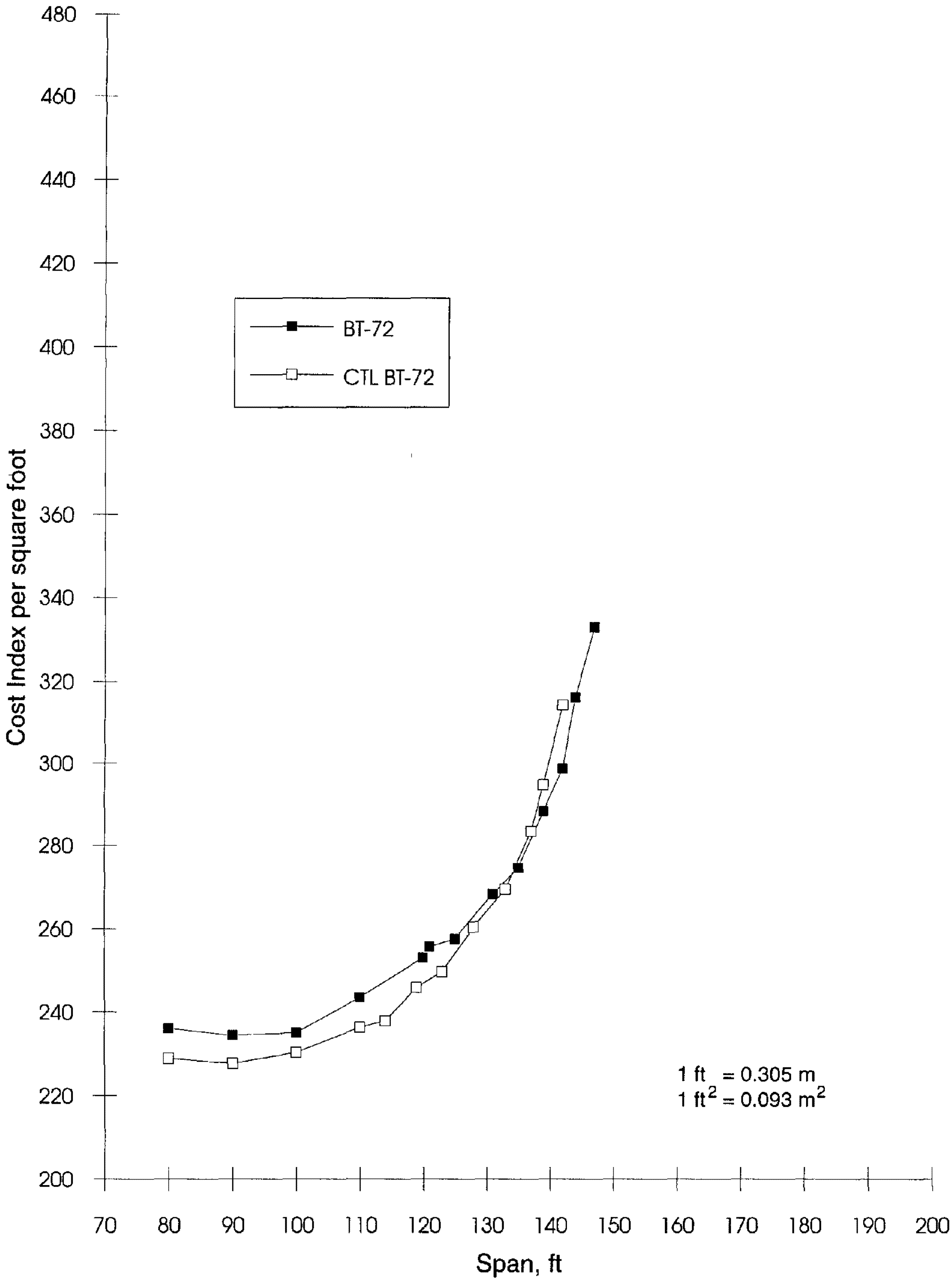


Figure 12. Comparison of a BT-72 with a CTL BT-72 at 6,000 psi (41 MPa).

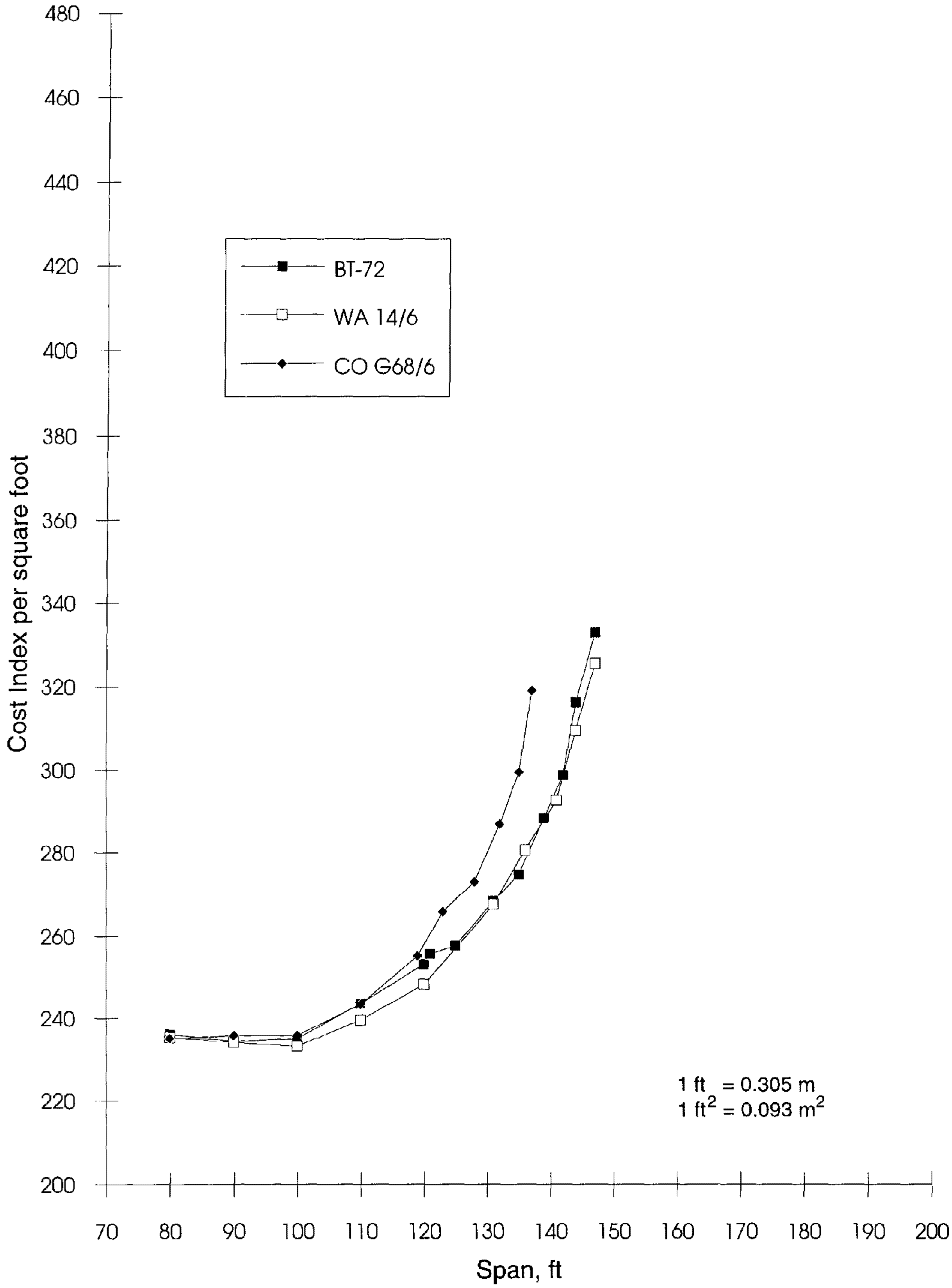


Figure 13. Comparison of BT-72, Washington 14/6, and Colorado G68/6 at 6,000 psi (41 MPa).

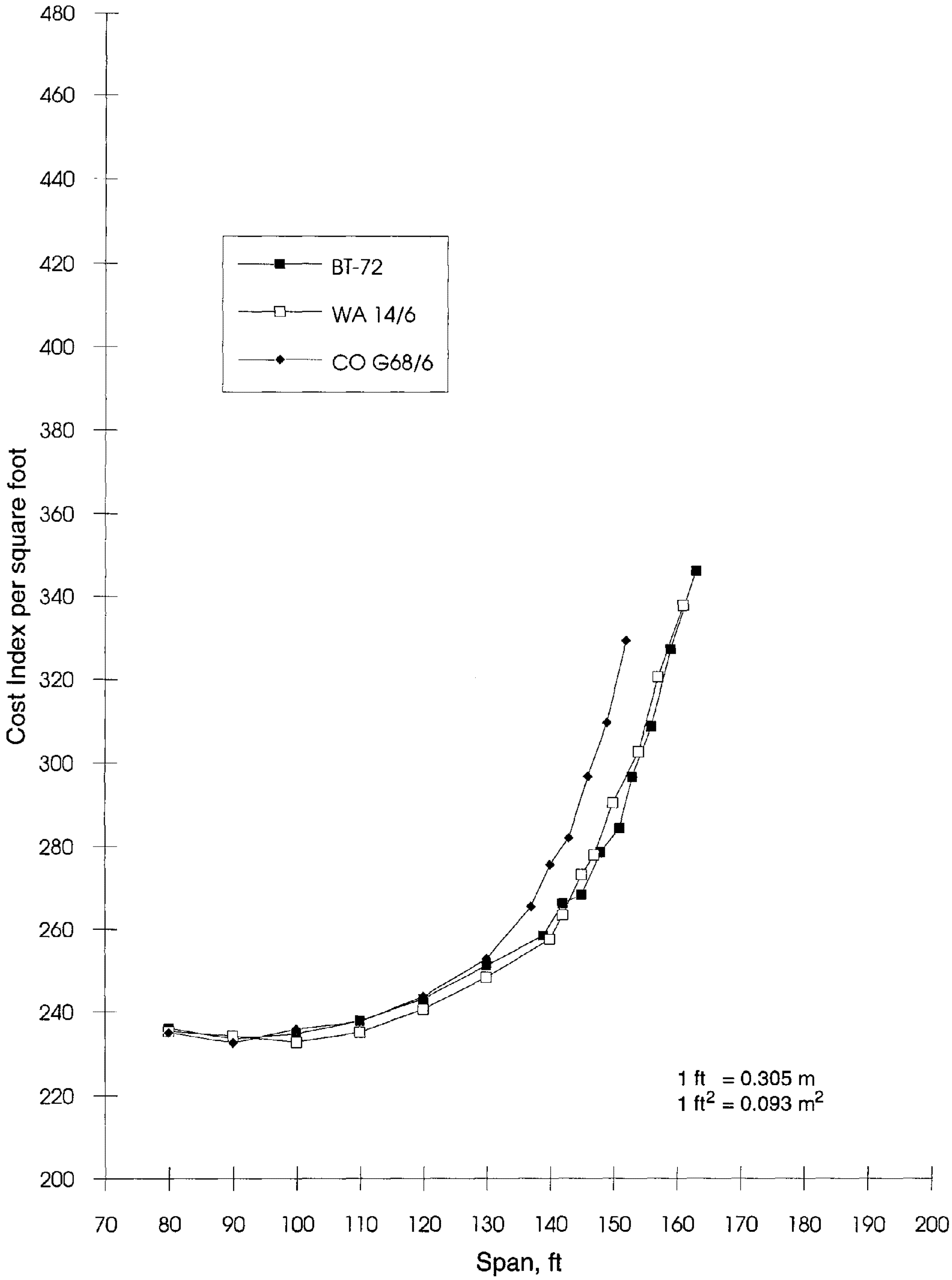


Figure 14. Comparison of BT-72, Washington 14/6, and Colorado G68/6 at 8,000 psi (55 MPa).

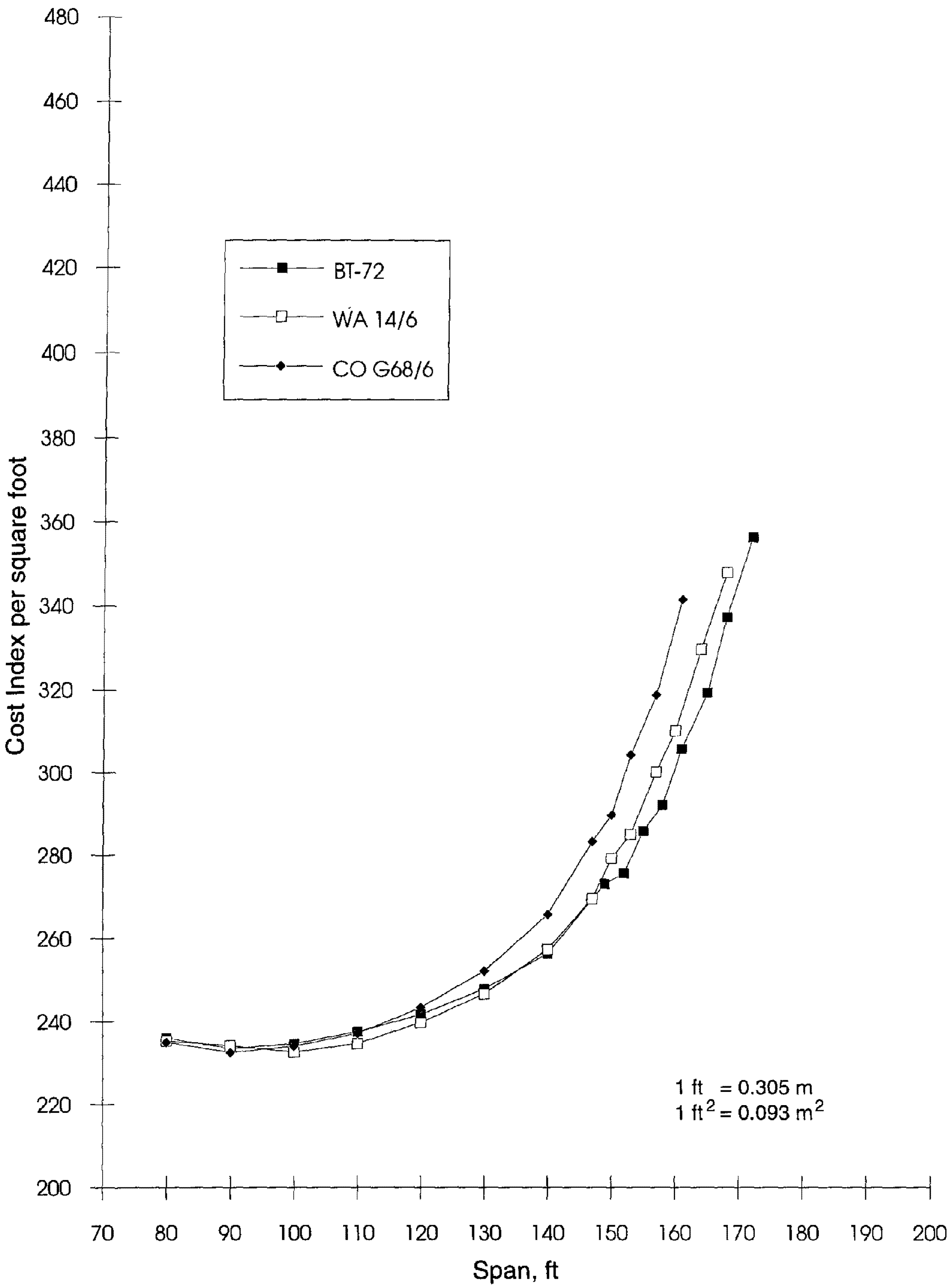


Figure 15. Comparison of BT-72, Washington 14/6, and Colorado G68/6 at 10,000 psi (69 MPa).

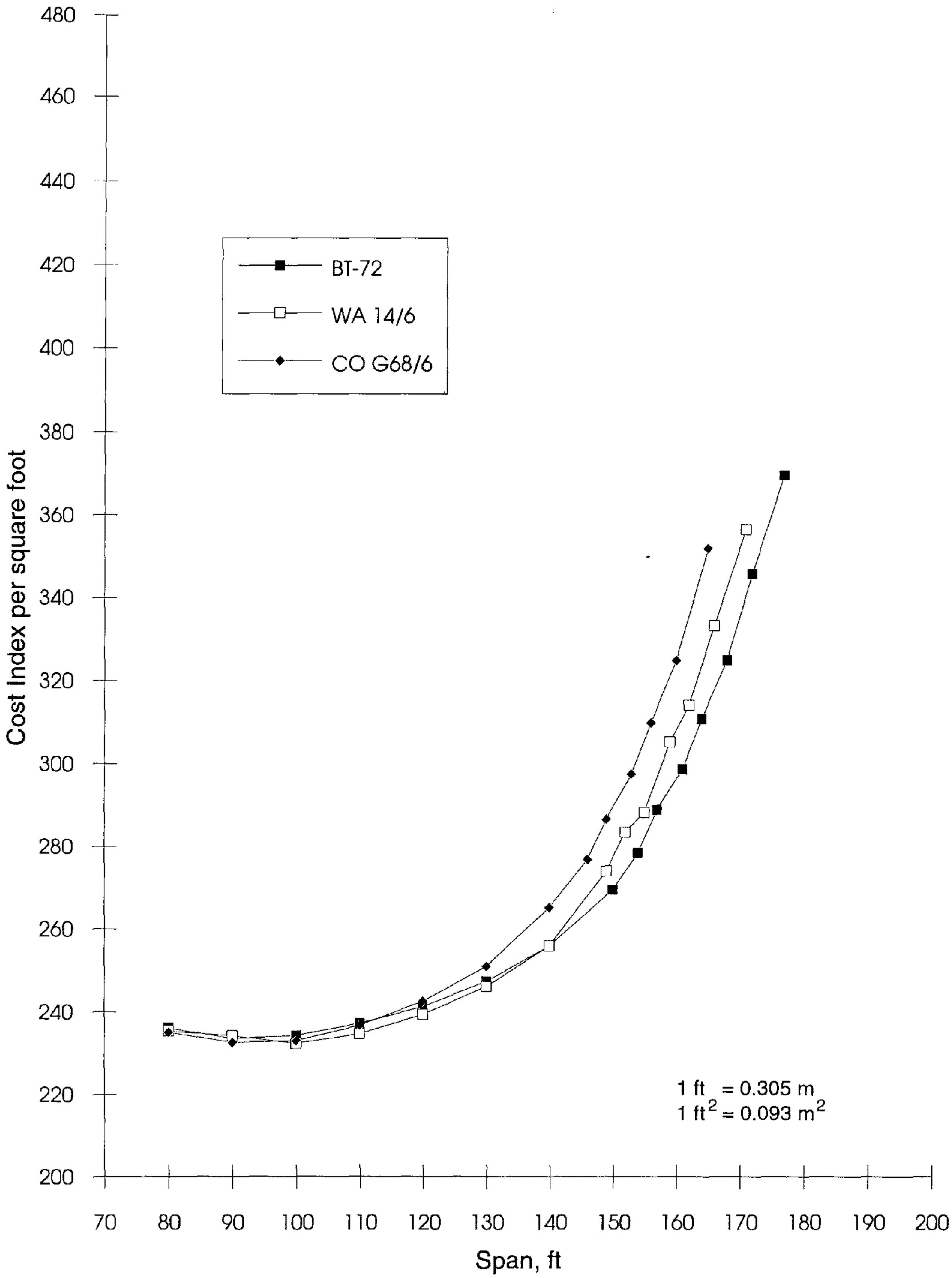


Figure 16. Comparison of BT-72, Washington 14/6, and Colorado G68/6 at 12,000 psi (83 MPa).

1. The curves are essentially identical along the horizontal portions.

2. The curves vary along the vertical portions. However, this variation is greatest for the CO G68/6 and is attributable to the shallower depth and thinner top flange. The BT-72 and WA 14/6 are very similar within the vertical portions.

3. Incremental shifts in the optimum cost curve for increasing girder strength vary for each section. The BT-72 undergoes the largest shifts, while the WA 14/6 undergoes the smallest. This is due primarily to the number of prestressing strands that can be placed within the flange. The BT-72, WA 14/6, and CO G68/6 can accommodate 39, 30, and 35 strands, respectively, within the bottom flange. Strand placement within the flange is more efficient than in the web and allows the CO G68/6 to gradually gain on the WA 14/6 as girder strength is incrementally increased.

Comparison No. 4 - Group 2 - NU 1800 and FL BT-72

The 72-in (1830-mm) Florida Bulb-Tee was compared to an equivalent depth Nebraska University metric section. Physical dimensions of the two sections are comparable; both have a 48-in (1220-mm) wide top flange and a larger bottom flange than the BT-72. Although the bottom flange for the FL BT-72 is narrower than the NU 1800's, it is deeper and can actually accommodate a larger number of prestressing strands, 59 versus 53.

Optimum cost curves for the two sections are shown in figures 17 through 20 at concrete strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa), respectively. As shown in these figures, the two sections have almost identical optimum cost curves at all four concrete strengths. No cost advantage exists for one section over the other.

Comparison No. 5 - Groupings

The next comparison deals with selected girders from the above groups with the addition of a modified AASHTO Type VI girder. Comparisons are based on the following cross section combinations:

1. Group 1: BT-72, WA 14/6, and CO G68/6.

2. Group 2: NU 1800 and FL BT-72.

3. Group 3: Type VI.

These sections have been grouped together on the basis of similarities in their cross section and, as a result, similarities in their optimum cost curves. It is interesting to note the maximum number of prestressing strands that can be placed within the bottom flange of each group. Group 1, Group 2, and Group 3 can accommodate 30 to 39, 53 to 59, and 81 strands,

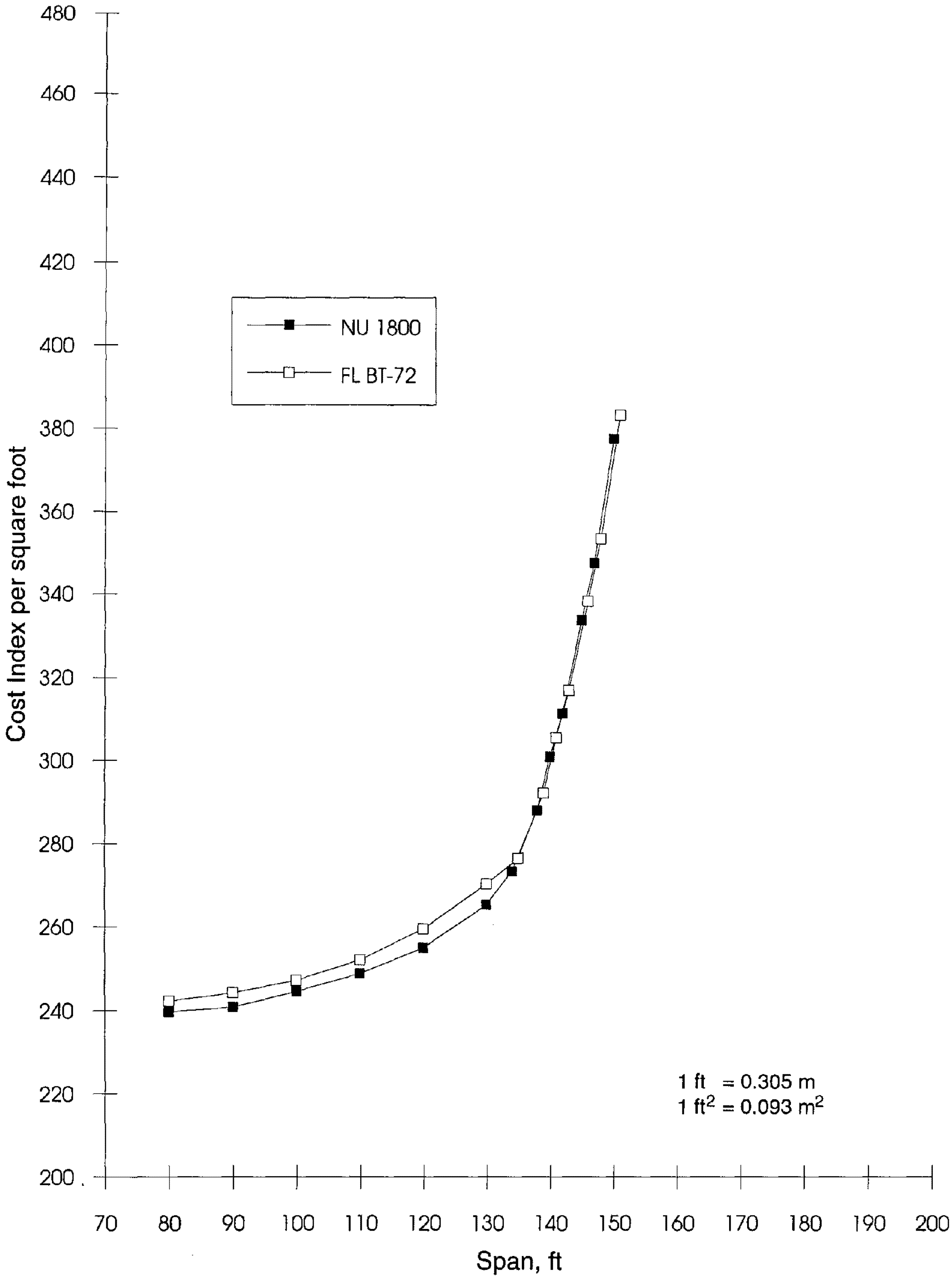


Figure 17. Comparison of NU 1800 and FL BT-72 at 6,000 psi (41 MPa).

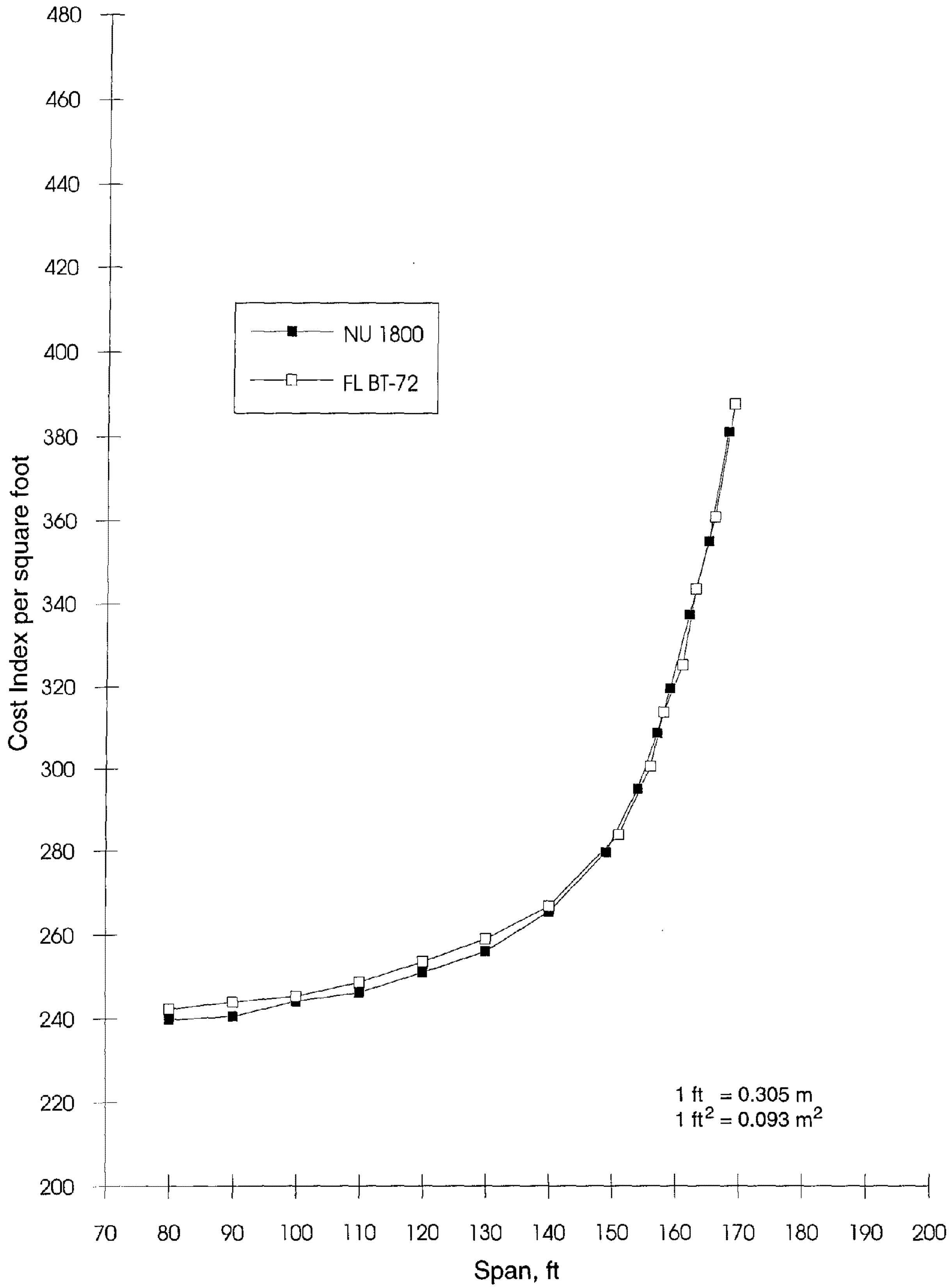


Figure 18. Comparison of NU 1800 and FL BT-72 at 8,000 psi (55 MPa).

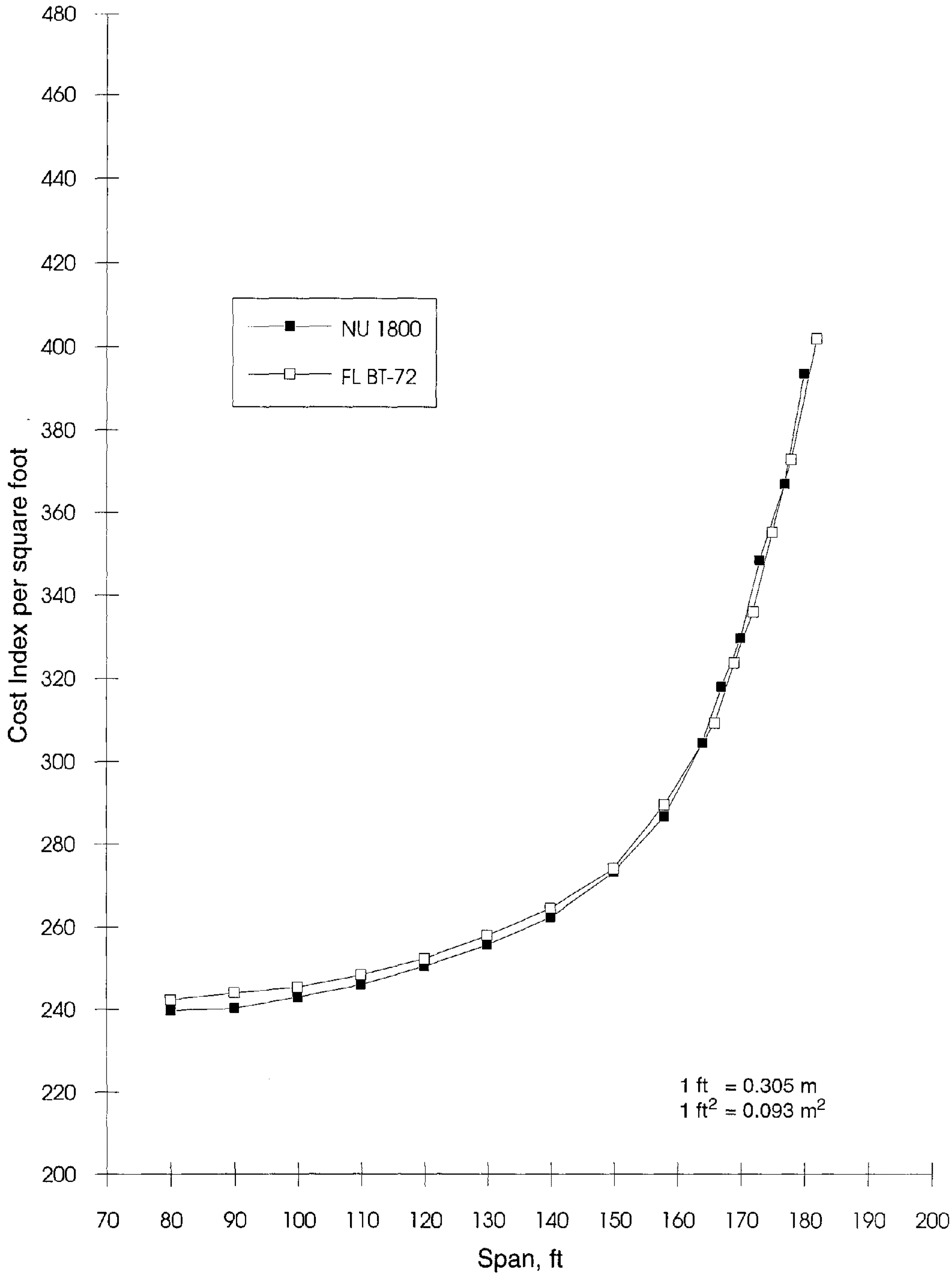


Figure 19. Comparison of NU 1800 and FL BT-72 at 10,000 psi (69 MPa).

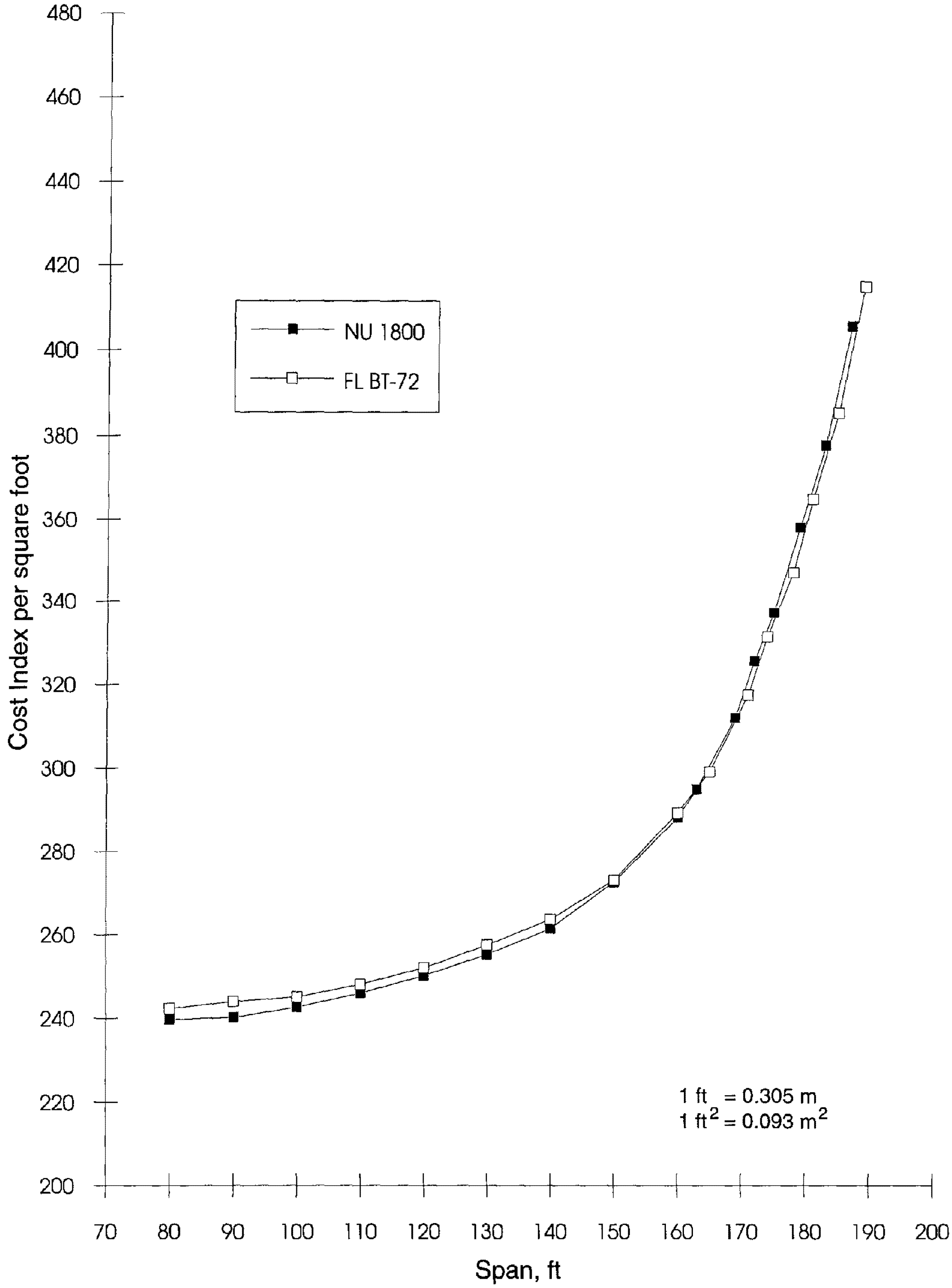


Figure 20. Comparison of NU 1800 and FL BT-72 at 12,000 psi (83 MPa).

respectively, within the bottom flange. For the comparison, each group is represented by the optimum cost curves of a single cross section as follows:

1. Group 1: BT-72.
2. Group 2: FL BT-72.
3. Group 3: Type VI.

Figures 21 through 24 are plots of the optimum cost curves for the BT-72, FL BT-72, and Type VI at concrete strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa), respectively. Important observations from these figures consist of the following:

1. At a girder strength of 6,000 psi (41 MPa), the BT-72 is the most cost-effective cross section with a savings of 1 to 6 percent over the FL BT-72 and 5 to 13 percent over the Type VI.

2. At all girder strengths, the BT-72 is the most cost-effective cross section for span lengths up to about 150 ft (45.7 m), with a savings of 3 to 6 percent over the FL BT-72 and 8 to 13 percent over the Type VI.

3. Incremental shifts in the optimum cost curve for increasing girder strength vary for each section. The FL BT-72 undergoes the largest shift, while the BT-72 undergoes the smallest. This is due primarily to the number of prestressing strands that can be placed within the bottom flange. However, it is also a function of the efficiency in which the strands are placed in the bottom flange. For instance, the FL BT-72 and BT-72 have wide rectangular bottom flanges, while the Type VI has a bottom flange that is more square. Although the Type VI can accommodate significantly more strands than the FL BT-72 (81 versus 59), their placement is not as efficient (less eccentricity) and, as a result, shifts in the FL BT-72 curve are greater than those for the Type VI.

4. As a result of the incremental shifts in the optimum cost curve as discussed in Item 3, the FL BT-72 becomes the most cost-effective cross section for span lengths exceeding about 150 ft (45.7 m) and girder strengths of 8,000 psi (55 MPa) and greater.

5. At a girder strength of 10,000 psi (69 MPa), the large capacity of the Type VI bottom flange allows it to become more cost-effective than the BT-72 for span lengths in excess of 165 ft (50.3 m).

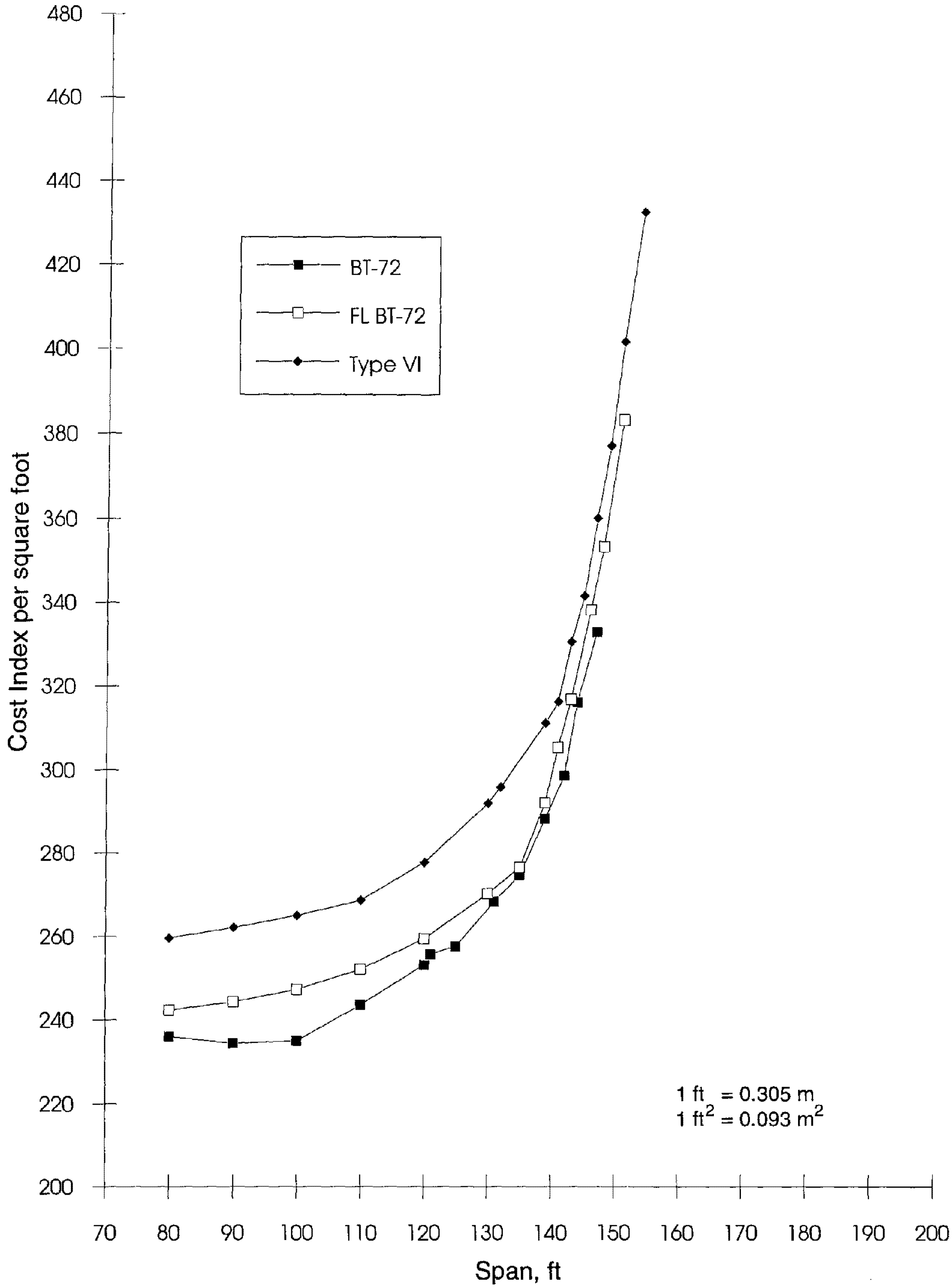


Figure 21. Comparison of BT-72, Florida BT-72, and AASHTO Type VI at 6,000 psi (41 MPa).

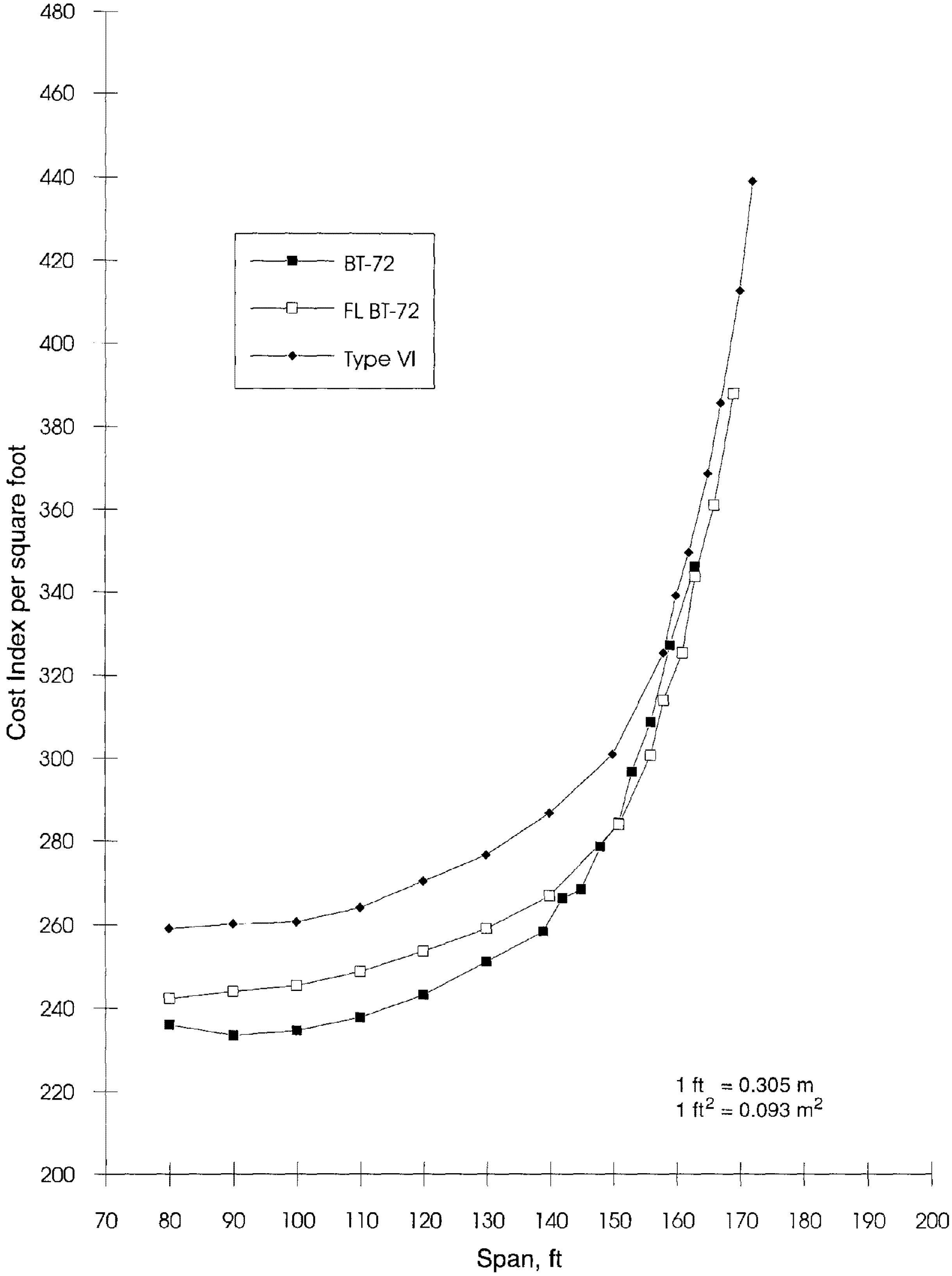


Figure 22. Comparison of BT-72, Florida BT-72, and AASHTO Type VI at 8,000 psi (55 MPa).

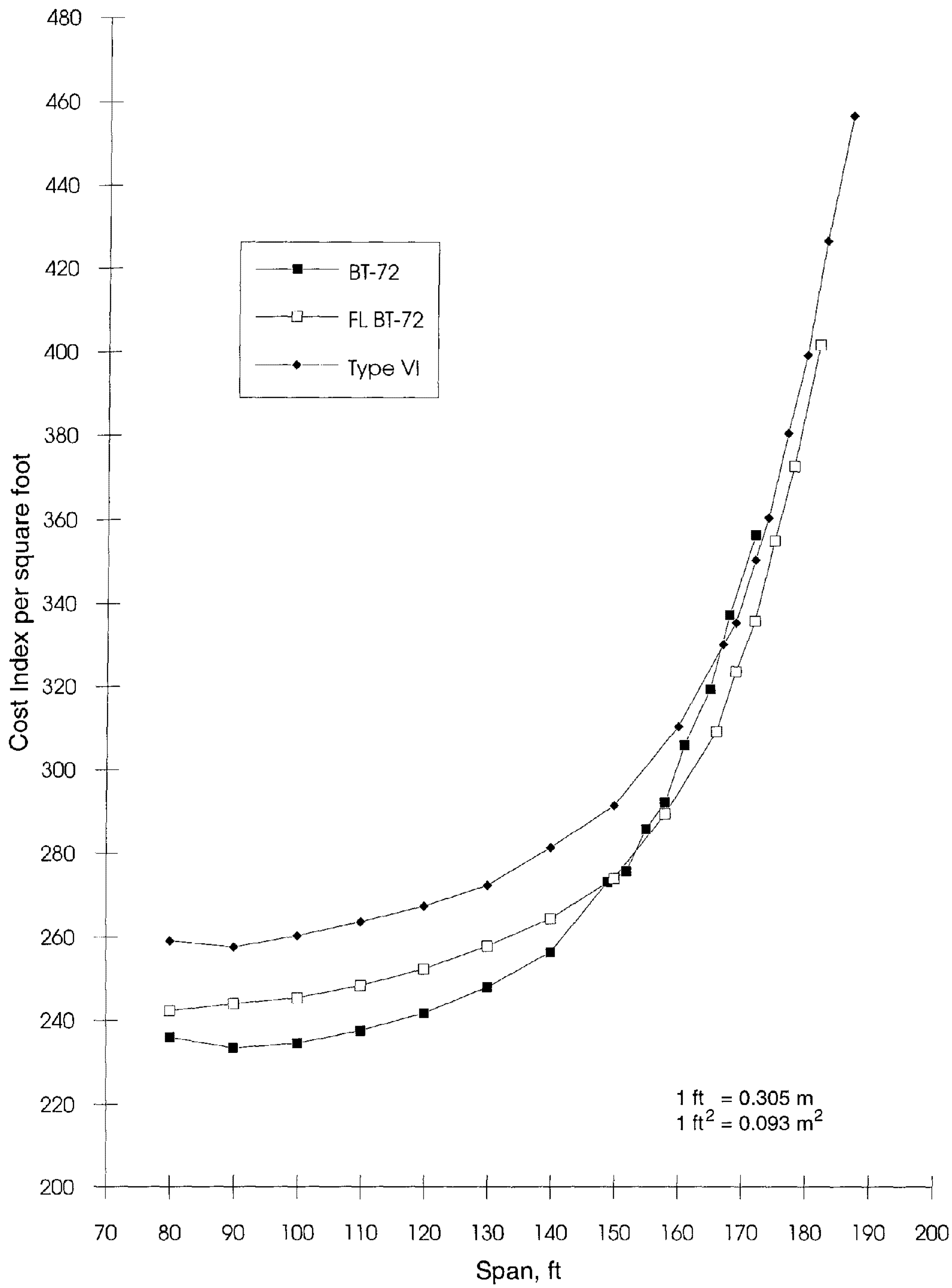


Figure 23. Comparison of BT-72, Florida BT-72, and AASHTO Type VI at 10,000 psi (69 MPa).

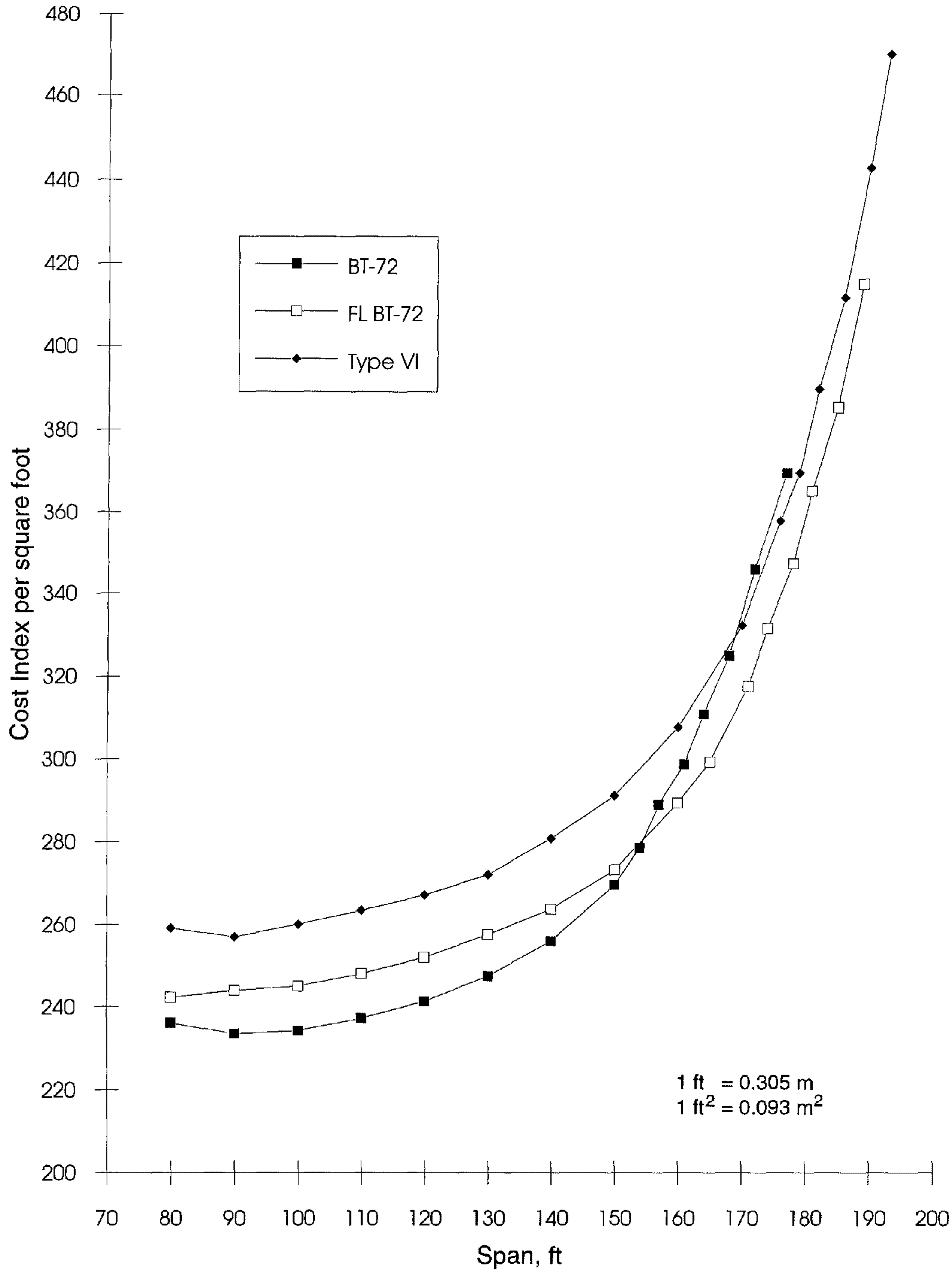


Figure 24. Comparison of BT-72, Florida BT-72, and AASHTO Type VI at 12,000 psi (83 MPa).

6. Although the FL BT-72 and Type VI enjoy greater horizontal shifts in their optimum cost curve than the BT-72, as a result of larger bottom flanges they pay a price at smaller span lengths. For these spans, the BT-72 is the more cost-effective cross section at all girder concrete strengths.

Comparison No. 6 - U54B and BT-54

The Texas U-beam was compared to an equivalent depth PCI Bulb-Tee. The U54B section was chosen over the U54A because of its thicker bottom flange — 8 in versus 6 in (203 mm versus 152 mm), which can accommodate a larger number of prestressing strands. The computer program BRIDGE was modified to allow for the analysis and design of the U54B cross section. For comparison purposes, the analysis was based on a complete cast-in-place deck, rather than on the composite precast/cast-in-place deck that is being used in Texas.

Figure 25 contains a set of optimum cost curves for both the U54B and BT-54. Each set of curves corresponds to girder concrete strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa), respectively. The upper set of curves correspond to the BT-54, and the lower set of curves correspond to the U54B. Figure 25 reveals the following:

1. For equal girder strengths below 10,000 psi (69 MPa), the U54B is a more cost-effective section for the shorter span lengths, while the BT-54 is more cost-effective for the longer span lengths.

2. At girder strengths exceeding 10,000 psi (69 MPa), the U54B is more cost-effective than the BT-54 at all span lengths.

3. Incremental shifts in the optimum cost curve for increasing girder strength are greater for the U54B.

The reason for the U54B cost-efficiency, both in terms of normal concrete strengths and high concrete strengths, can be determined by examining one-half of the section and comparing it to the BT-54. The U54B has a 5-in (127-mm) thick web that was shown by Rabbat and Russell to result in a significant savings over an equivalent section with a 6-in (152-mm) thick web.(1) However, the 6-in (152-mm) web was recommended as a national standard in order to facilitate concrete consolidation in all regions of the United States. An advantage of the U54B section is the sloping webs, which may reduce the problem of consolidation within the bottom flange and allow for the benefits of a thinner web to be realized.

In contrast, large incremental shifts in the U54B optimum cost curve are attributable to the relatively large bottom flange, which does not waste concrete with significant tapering, and the relatively narrow top flange. The large bottom flange allows efficient strand placement as the

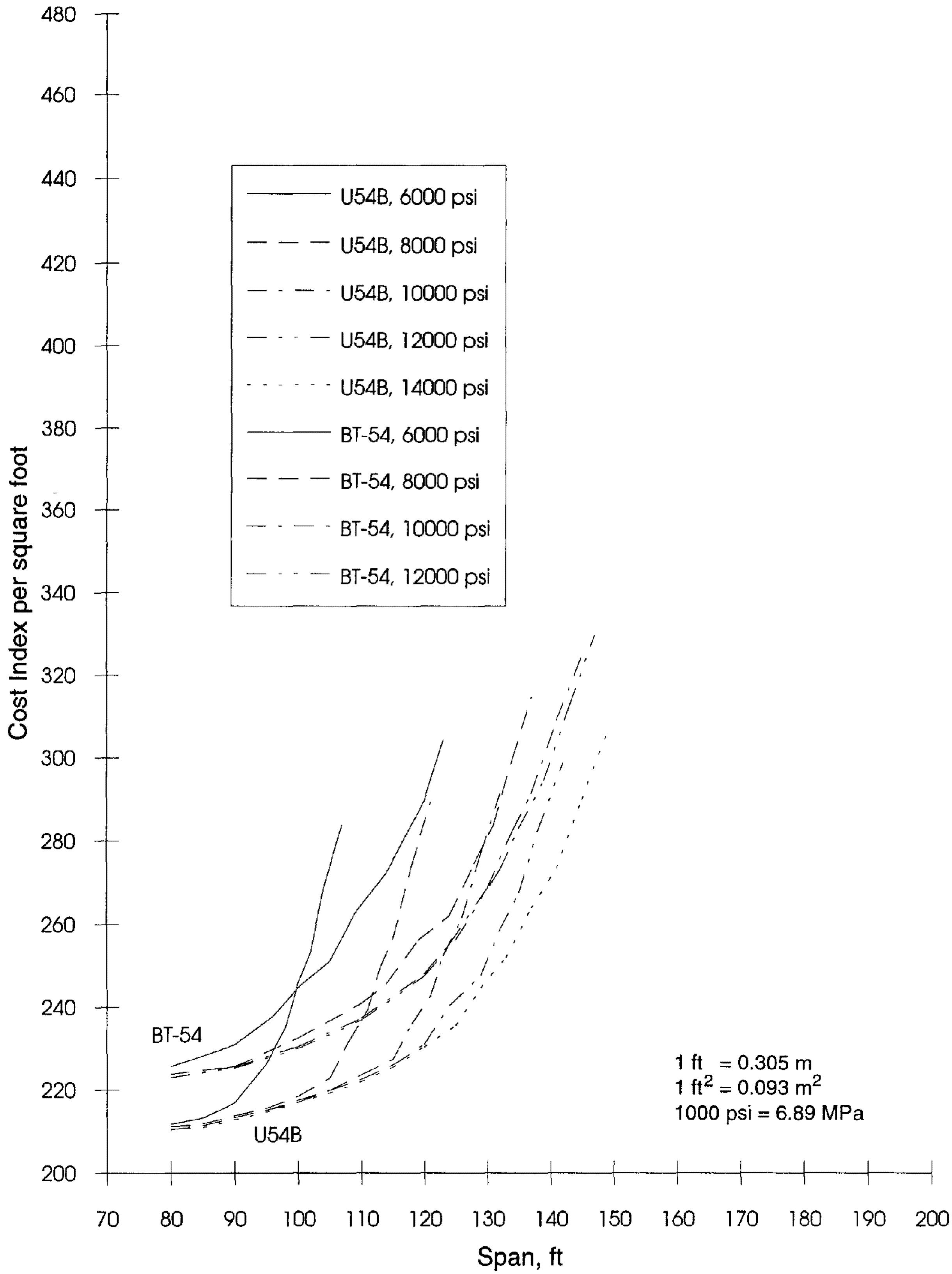


Figure 25. Comparison of a U54B and a BT-54.

girder strength is increased. For example, at a girder spacing of 9 ft (2.7 m), strand placement is not required within the web until the girder strength exceeds 10,000 psi (69 MPa). The narrow top flange, on the other hand, results in compression at midspan controlling the design up to a girder strength of 14,000 psi (97 MPa). At 14,000 psi (97 MPa), both tension and compression control the design — an efficient use of the material strengths.

The narrow top flange does result in increased deck costs, due to larger deck span lengths in the transverse direction. However, this increase is not significant since existing T-shaped sections do not fulfill the AASHTO requirement for top flange width/thickness ratios below a value of 4.0, whereas the U54B section does fulfill this requirement. As a result, the effective deck spans, and ultimately deck costs, do not vary significantly between a U54B and a BT-54 with the same girder spacing. The effective deck spans as defined by AASHTO are illustrated in figure 61 of Appendix C.

RELATIVE UNIT COSTS

The previous investigation by Rabbat and Russell identified that costs of materials and labor varied from region to region within the United States and that they also varied between States within a given region, between districts of a State, and within a district according to bridge location.(1) Consequently, cost analyses were performed by comparing the costs of the recommended sections based on a common ground. From survey data, average costs for girder concrete, deck concrete, reinforcing steel and prestressing strands were determined. Average costs included costs of material and labor. For girder concrete, the cost also included transportation and erection. These average costs were then reduced to relative costs per pound of each respective in-place material and were used in the analysis.

In the present investigation, the relative unit costs for in-place materials were assumed to be the same as used in the previous report.(1) However, it was recognized that the relative material costs may have changed since the previous investigation. Consequently, limited analyses were made to investigate the sensitivity of the cost index per square foot to the assumed relative unit costs. Based on the data described in Appendix B, the following relative unit costs of each respective material were used for the analyses:

Material	Measure	Minimum Ratio	Intermediate Ratio	Maximum Ratio
Concrete (girder)	unit/unit weight of girder concrete	1	1	1
Concrete (deck)	unit/unit weight of deck concrete	1	1	1
Strands	unit/unit weight of strand	8	15	30
Uncoated Reinforcement	unit/unit weight of reinforcement	9	25	25
Epoxy-Coated Reinforcement	unit/unit weight of epoxy-coated reinforcement	12	30	50

Comparisons were made based on the BT-72 with 6,000-psi (41-MPa) compressive strength concrete. A comparison of the optimum cost curves for the three assumed ratios is shown in figure 26. The optimum cost index curves were displaced upward, with the assumption of higher costs for the prestressing steel and reinforcement compared to the cost of the concrete. The largest increase in cost indexes occurred as the result of deck design changes, particularly when the cost of the deck reinforcement was significantly higher. However, since all designs are being compared on a relative basis, the same information shown in figure 26 is reproduced in figure 27 normalized relative to 100 percent for an 80-ft (24.4-m) span girder. These data indicate that for the maximum range of relative unit costs assumed, the relative cost index only increased by 15 percent compared to the minimum relative unit cost curve. Based on this analysis, it was concluded that the cost index per square foot on a comparative basis is relatively insensitive to the assumed relative costs.

Comparisons were also made to determine the effect of the premium cost for higher strength concretes on the cost index per square foot. Based on the information described in Appendix B, the following ratios were assumed for the premium costs of higher strength concrete:

Strength	Minimum Ratio	Intermediate Ratio	Maximum Ratio
6,000 psi (41 MPa)	1.00	1.00	1.00
8,000 psi (55 MPa)	1.00	1.05	1.10
10,000 psi (69 MPa)	1.00	1.13	1.25
12,000 psi (83 MPa)	1.00	1.25	1.50

The comparisons of optimum cost curves were made for the BT-72 for compressive strengths from 6,000 to 12,000 psi (41 to 83 MPa). Data from the three sets of cost index curves are shown in figure 28. The effect of the premium costs is to displace the relative positions of the

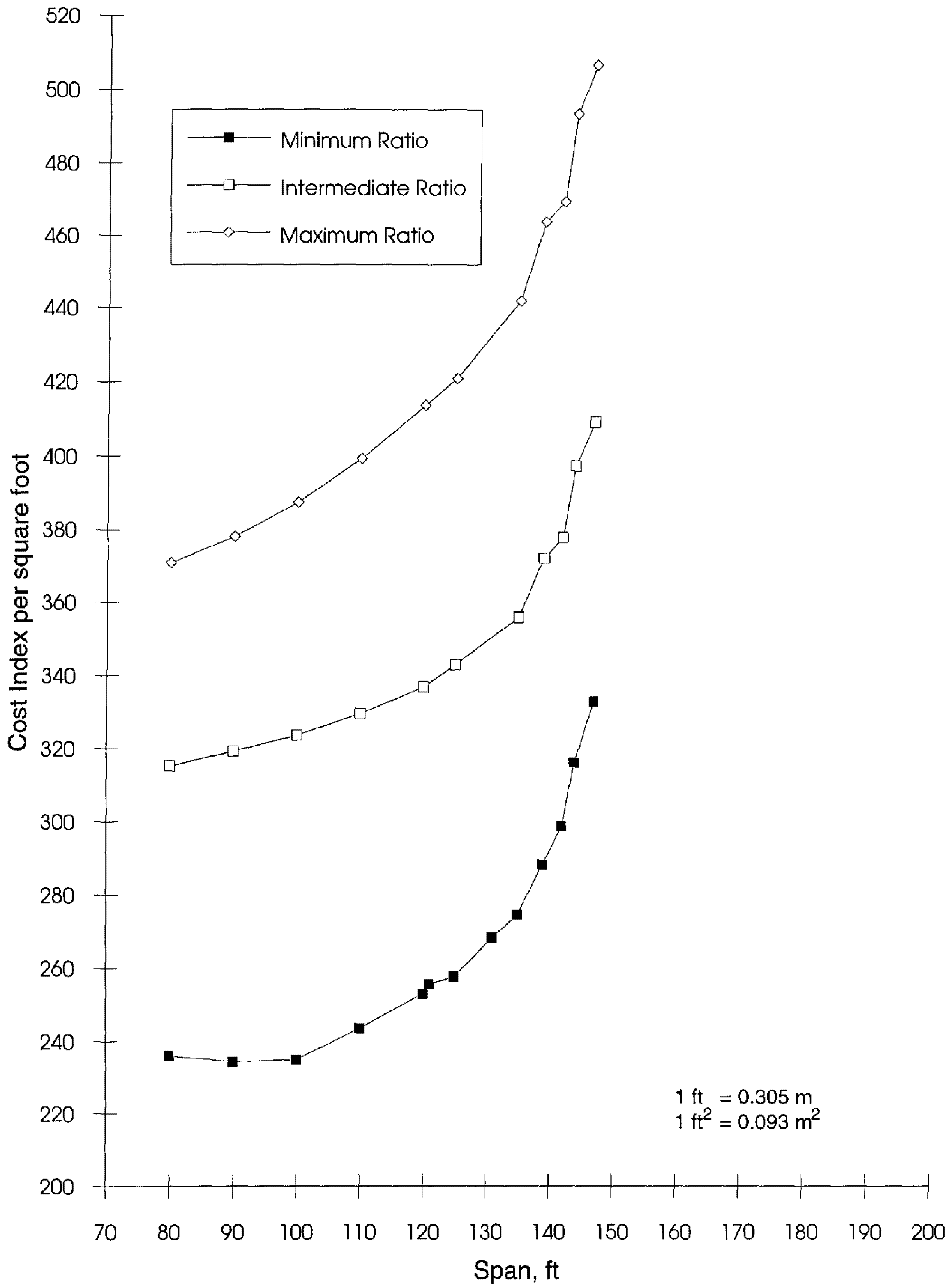


Figure 26. Effect of Relative Unit Costs on Cost Index.

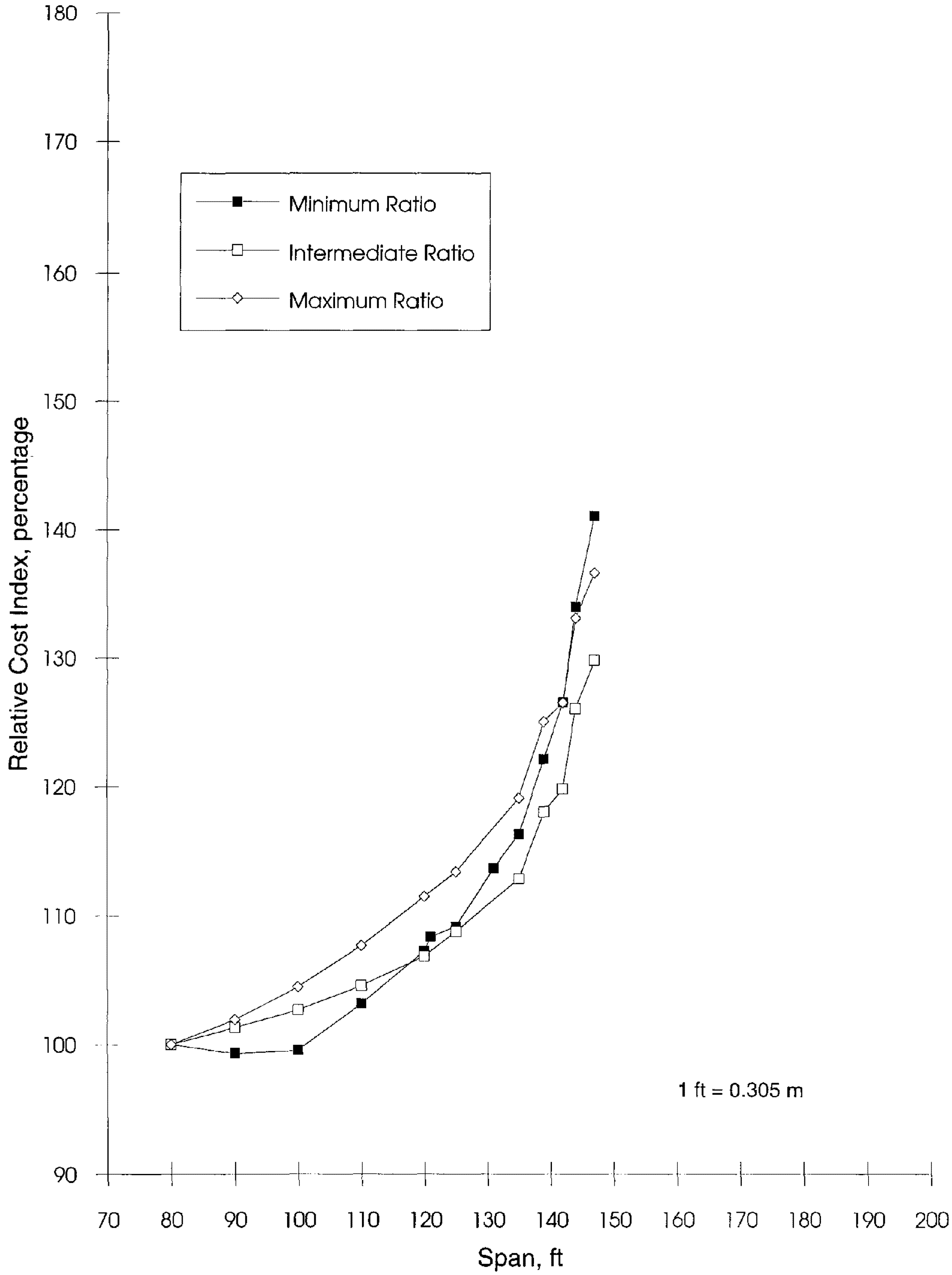


Figure 27. Effect of Relative Unit Costs on Relative Cost Index.

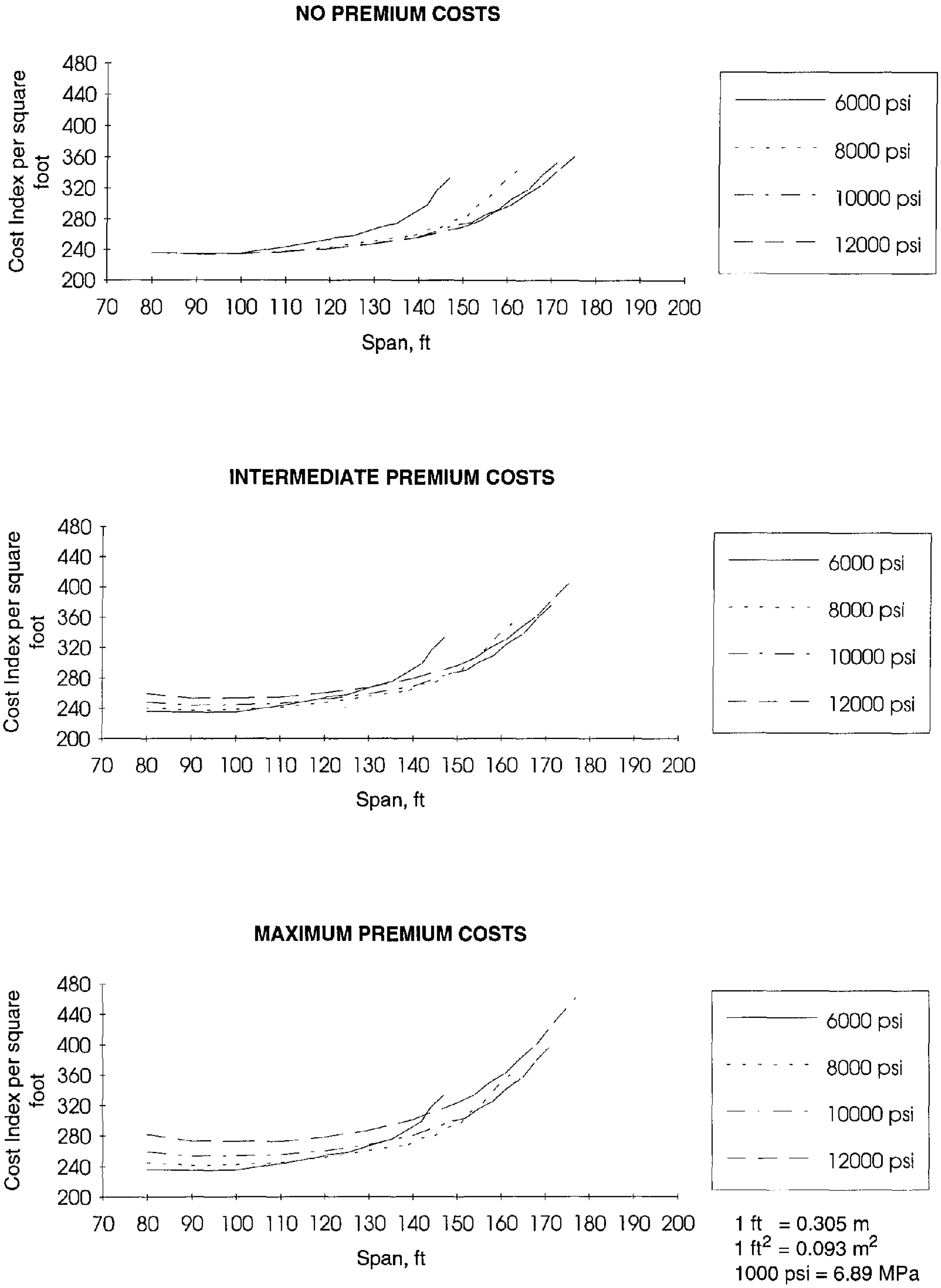


Figure 28. Effect of Premium Concrete Costs on Cost Index.

curves for the different concrete strengths. These data indicate that as the premium for the higher strength concretes increases, it becomes more economical to utilize a lower strength concrete for longer span lengths. For example, with no premium concrete costs, the 8,000-psi (55-MPa) compressive strength concrete is the most economical up to a span length of approximately 120 ft (36 m). However, with the maximum premium costs, it is more economical to use 6,000 psi (41 MPa) up to a span length of approximately 120 ft (36 m) and then to utilize 8,000 psi (55 MPa) up to a span length of 150 ft (46 m). However, on a relative basis when comparing different cross sections, the effect of the premium concrete costs is to displace cost index curves by a similar amount. Consequently, although the premium costs are important when comparing concrete strengths for the same girder cross section, they are less significant when comparing different cross sections with the same concrete strength.

3. ANALYSES OF MODIFIED CROSS SECTIONS

Cost-efficiency analyses discussed in Chapter 2 identified the most cost-effective existing cross sections and the strength levels beyond which high-strength concrete cannot be effectively utilized with existing girders. The PCI Bulb-Tee still maintained its cost advantage over the majority of span lengths and girder strengths studied; however, the Florida Bulb-Tee and the equivalent Nebraska metric section revealed cost-efficiencies surpassing the PCI Bulb-Tee for span lengths exceeding 150 ft (45.7 m) at girder strengths of 8,000 psi (55 MPa) and greater. Furthermore, the 54-in (1370-mm) deep Texas U-beam indicated cost-efficiencies over an equivalent depth PCI Bulb-Tee at several span lengths and girder strengths. In general, for all sections analyzed, high-strength concrete offered cost-effective advantages.

However, these advantages are limited by factors that exist in current precast, prestressed bridge girder construction. These factors consist of the following:

1. Physical limitations of the section's bottom flange.
2. Cost-effectiveness of conventionally reinforced decks with large girder spacings.

Physical limitations of the section's bottom flange relate directly to the amount and efficient placement of prestressing strand within the girder. The chief structural benefit of higher strength concrete is the greater amount of prestressing force that can be imposed on the section. Physical limitations on strand placement not withstanding, if the concrete strength doubles, the maximum amount of prestressing force doubles and, therefore, the section's allowable service load moment doubles. However, physical dimensions of the section's bottom flange limit the amount and location of prestressing strands. Once the bottom flange reaches its capacity for total number of strands, the section's efficiency falls off dramatically. Furthermore, the shape of the bottom flange directly influences the efficiency of strand placement. For the same amount of concrete, wider and thinner flanges are more efficient since they maximize strand eccentricity.

The second factor that limits the application of high-strength concrete is the cost-effectiveness of conventional concrete decks with large girder spacings. As indicated by previous research and verified by this investigation, increased girder strengths allow increased girder spacings for the same span length and girder cross section. These increased girder spacings for a given span will normally result in lower total unit costs per square foot of bridge deck. However, based on the cost-efficiency analyses performed in this study, there is a point at which the increase in unit deck costs alone begin to outweigh the savings in unit girder costs associated with larger girder spacings. For instance, a 10,000-psi (69-MPa) BT-72 can be spaced at 16 ft (4.9 m) on center for a 100-ft (30.5-m) span. However, a lower unit cost structure results at a girder spacing of only 10.5 ft (3.2 m). For long span decks, a reinforced concrete slab is not cost-effective and limits the potential benefits of higher strength concrete in the girder.

The limitation of bottom flange size is addressed in the following section of this report, which investigates modifications to existing cross sections that may overcome this limitation. These modifications include strand size, strand spacing, strand strength, and section geometry. Increased efficiency for longer span decks is beyond the scope of this report.

STRAND SPACING AND SIZE

One alternative to the bottom flange physical limitation on strand placement is to alter existing strand sizes and/or spacing. This allows a greater prestressing force within the bottom flange. For this parametric investigation, a 72-in (1830-mm) deep PCI Bulb-Tee served as the standardized cross section. Furthermore, the baseline condition used 0.5-in (12.7-mm) diameter prestressing strands spaced 2 in (51 mm) on center with a 2-in (51-mm) concrete surface to center of strand spacing. Two strands were allowed within each row of the web. Optimum cost curves for this baseline condition are given in figure 6.

Figures 29 through 32 compare optimum cost curves at concrete strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa), respectively, when the strand spacing is reduced from 2 in (51 mm) to 1.5 in (38 mm). As can be seen from these figures, the spacing reduction has the largest impact on the 12,000-psi (83-MPa) plot and the smallest impact on the 6,000-psi (41-MPa) plot, with the 8,000- and 10,000-psi (55- and 69-MPa) plots falling somewhere between the two extremes. This behavior is consistent with the previous conclusion that the bottom flange size limited the effectiveness of higher concrete strengths. With a smaller strand spacing, more prestressing force can be placed within the cross section. Span length capacities are increased by a maximum of 16 percent, but, more importantly, span lengths from 130 to 175 ft (39.6 to 53.3 m) are now more cost-effective by 3 to 30 percent.

The next modification examined the use of 0.6-in (15.2-mm) diameter prestressing strands for both a 2-in (51-mm) and 2.5-in (64-mm) strand spacing. These curves were compared to the 0.5-in (12.7-mm) diameter strand spaced at 1.5 in (38 mm) and 2 in (51 mm). The comparisons are shown in figures 33 through 36 for girder concrete strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa), respectively. As shown in these figures, a definite pairing occurs corresponding to the ratio of strand spacing to strand diameter. This result is directly attributable to the maximum area of prestressing steel that can be placed within the bottom flange. For a spacing of approximately three diameters, the 0.5-in (12.7-mm) and 0.6-in (15.2-mm) diameter strands result in a maximum prestressing steel area of 9.33 and 8.46 in^2 (6020 and 5460 mm^2), respectively, within the bottom flange; at a spacing of approximately four diameters , the areas are 5.97 and 5.43 in^2 (3850 and 3500 mm^2). As

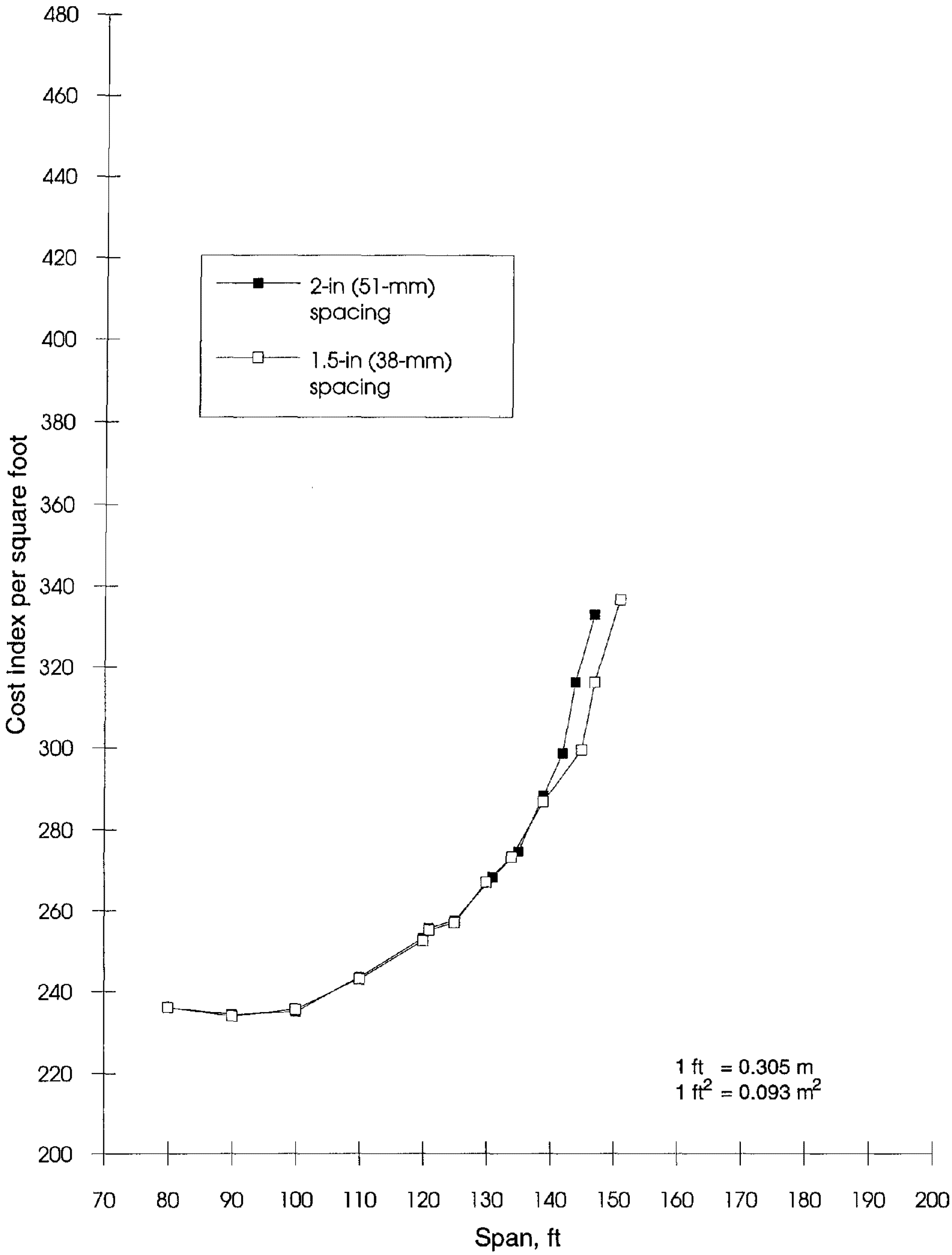


Figure 29. BT-72 with 0.5-in (12.7-mm) Diameter Strand at Various Spacings at 6,000 psi (41 MPa).

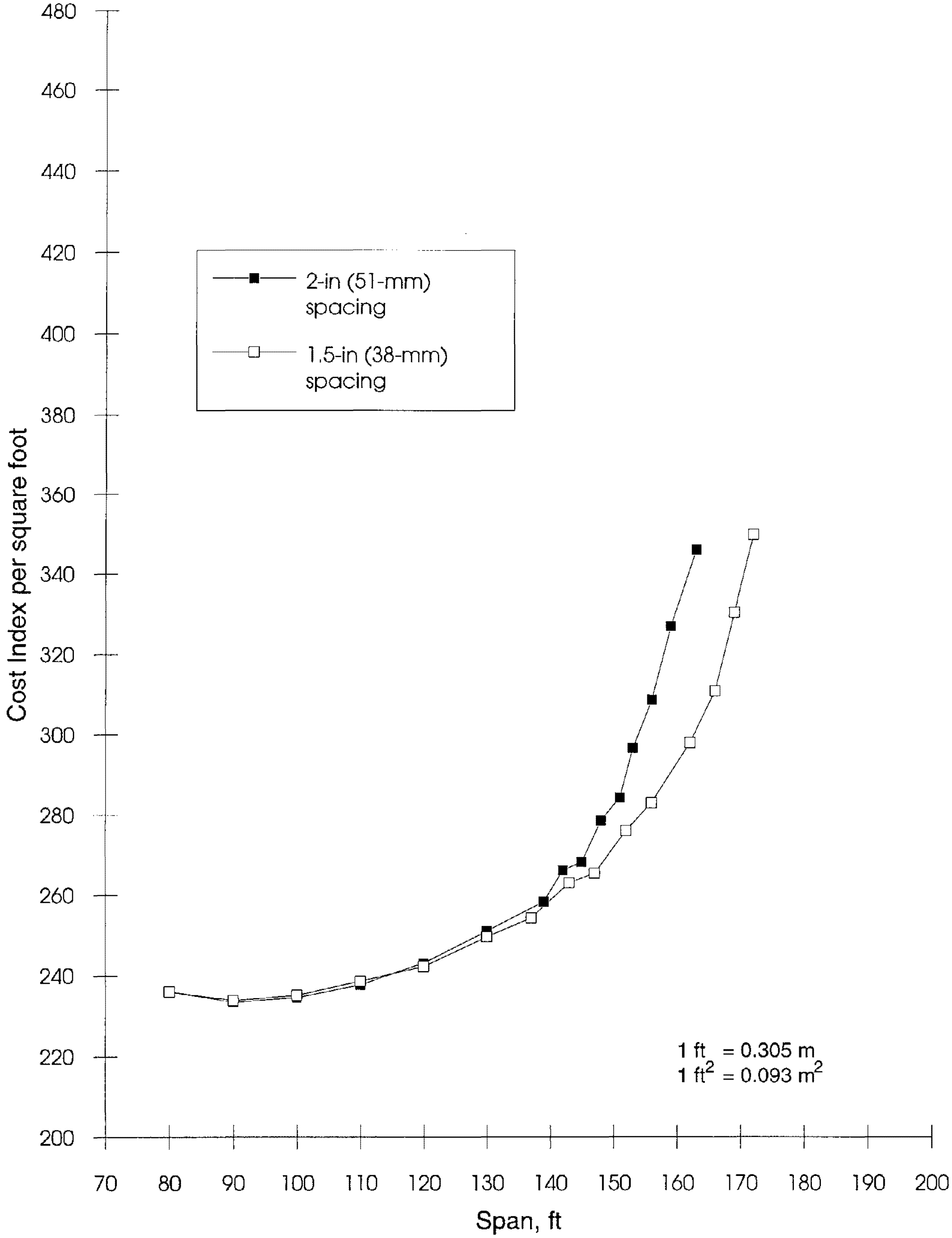


Figure 30. BT-72 with 0.5-in (12.7-mm) Diameter Strand at Various Spacings at 8,000 psi (55 MPa).

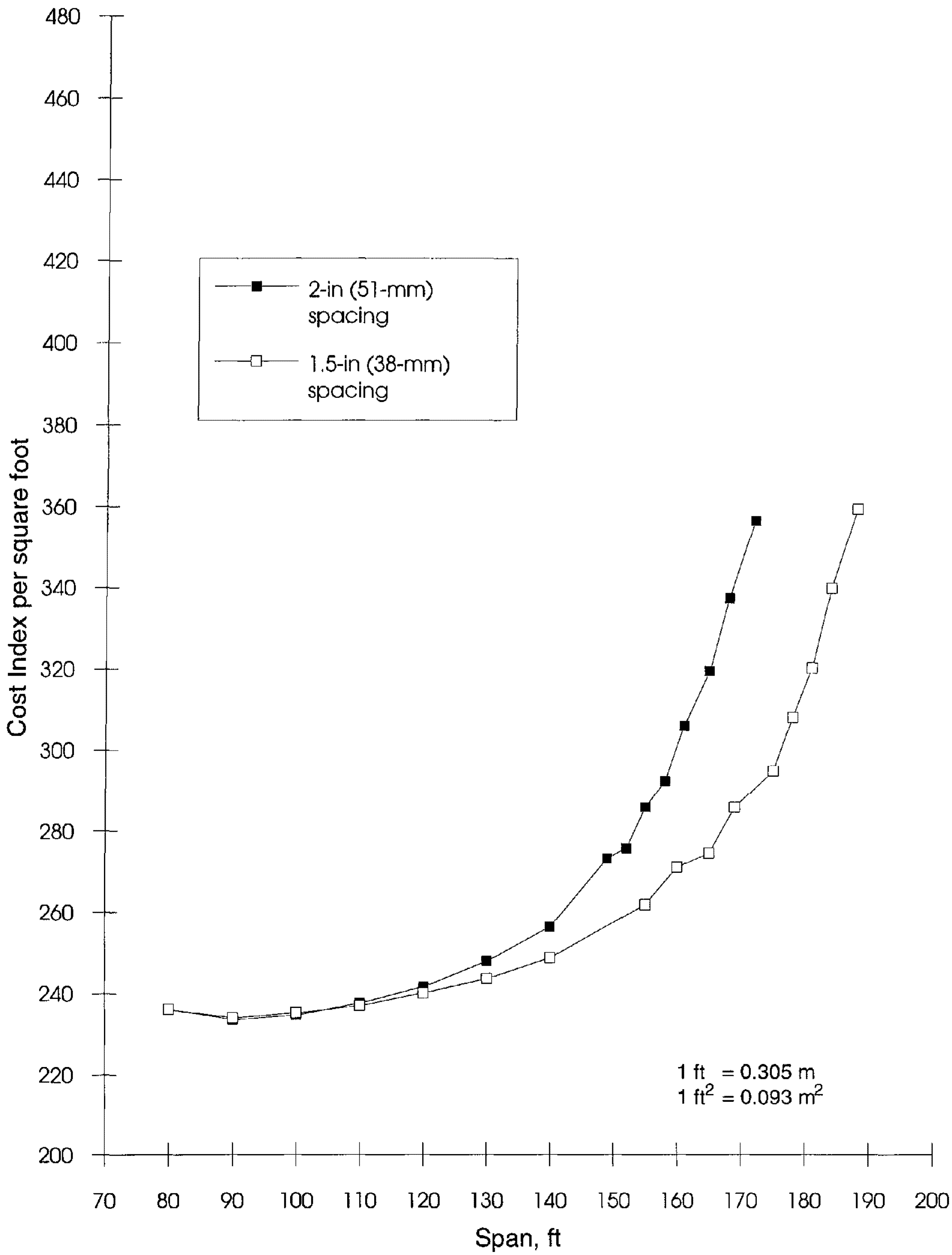


Figure 31. BT-72 with 0.5-in (12.7-mm) Diameter Strand at Various Spacings at 10,000 psi (69 MPa).

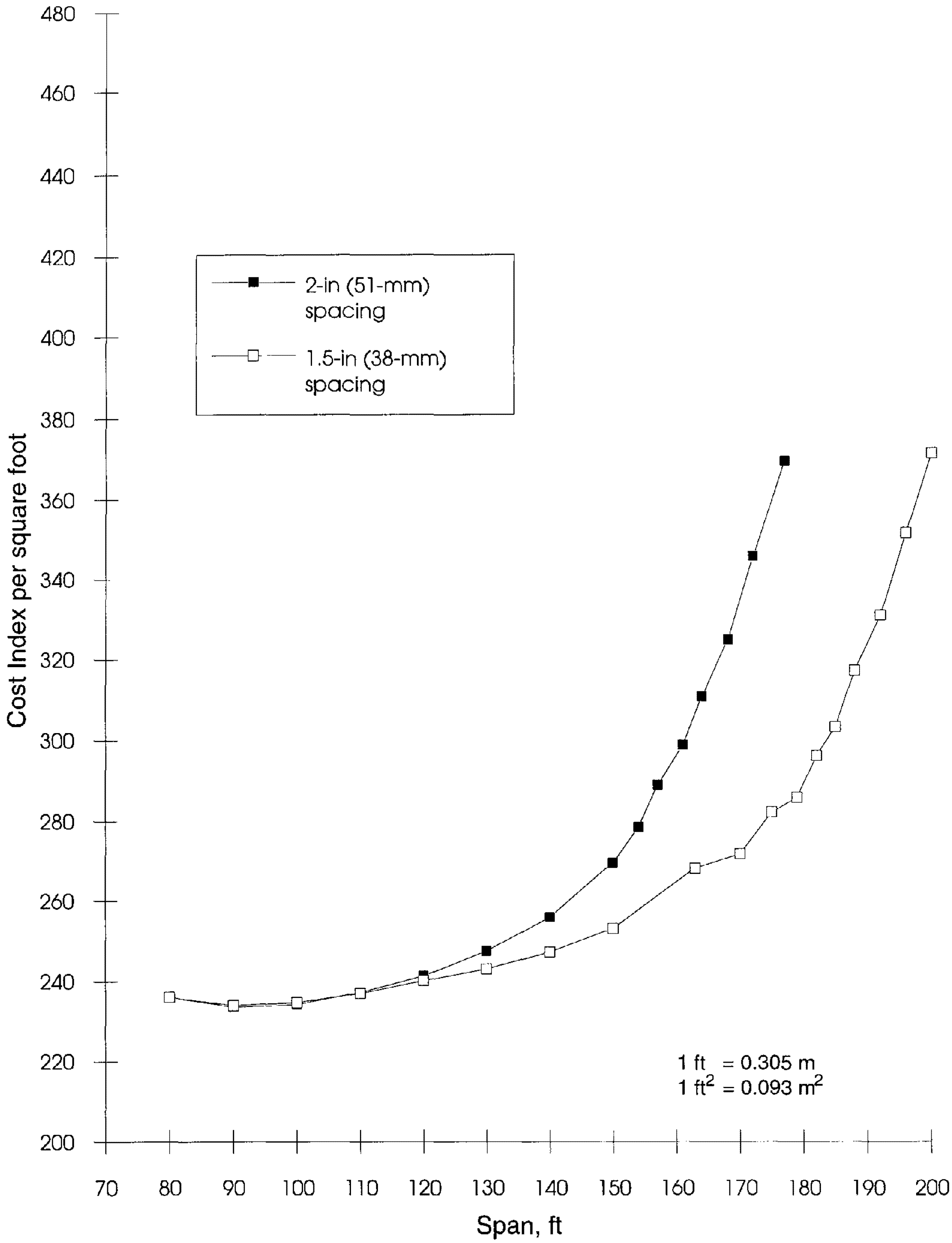


Figure 32. BT-72 with 0.5-in (12.7-mm) Diameter Strand at Various Spacings at 12,000 psi (83 MPa).

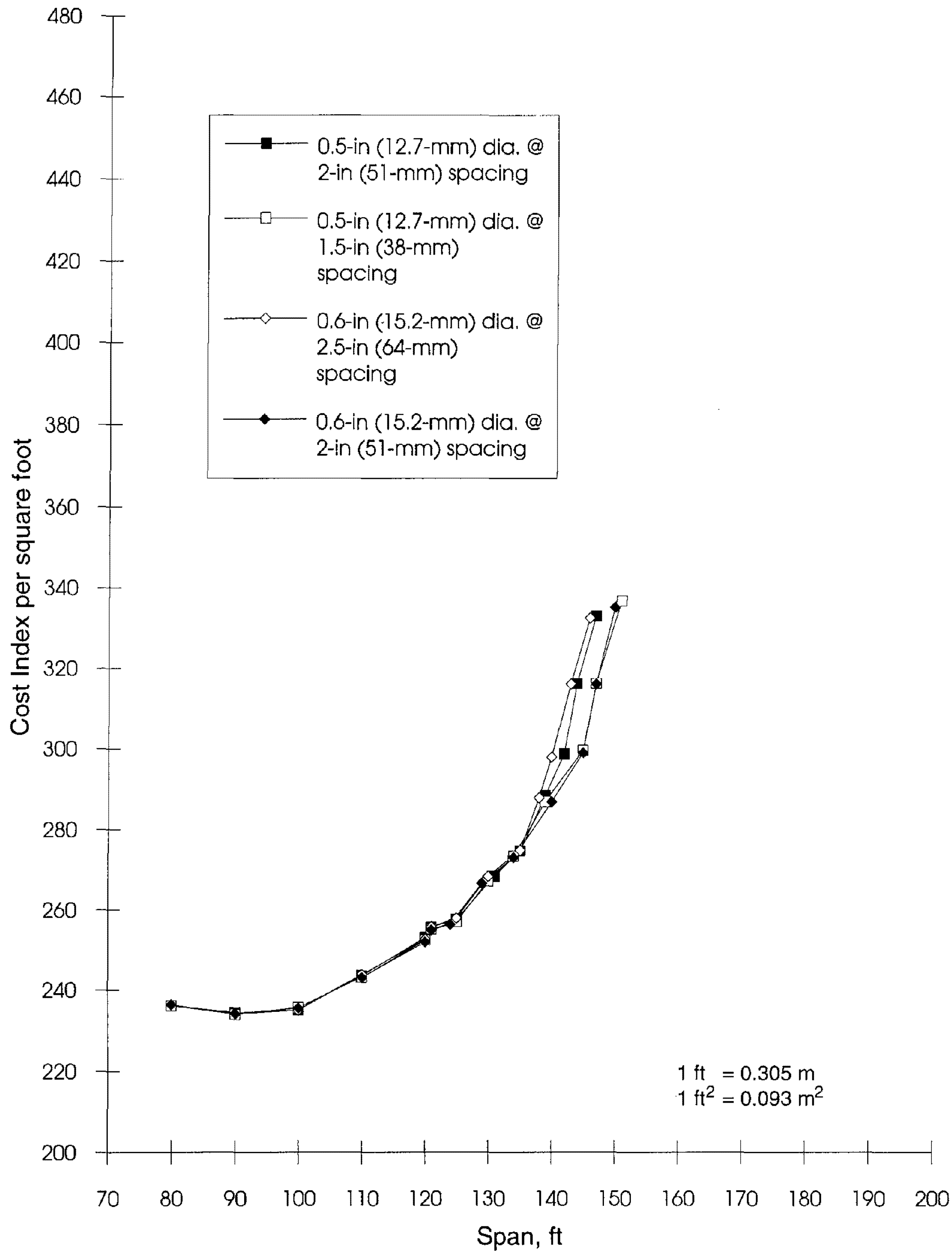


Figure 33. BT-72 with 0.5-in and 0.6-in (12.7-mm and 15.2-mm) Diameter Strand at Various Spacings at 6,000 psi (41 MPa).

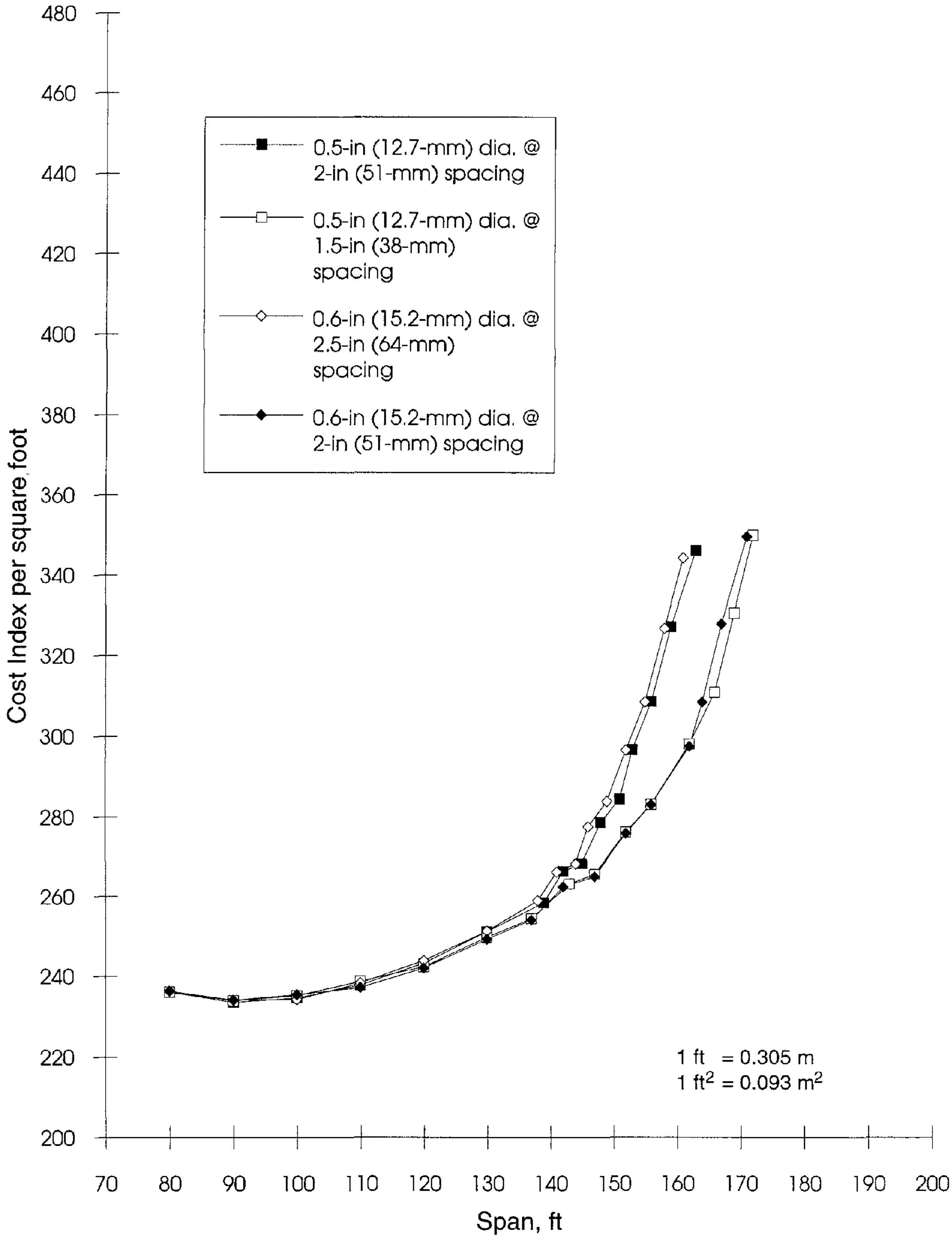


Figure 34. BT-72 with 0.5-in and 0.6-in (12.7-mm and 15.2-mm) Diameter Strand at Various Spacings at 8,000 psi (55 MPa).

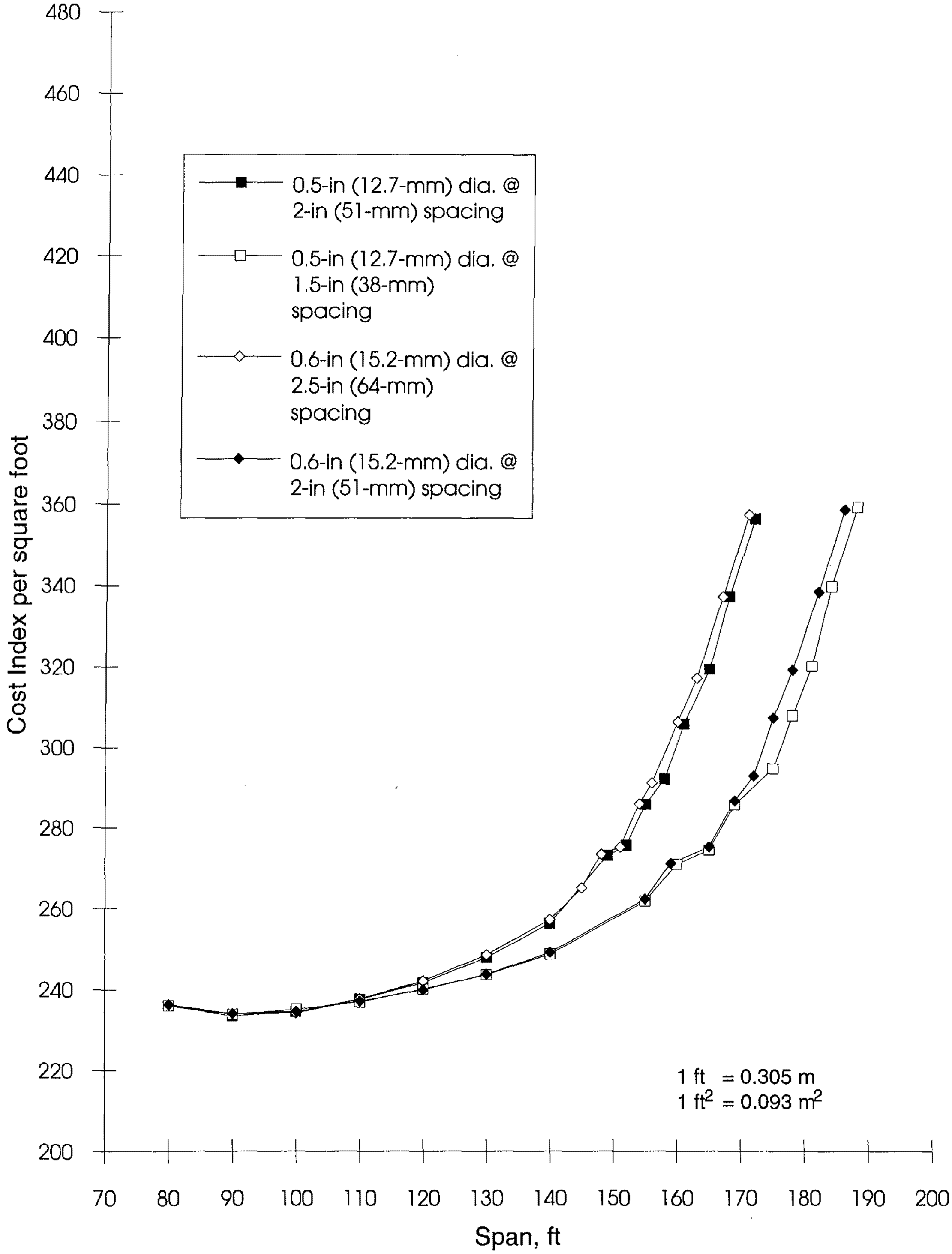


Figure 35. BT-72 with 0.5-in and 0.6-in (12.7-mm and 15.2-mm) Diameter Strand at Various Spacings at 10,000 psi (69 MPa).

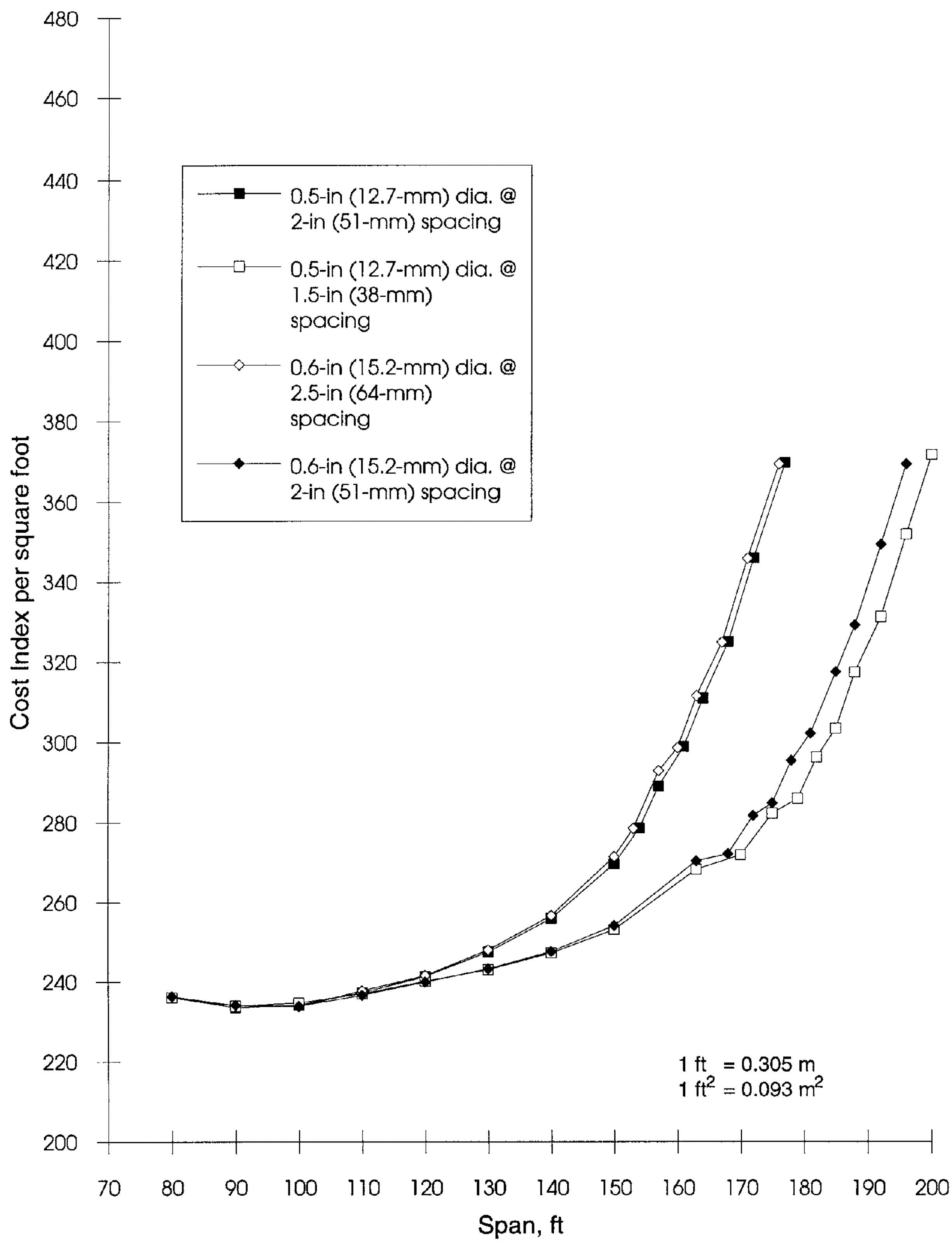


Figure 36. BT-72 with 0.5-in and 0.6-in (12.7-mm and 15.2-mm) Diameter Strand at Various Spacings at 12,000 psi (83 MPa).

with the 0.5-in (12.7-mm) diameter strands, the 0.6-in (15.2-mm) diameter strands spaced at approximately three diameters result in significant cost benefits over an approximate spacing of four diameters, particularly at higher concrete strengths.

The following modifications examined strand sizes greater than 0.6 in (15.2 mm) in diameter. These strand sizes are not readily available in the United States; however, the Japanese manufacture 270-ksi (1860-MPa) strands with diameters of 0.7 and 0.9 in (17.8 and 21.8 mm). Based on the discussion above, potential benefits of these strands for high-strength concrete can be examined by directly comparing the maximum areas of prestressing steel that can be placed within the bottom flange of a BT-72. Table 1 compares 0.5-, 0.6-, 0.7-, and 0.9-in (12.7-, 15.2-, 17.8-, and 21.8-mm) diameter strands at various spacings. Based on table 1, the 0.7-in (17.8-mm) diameter strand spaced at 2 in (51 mm) on center will result in the longest span lengths and most cost-effective structure, while the 0.9-in (21.8-mm) diameter strand spaced at 4 in (102 mm) on center will result in the shortest span lengths and least cost-effective structure. All other combinations within table 1 will fall somewhere in-between these extremes. Figures 37 and 38 verify this conclusion. Figures 37 and 38 compare the 0.7- and 0.9-in (17.8- and 21.8-mm) diameter strands, respectively, with the 0.5-in (12.7-mm) diameter strand at a girder concrete strength of 10,000 psi (69 MPa). The relative locations of the optimum cost curves in both figures follow the pattern established in table 1. It is noted that transfer and development length considerations could be a significant factor with the 0.7- and 0.9-in (17.8- and 21.8-mm) diameter strands.

STRAND STRENGTH

Another alternative to the bottom flange physical limitation on strand placement is to increase the strand strength; the purpose, again, is to allow greater prestressing within the bottom flange. The same 72-in (1830-mm) deep PCI Bulb-Tee served as the standardized cross section, and, again, the baseline condition was 270 ksi (1860 MPa), 0.5-in (12.7-mm) diameter prestressing strands spaced 2 in (51 mm) on center with a 2-in (51-mm) concrete surface to center of strand spacing. The 300-ksi (2070-MPa) strand, also 0.5 in (12.7 mm) in diameter with the same spacings, had the properties defined by the following idealized trilinear stress-strain curve:

FPY = 260,000 psi (1795 MPa), SY = FPY/EPS

FPM = 285,000 psi (1965 MPa), SM = 0.013

FPU = 300,000 psi (2070 MPa), SU = 0.040

where EPS = 28,000,000 psi (193 GPa)

Table 1. Maximum Bottom Flange Prestressing for a BT-72.

Strand Diameter (in)	Strand Spacing (in)		Maximum No. of Strands in Bottom Flange	Total Area of Prestressing Steel (in^2)
	Center-to-Center	Concrete Surface to Center		
0.5	1.5	2	61	9.33
0.5	2	2	39	5.97
0.6	2	2	39	8.46
0.6	2.5	2	25	5.43
0.7	2	2	39	12.60
0.7	3	3	12	3.88
0.9	3	3	12	5.82
0.9	4	4	5	2.43

1 in = 25.4 mm

1 in^2 = 645 mm^2

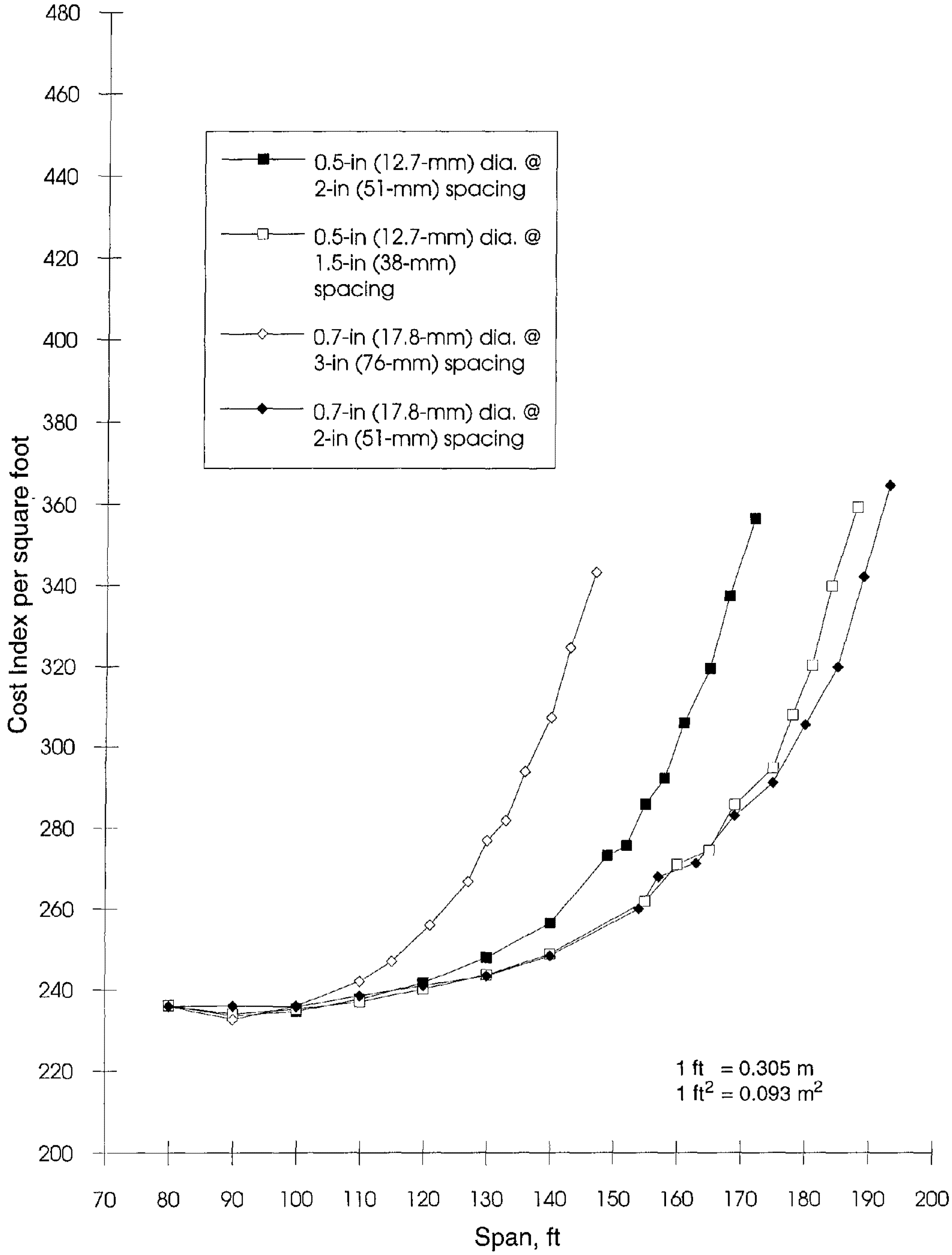


Figure 37. BT-72 with 0.5-in and 0.7-in (12.7-mm and 17.8-mm) Diameter Strand at Various Spacings at 10,000 psi (69 MPa).

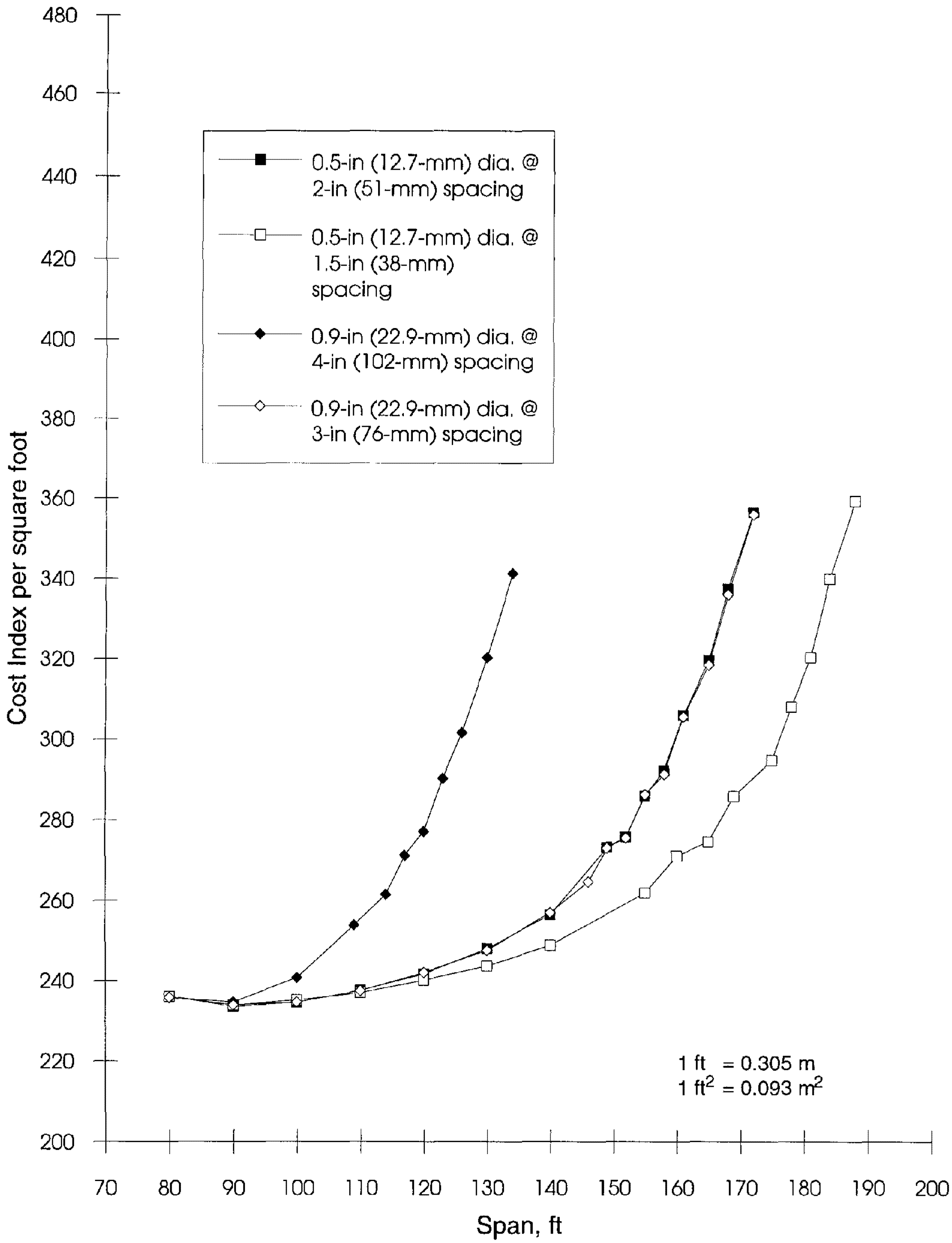


Figure 38. BT-72 with 0.5-in and 0.9-in (12.7-mm and 22.9-mm) Diameter Strand at Various Spacings at 10,000 psi (69 MPa).

The properties are based on 300-ksi (2070-MPa) Lo-Lax strand produced by Florida Wire and Cable, Inc. The variables are defined in figure 60 of Appendix C.

Figures 39 through 42 compare optimum cost curves at concrete strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa), respectively, when the strand strength is increased from 270 to 300 ksi (1860 to 2070 MPa). As can be seen from these figures, the strand strength increase has the largest impact on the 12,000-psi (83-MPa) plot and the smallest impact on the 6,000-psi (41-MPa) plot, with the 8,000- and 10,000-psi (55- and 69-MPa) plots falling somewhere between the two extremes. This behavior is again consistent with the previous conclusion that the bottom flange size limited the effectiveness of higher concrete strengths. With a greater strand strength, and ultimately greater prestressing force for the same area of steel, more effective prestressing can be placed within the cross section.

SECTION GEOMETRY

Another alternative to the bottom flange physical limitation on strand placement is to alter the section geometry of the bottom flange; the purpose is to allow a larger number of strands within the bottom flange and, thus, more efficient strand placement. The same 72-in (1830-mm) deep PCI Bulb-Tee served as the standardized cross section, and, again, the baseline condition used 270-ksi (1860-MPa), 0.5-in (12.7-mm) diameter prestressing strands spaced 2 in (51 mm) on center with a 2-in (51-mm) concrete surface to center of strand spacing. Two strands were allowed within each row of the web.

Two modifications of the bottom flange were studied. Modification no. 1 consisted of increasing the bottom flange edge thickness from 6 in (152 mm) to 8 in (203 mm) while maintaining the overall 72-in (1830-mm) section depth. Modification no. 2 consisted of increasing the bottom flange thickness from 6 in (152 mm) to 8 in (203 mm) by increasing the section's overall depth from 72 in to 74 in (1830 mm to 1880 mm). The two modifications are shown in figure 43.

Figures 44 through 47 compare optimum cost curves at concrete strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa), respectively, when the first modification is incorporated in the BT-72 bottom flange. As can be seen from these figures, modification no. 1 has the largest impact on the 12,000-psi (83-MPa) plot and the smallest impact on the 6,000-psi (41-MPa) plot, with the 8,000- and 10,000-psi (55- and 69-MPa) plots falling somewhere between the two extremes. In fact, at 6,000 psi (41 MPa), modification no. 1 results in a slightly less cost-effective section than the original BT-72. At 8,000 psi (55 MPa), the two sections are nearly identical in cost-effectiveness. Benefits of the bottom flange modification are eventually realized at concrete strengths in excess of 8,000 psi (55 MPa) and

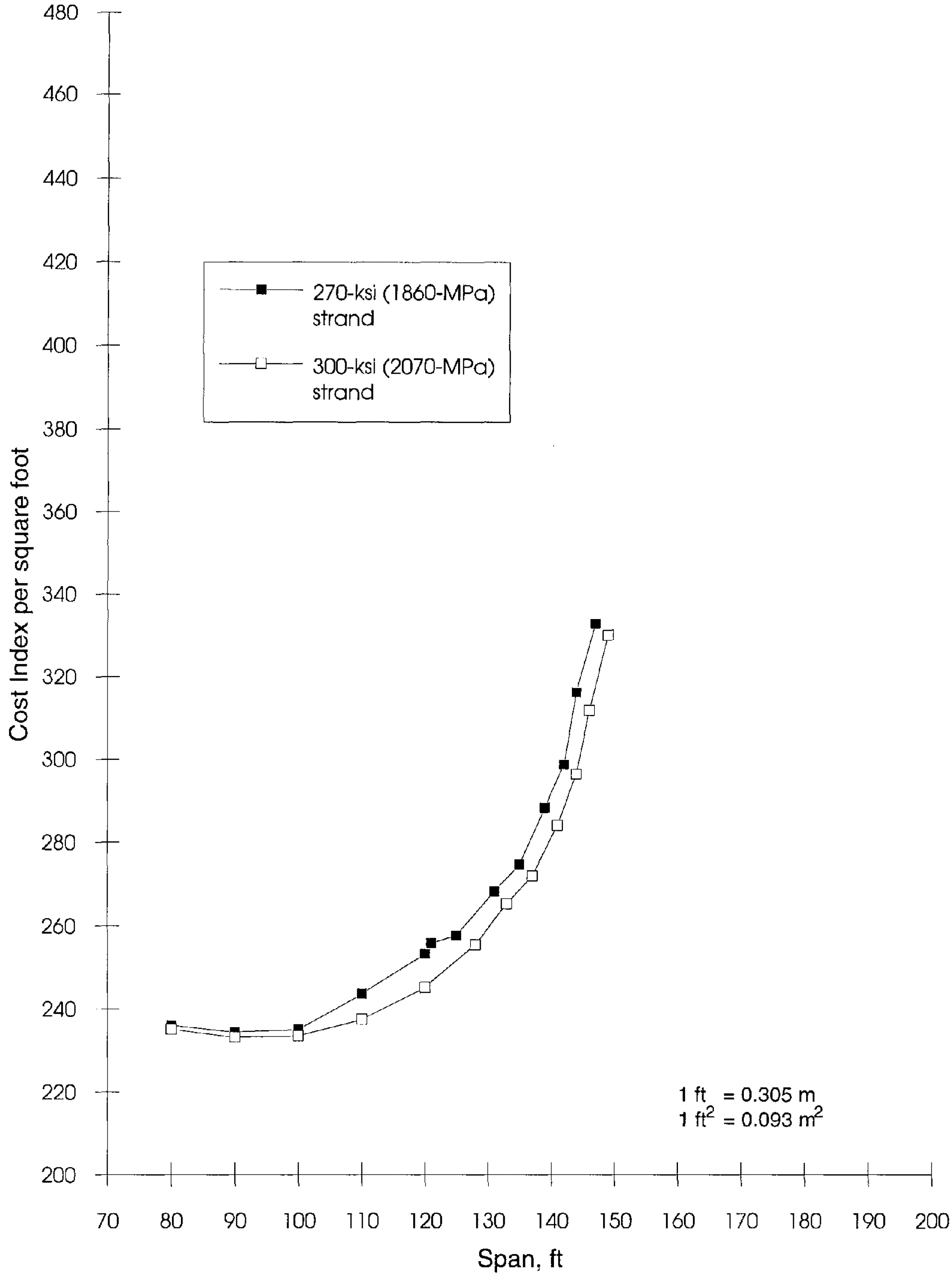


Figure 39. BT-72 with 270- and 300-ksi (1860- and 2070-MPa) Strand at 6,000 psi (41 MPa).

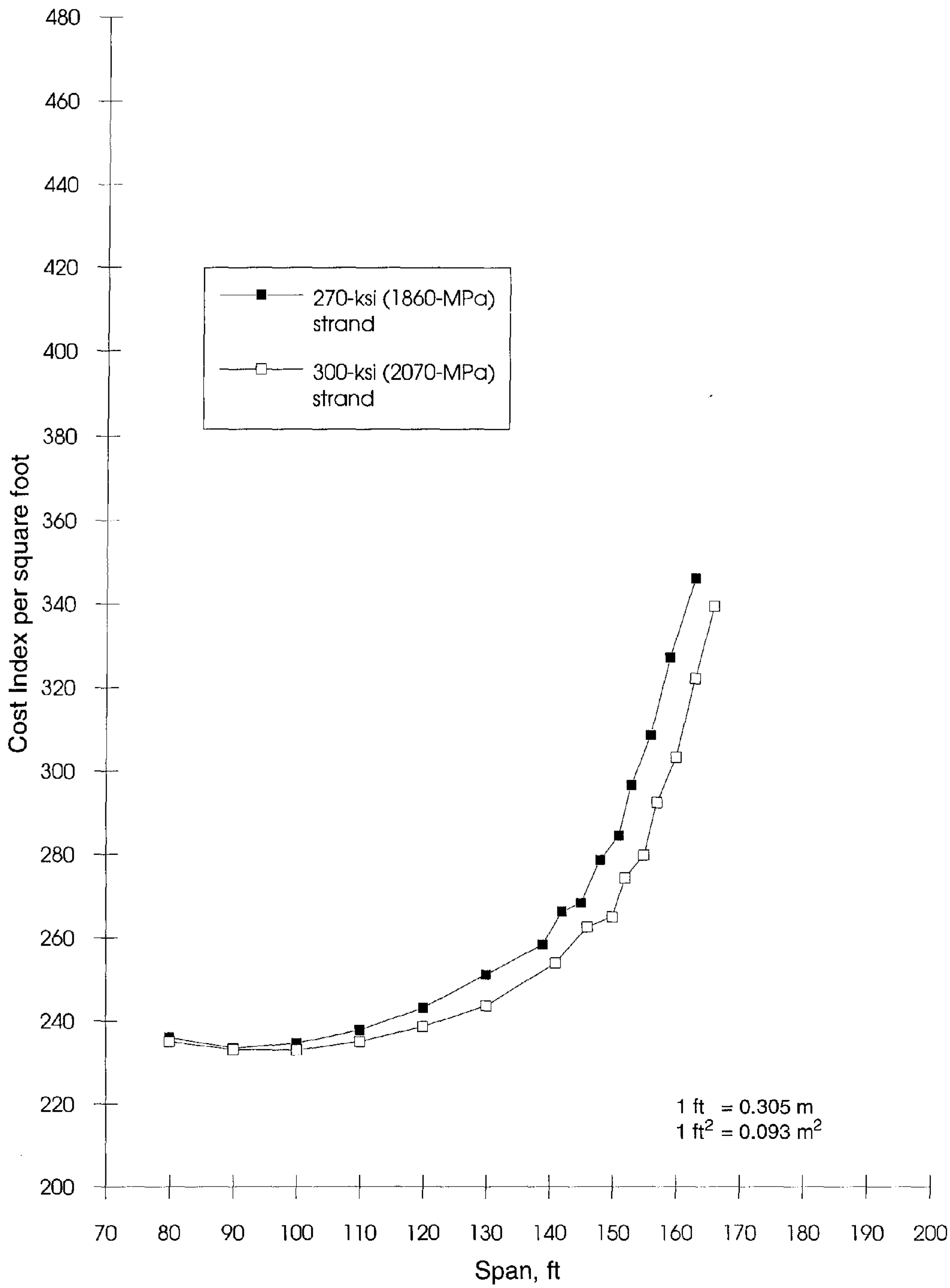


Figure 40. BT-72 with 270- and 300-ksi (1860- and 2070-MPa)Strand at 8,000 psi (55 MPa).

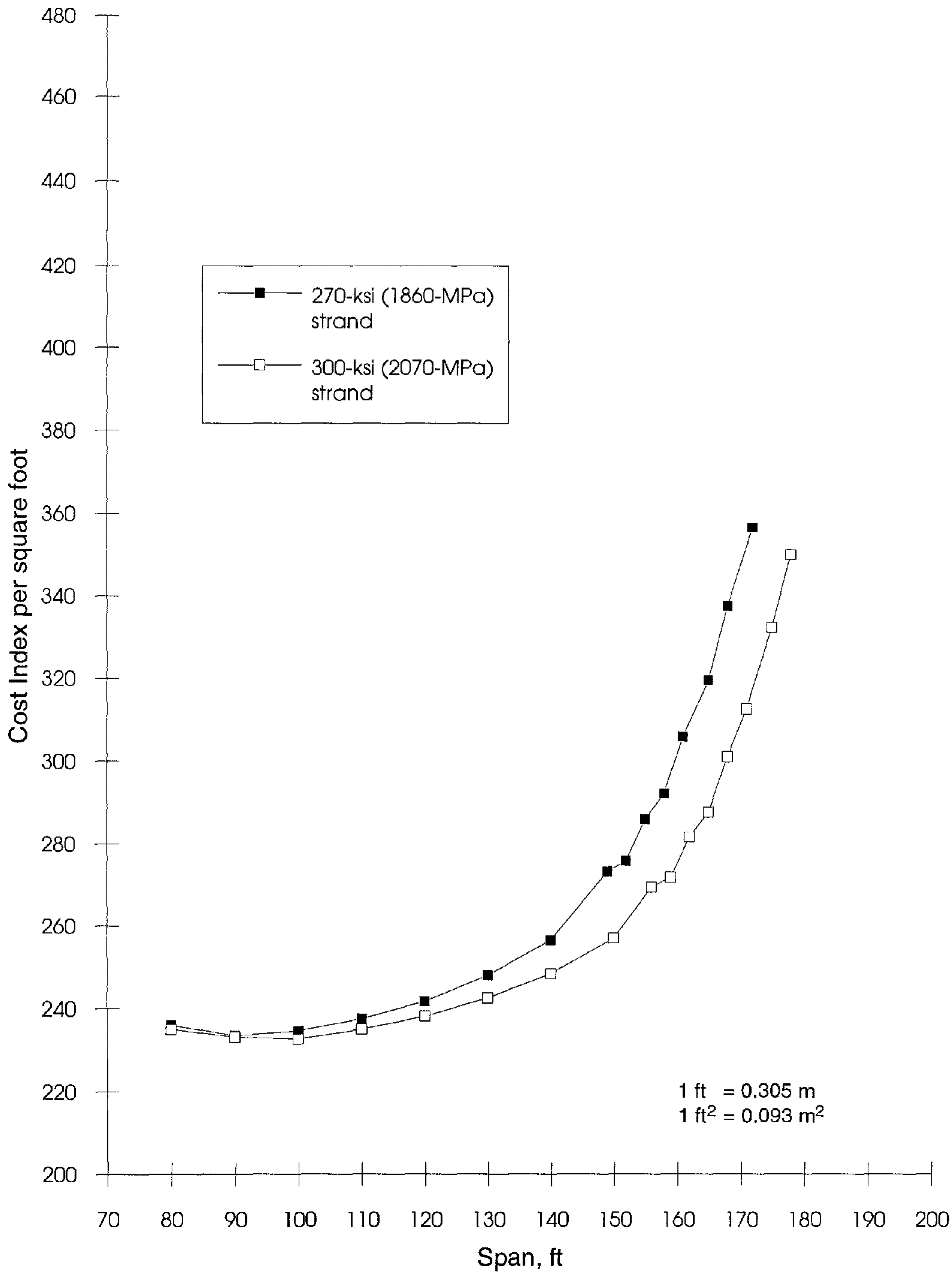


Figure 41. BT-72 with 270- and 300-ksi (1860- and 2070-MPa) Strand at 10,000 psi (69 MPa).

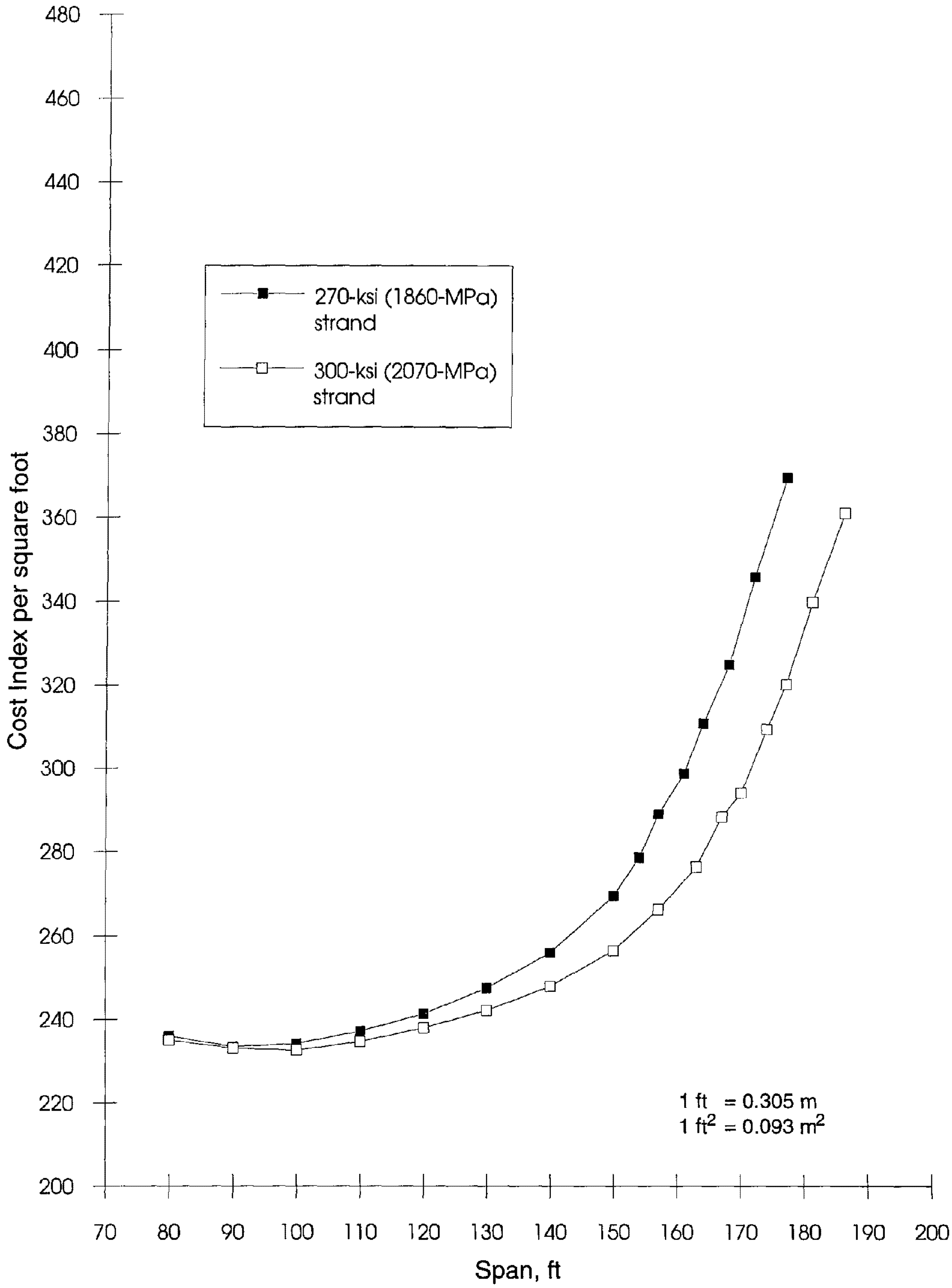
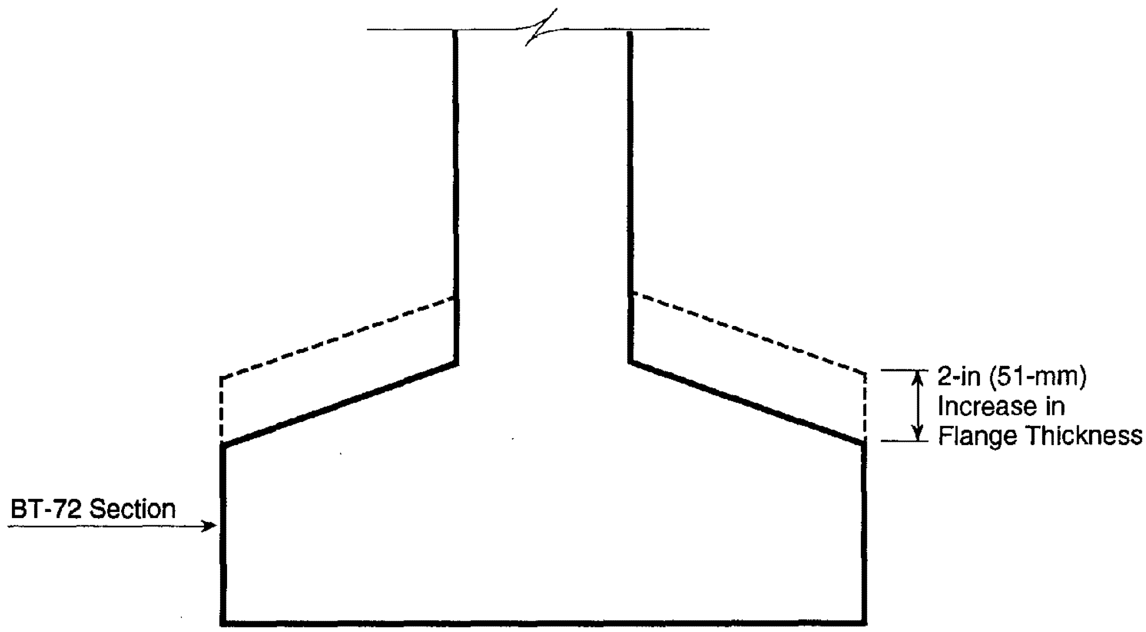
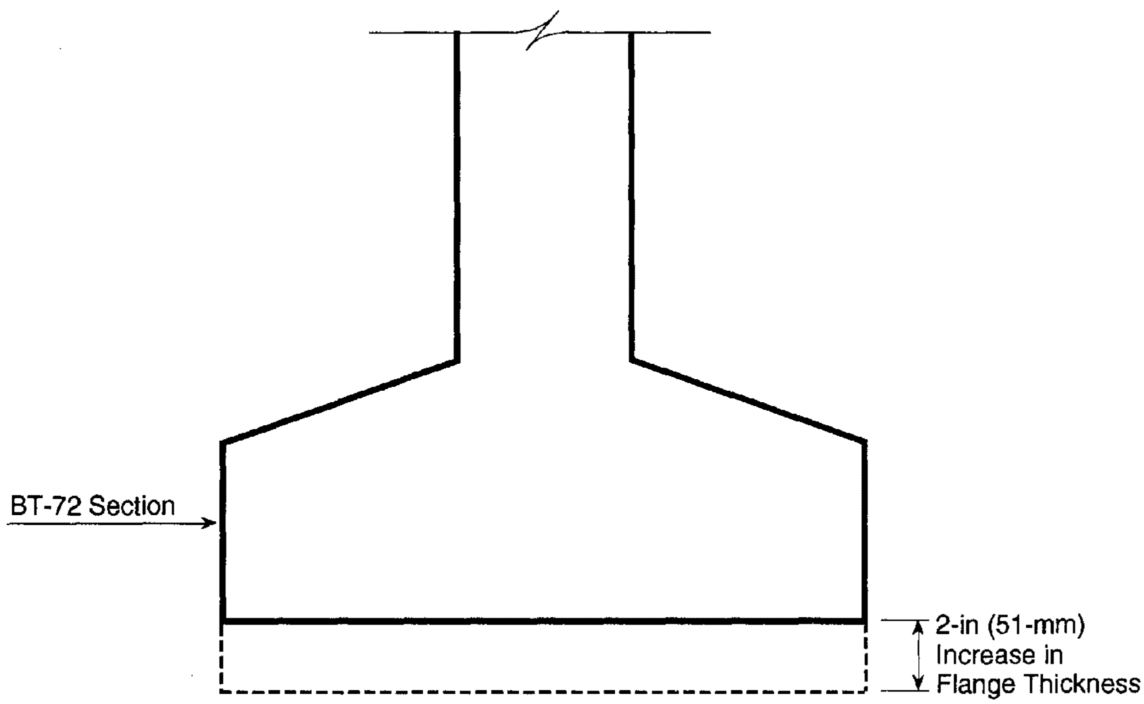


Figure 42. BT-72 with 270- and 300-ksi (1860- and 2070-MPa) Strand at 12,000 psi (83 MPa).



a) No. 1 - Maintain Overall 72-in (1830-mm) Depth.



b) No. 2 - Increase Depth From 72 in to 74 in (1830 mm to 1880 mm).

Figure 43. BT-72 Bottom Flange Modifications.

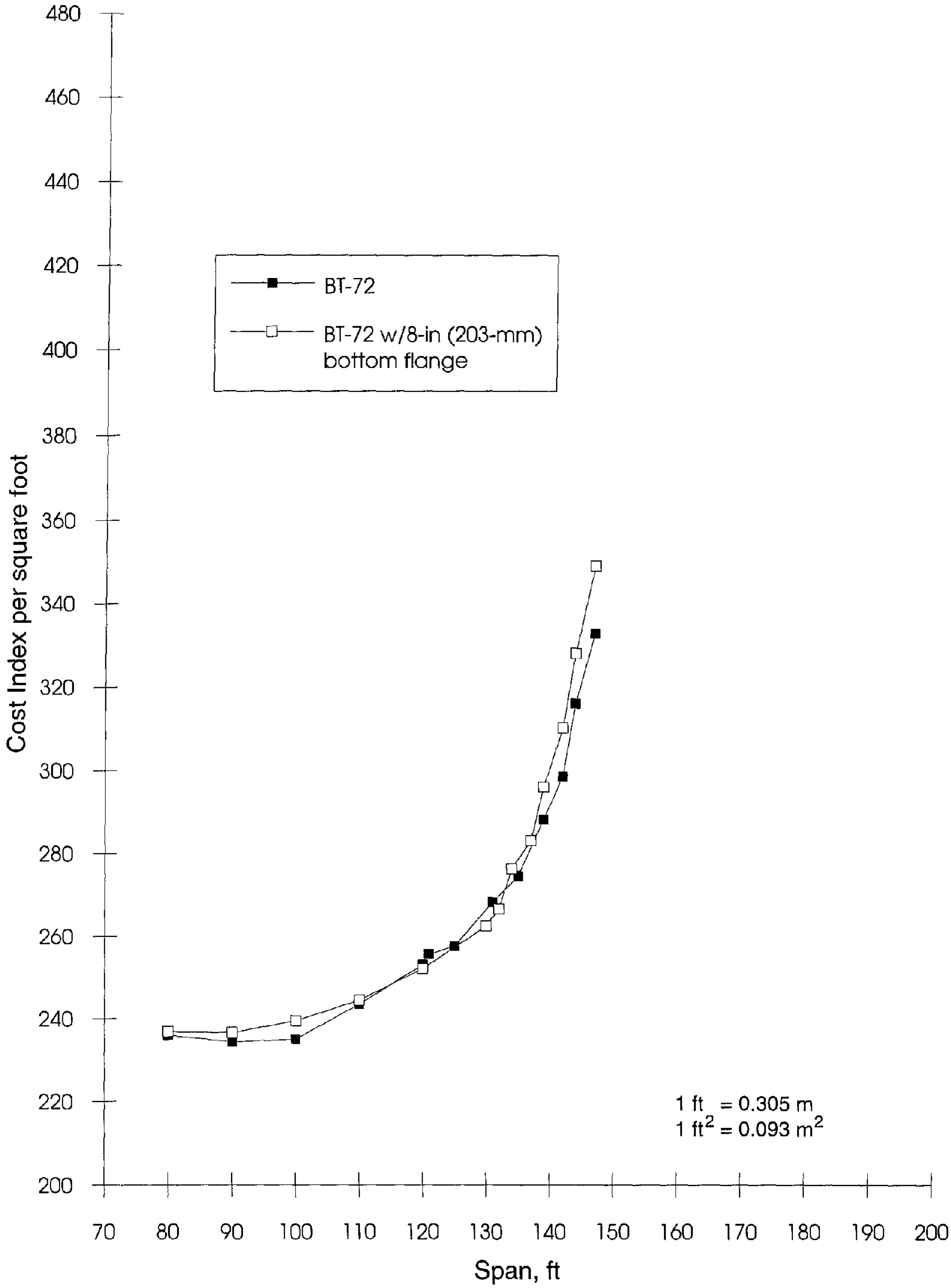


Figure 44. BT-72 Bottom Flange Modification No. 1 at 6,000 psi (41 MPa).

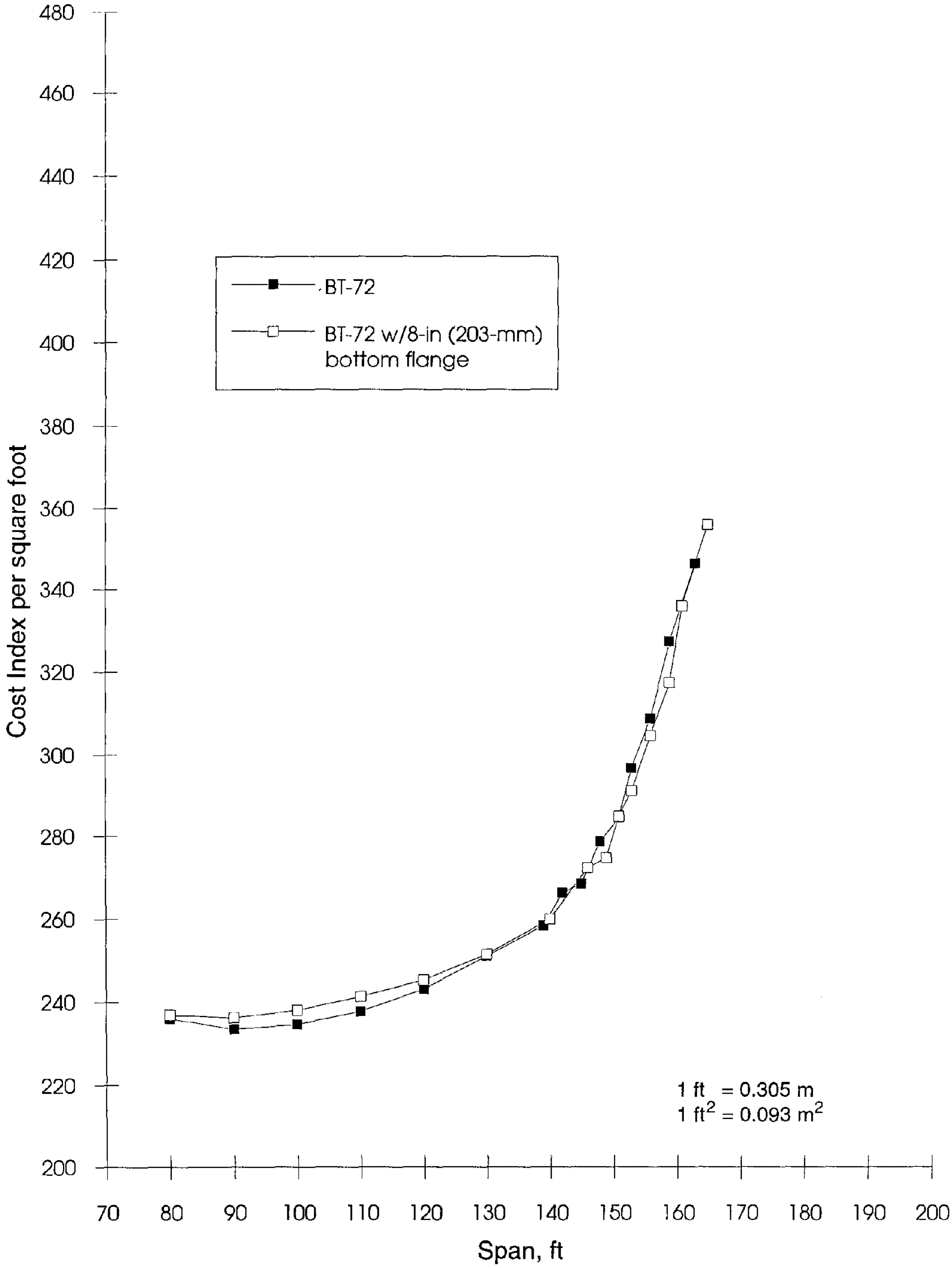


Figure 45. BT-72 Bottom Flange Modification No. 1 at 8,000 psi (55 MPa).

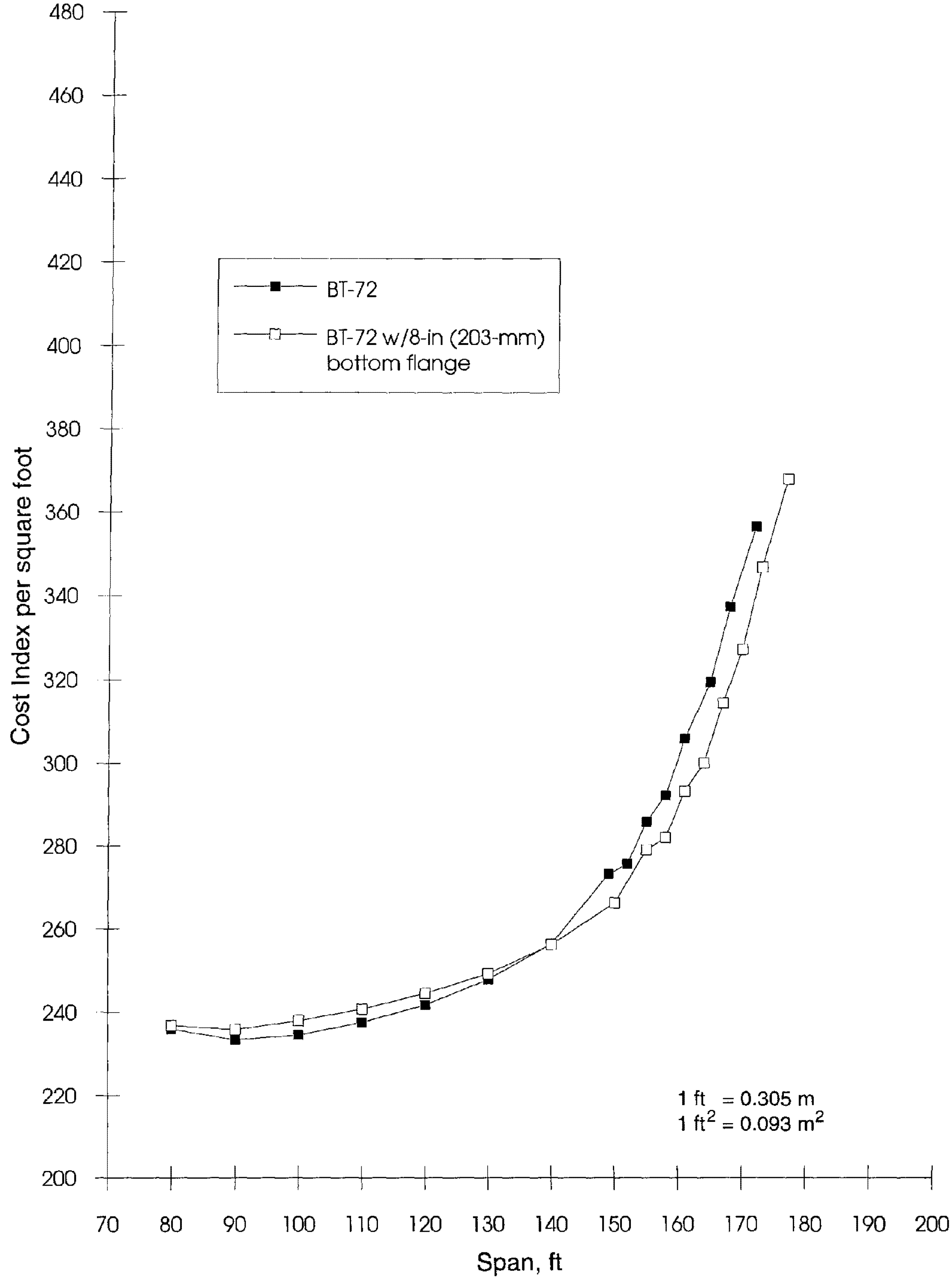


Figure 46. BT-72 Bottom Flange Modification No. 1 at 10,000 psi (69 MPa).

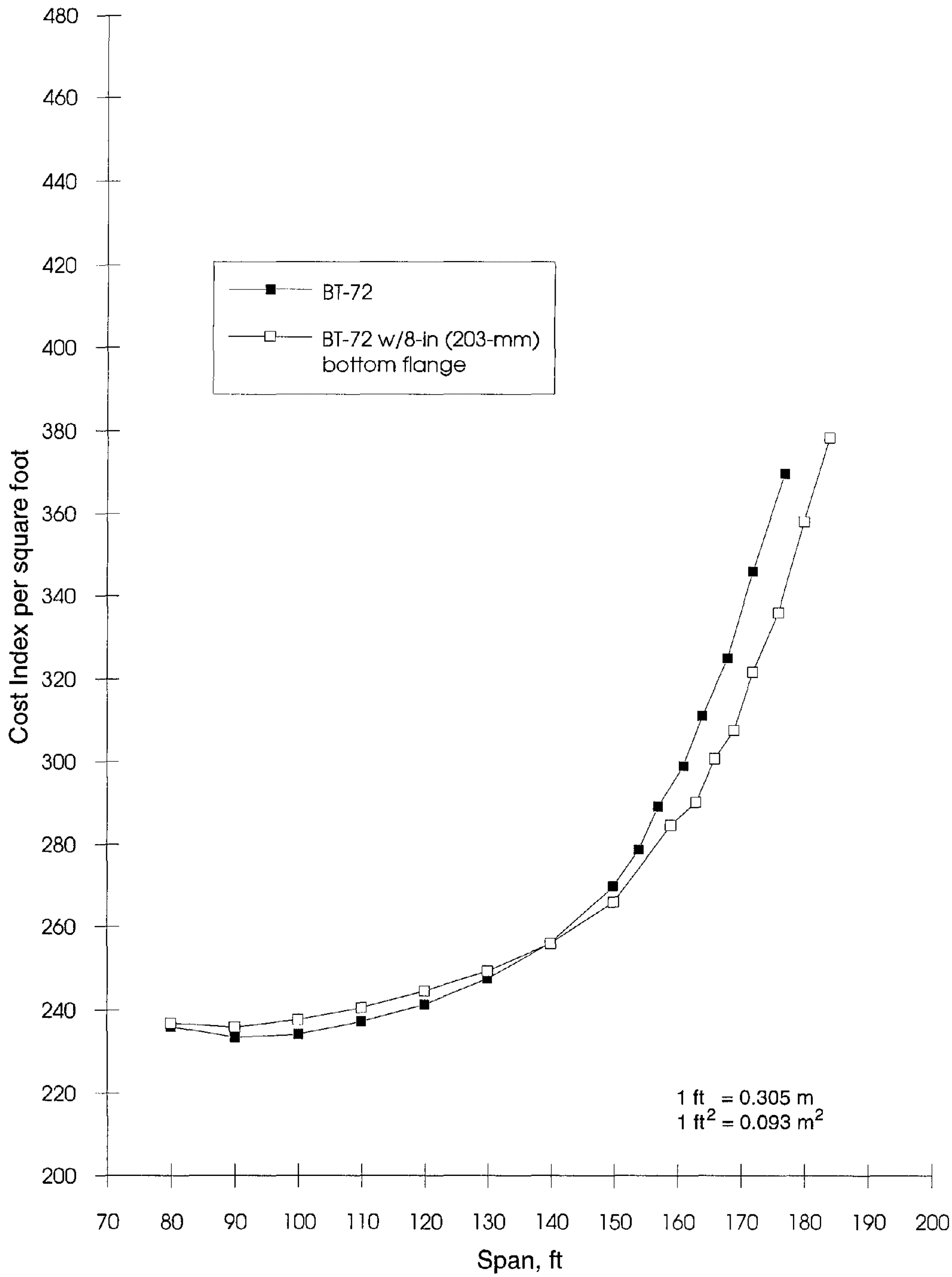


Figure 47. BT-72 Bottom Flange Modification No. 1 at 12,000 psi (83 MPa).

span lengths in excess of 140 ft (42.7 m). This fact is similar to the benefits observed for the FL BT-72 and the modified Type VI.

The behavior of the optimum cost curve for modification no. 1 is consistent with the previous conclusion that the bottom flange limited the effectiveness of higher concrete strengths; however, the behavior is slightly different than that which was experienced with modifications to strand size, spacing, and strength. A penalty is paid in the form of increased volume of concrete and corresponding weight when the bottom flange size is increased to incorporate more prestressing, as opposed to reducing strand spacing to obtain the same result. This penalty offsets some of the potential benefits of more prestressing within the bottom flange, and, consequently, cost benefits are not realized until concrete strengths exceed 8,000 psi (55 MPa). In addition, this penalty results in the original BT-72 being more cost-effective at all concrete strengths for span lengths of 140 ft (42.7 m) and less.

Figures 48 through 51 compare optimum cost curves for modification nos. 1 and 2 at concrete strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa), respectively. As can be seen from these figures, modification no. 2 is a slightly more cost-effective alternative than modification no. 1. However, it is interesting to note that the shift in the optimum cost curve between modification no. 1 and modification no. 2 is virtually identical at all concrete strengths. This fact occurs because the benefit of modification no. 2 over modification no. 1 is only a slightly deeper section. Both revised sections accommodate the same maximum number of prestressing strands within their larger bottom flange.

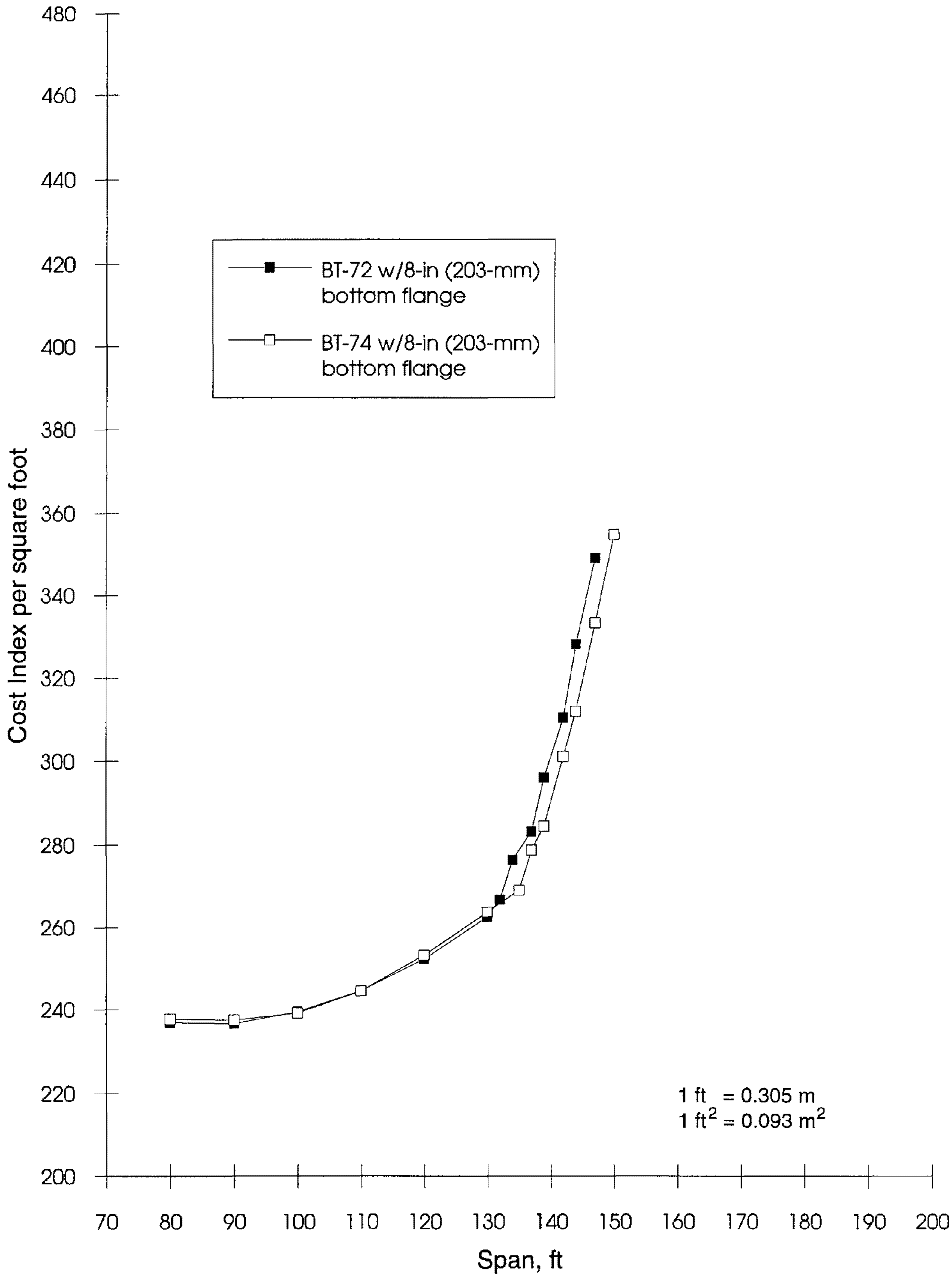


Figure 48. Comparison of BT-72 and BT-74 Flange Modifications at 6,000 psi (41 MPa).

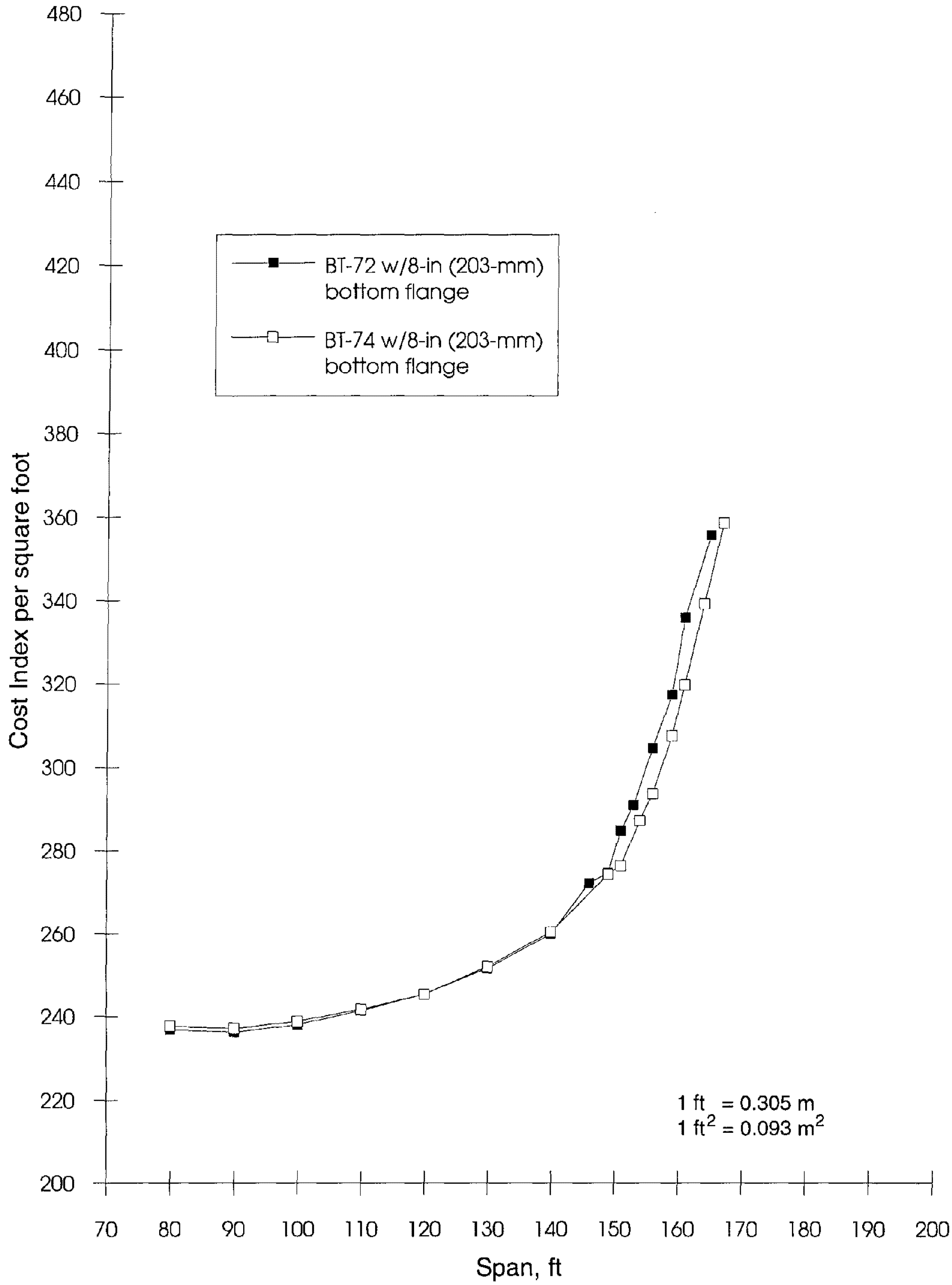


Figure 49. Comparison of BT-72 and BT-74 Flange Modifications at 8,000 psi (55 MPa).

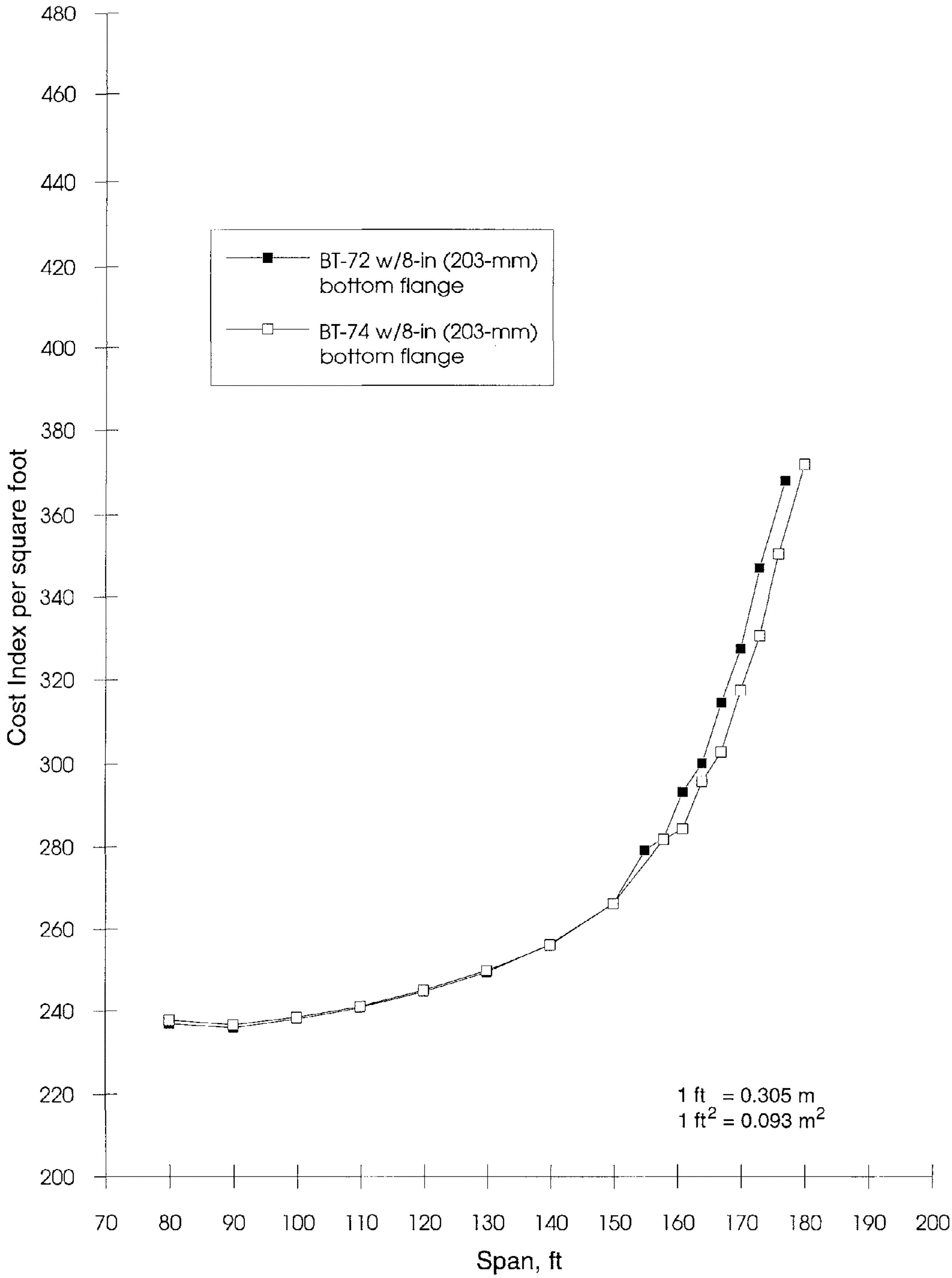


Figure 50. Comparison of BT-72 and BT-74 Flange Modifications at 10,000 psi (69 MPa).

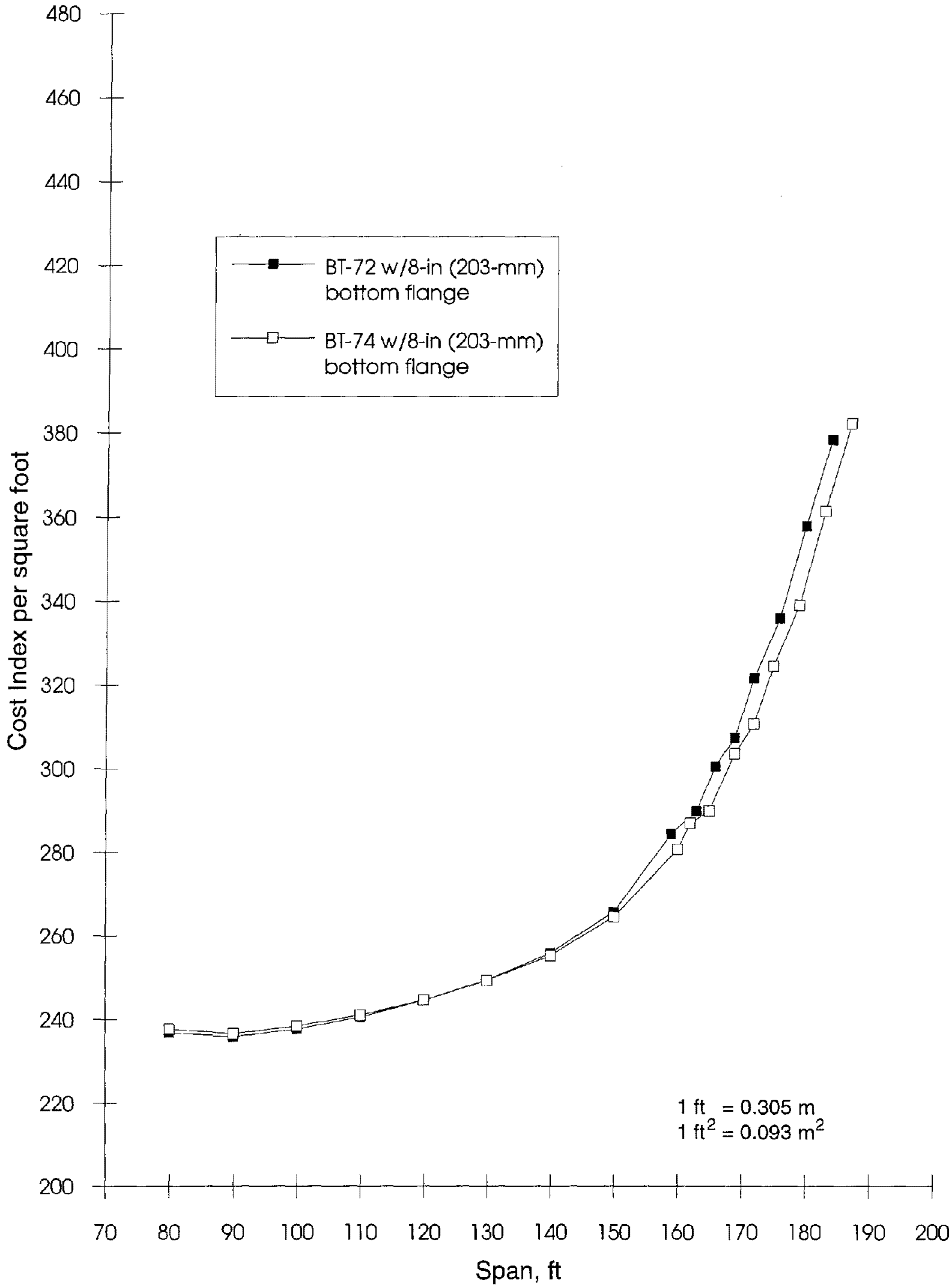


Figure 51. Comparison of BT-72 and BT-74 Flange Modifications at 12,000 psi (83 MPa).

4. ANALYSES OF POST-TENSIONED GIRDERS

Chapter 3 analyses investigated various parameters to determine the most cost-effective way of overcoming the physical limitations of the section's bottom flange. These analyses studied strand spacing, strand size, strand strength, and section geometry. All of the approaches focused on increasing the prestressing force while maintaining maximum eccentricity. In general, strand spacing and size variations offered the most promise, while modifications considered for the section's bottom flange geometry offered the least. However, all approaches extended the beneficial effects of high-strength concrete in precast, prestressed bridge girder construction described in Chapter 2.

Another alternative to overcoming the bottom flange physical limitation is the use of post-tensioning. Instead of utilizing numerous small-diameter strands cast within the bottom flange, a post-tensioned section involves the use of large-diameter tendons, constructed of multiple steel strands, installed within ducts cast within the section. A post-tensioning system may offer some advantages over pretensioning, and the following analyses investigated the alternative of post-tensioning applied to precast, prestressed bridge girders constructed of high-strength concrete.

The post-tensioned analyses used a 6-19 tendon, which consists of nineteen 0.6-in (15.2-mm) diameter strands for a total cross-sectional area of 4.12 in^2 (2660 mm^2) per tendon. The first, second, and third tendons were located 4, 10, and 16 in (102, 254, and 406 mm), respectively, above the bottom of the girder, as shown in figure 52. As in Chapter 3, the 72-in (1830-mm) deep PCI Bulb-Tee served as the standardized cross section, except that a 7-in (178-mm) thick web was used for the post-tensioned analyses in order to allow for adequate concrete cover and consolidation. The baseline condition still used the 6-in (152-mm) thick web with 270-ksi (1860-MPa), 0.5-in (12.7-mm) diameter prestressing strands spaced 2 in (51 mm) on center with a 2-in (51-mm) concrete surface to center of strand spacing.

Figures 53 through 56 compare optimum cost curves at concrete strengths of 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa), respectively, for a pretensioned and post-tensioned BT-72 as described above. One important aspect of the plots to be observed is the abrupt shifts in the post-tensioned optimum cost curves. These shifts occur when the girder concrete strength has attained a sufficient level to allow a second or third tendon to be added. For instance, at a concrete strength of 6,000 psi (41 MPa), the post-tensioned optimum cost curve is relatively smooth and represents a girder with only one 6-19 tendon. However, at a concrete strength of 8,000 psi (55 MPa), two tendons can be added to the section for span lengths exceeding 120 ft (36.6 m), which corresponds to the abrupt shift in the curve at this point. Correspondingly, three tendons can be used for a concrete strength of 10,000 psi

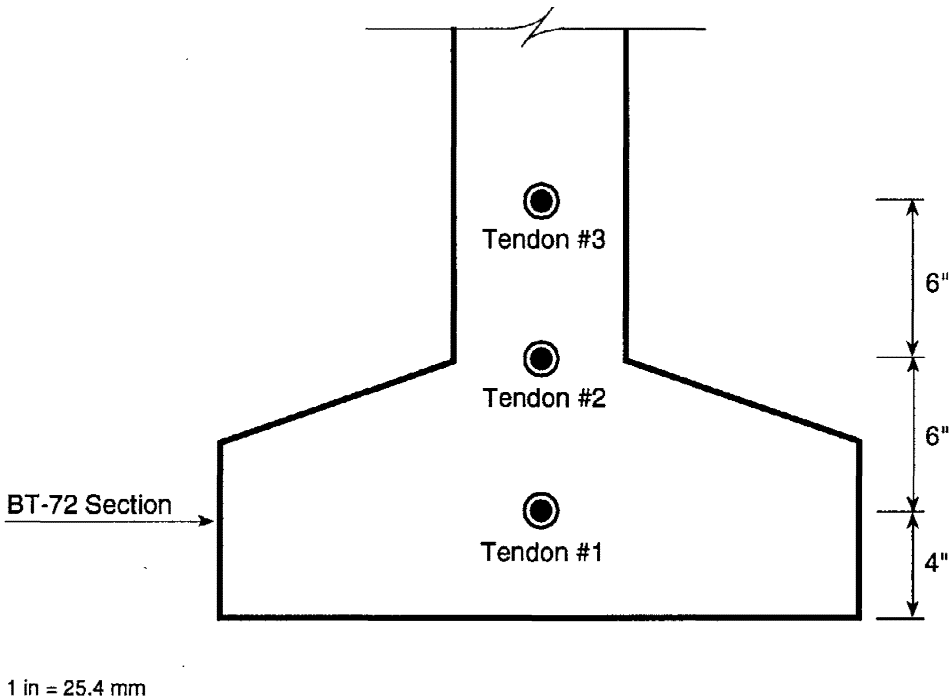


Figure 52. Post-Tensioning Tendon Layout.

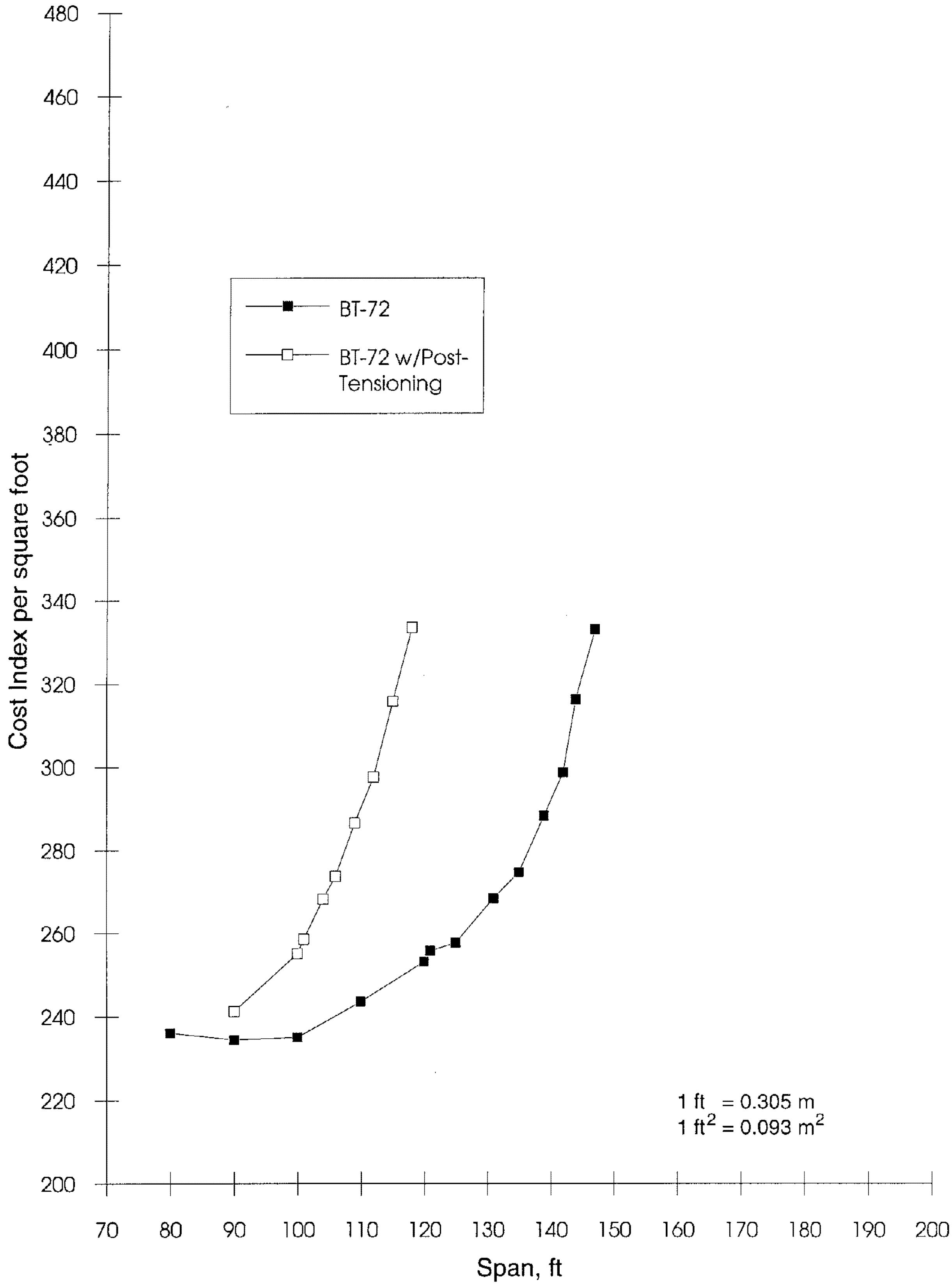


Figure 53. BT-72 Post-Tensioning Comparison at 6,000 psi (41 MPa).

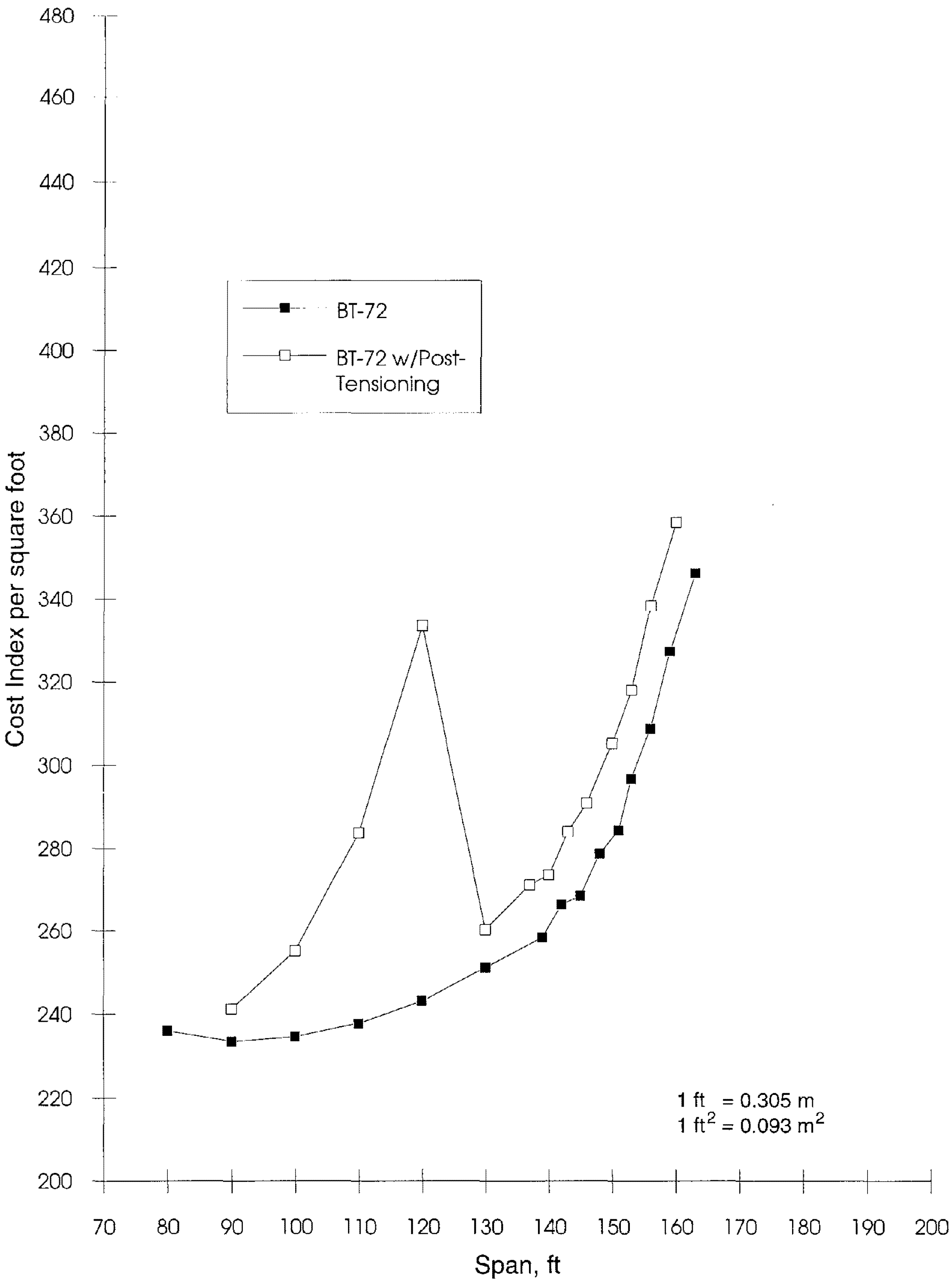


Figure 54. BT-72 Post-Tensioning Comparison at 8,000 psi (55 MPa).

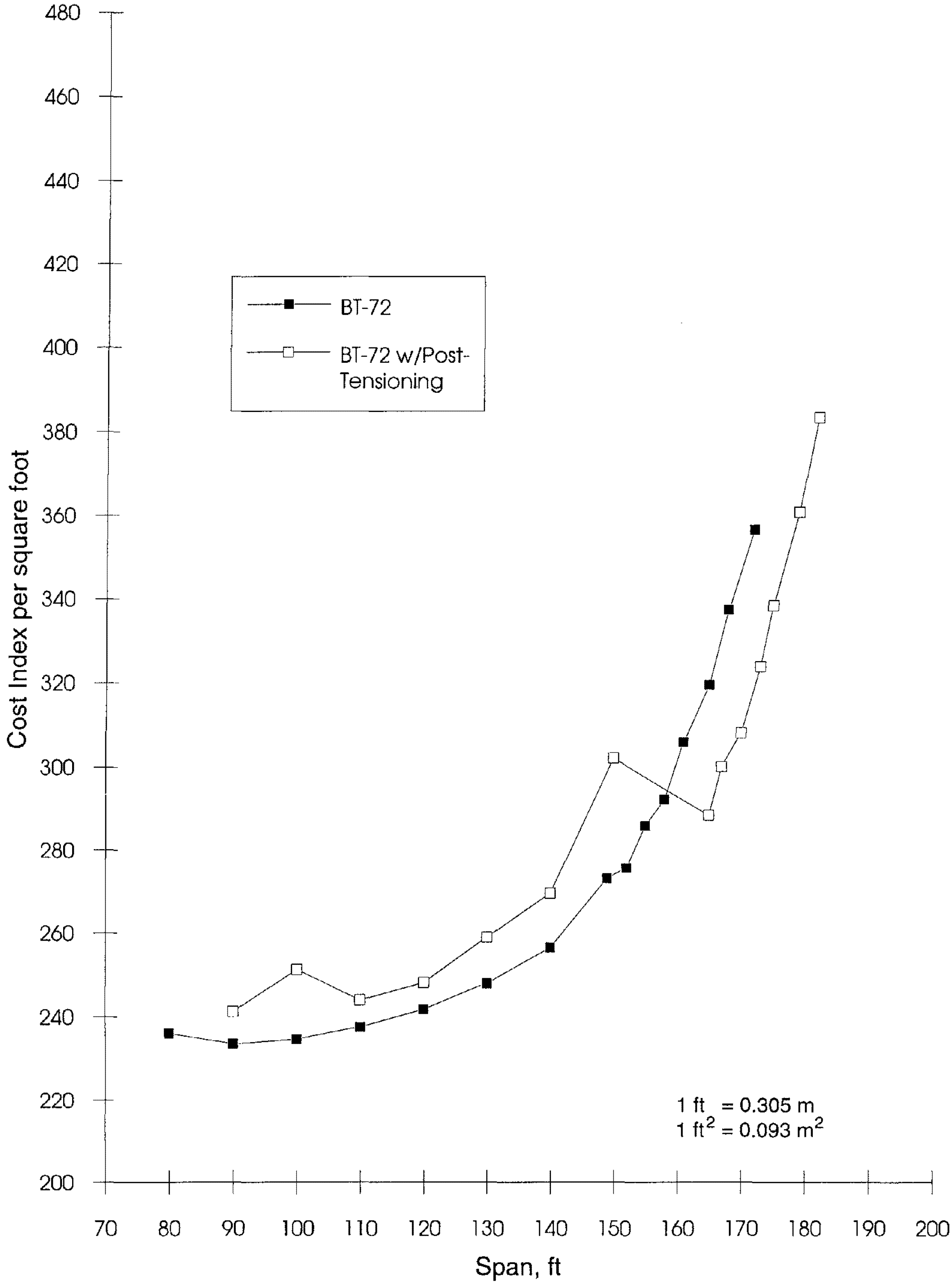


Figure 55. BT-72 Post-Tensioning Comparison at 10,000 psi (69 MPa).

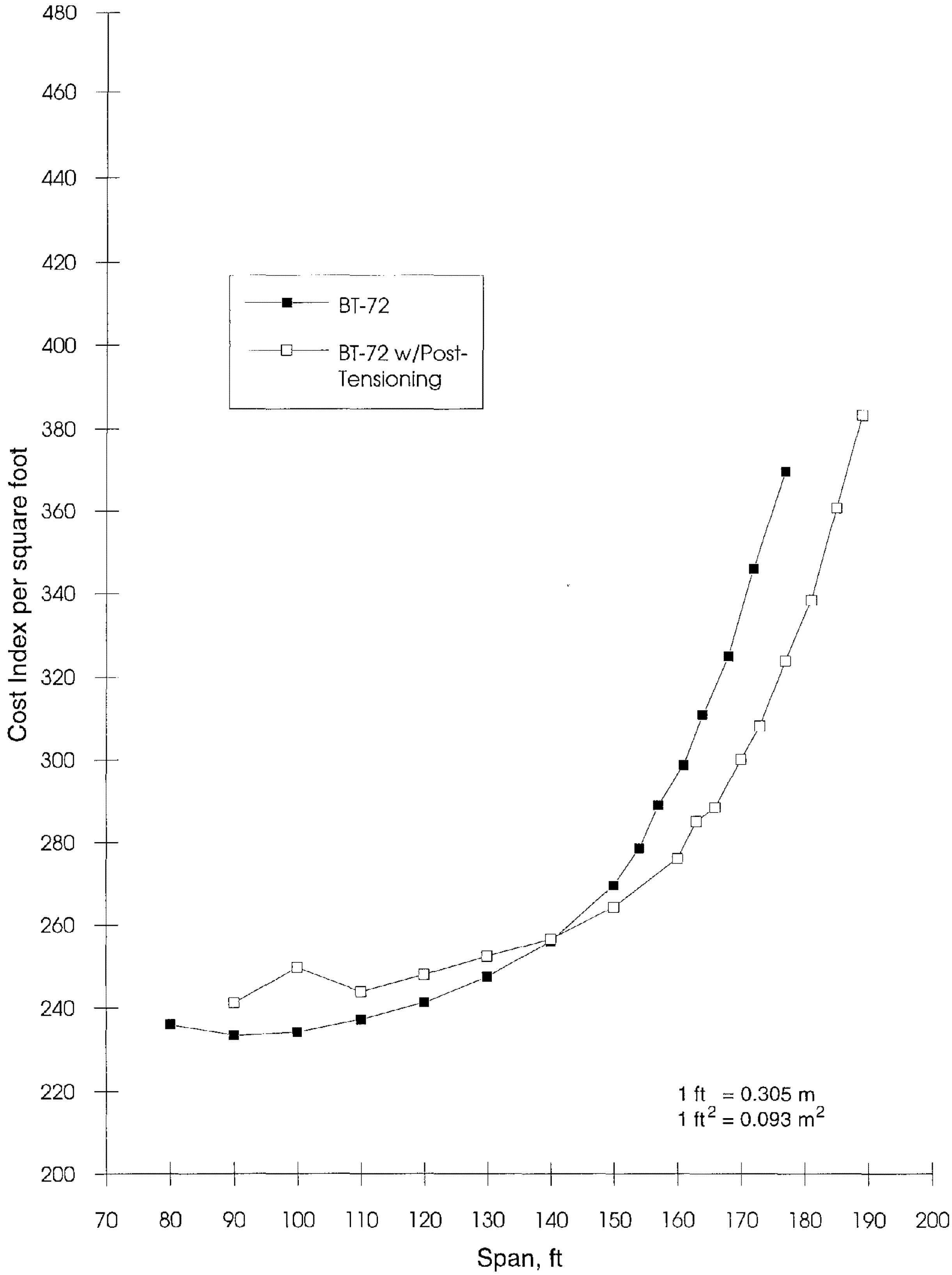


Figure 56. BT-72 Post-Tensioning Comparison at 12,000 psi (83 MPa).

(69 MPa) at span lengths exceeding 150 ft (45.7 m). It is at this point, as shown in figure 55, that the post-tensioned girder achieves a cost and span length superiority over the pretensioned section.

The reason for this superiority is that at this point, the three post-tensioning tendons provide a greater amount of prestressing force at a greater eccentricity than the pretensioning strands. This result is consistent with the conclusion developed in Chapters 2 and 3 that the bottom flange size limited the effectiveness of higher concrete strengths. With three tendons, the post-tensioned section takes greater advantage of the high-strength concrete. If fact, at a concrete strength of 12,000 psi (83 MPa), as shown in figure 56, the post-tensioned section is superior to the pretensioned section at span lengths exceeding 140 ft (42.7 m), with cost benefits ranging from 2 to 14 percent. The benefits of post-tensioning would have been more significant if the standard 6-in (152-mm) web thickness could have been used, as opposed to the 7-in (178-mm) web thickness selected for the above analyses to allow for adequate concrete cover and consolidation. Further consideration should be given to the concept of a hybrid section that combines pretensioned strands with post-tensioned tendons to produce simple span or continuous structure.

5. CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Based on the cost analyses described in this report, the following conclusions are made.

Effect of Concrete Strength on Existing Cross Sections

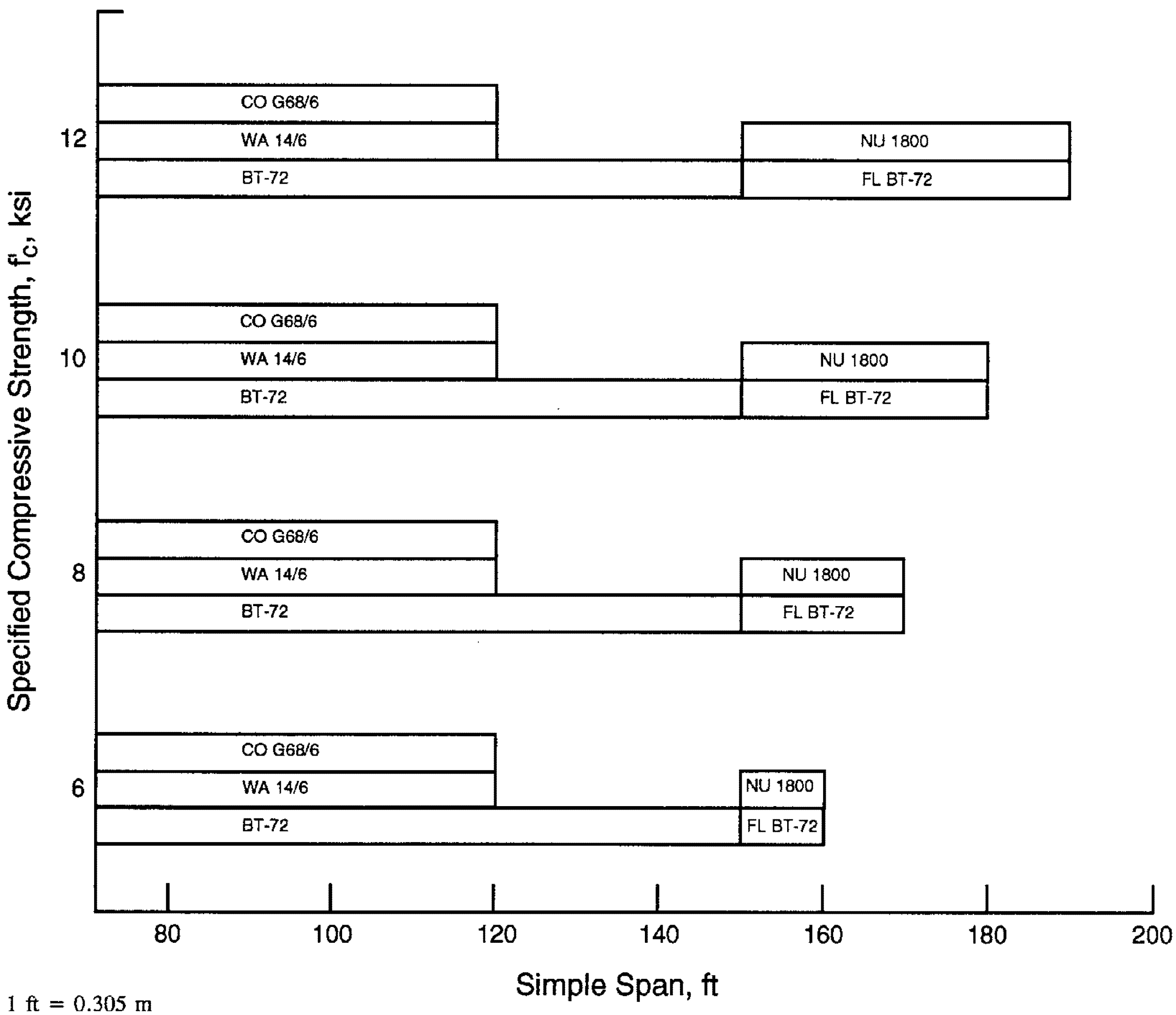
For existing cross sections designed using 0.5-in (12.7-mm) diameter Grade 270 strand at 2-in (51-mm) centers with 2-in (51-mm) cover, the BT-72 was the most cost-effective cross section for span lengths up to 150 ft (45.7 m) at all concrete compressive strengths. However, the WA 14/6 and CO G68/6 were equally cost-effective for span lengths up to about 120 ft (36.6 m). For span lengths greater than 150 ft (45.7 m) and all concrete compressive strengths, the FL BT-72 and NU 1800 were the most cost-effective. This is illustrated in figure 57.

For all existing sections designed using 0.5-in (12.7-mm) diameter Grade 270 strand at 2-in (51-mm) centers with 2 in (51 mm) of cover, the maximum useful concrete compressive strength was in the range of 9,000 to 10,000 psi (62 to 69 MPa). Above this strength level, sufficient prestressing force cannot be introduced into the cross section to take advantage of any higher concrete compressive strengths. For the Texas U54B cross section, the maximum useful concrete compressive strength was in the 12,000- to 14,000-psi (83- to 97-MPa) range.

Effect of Concrete Strength on Span Length

For all the cross sections analyzed, the use of a higher strength concrete enabled a given section to be designed for a longer span length. The increase in span length with compressive strength is greater when additional prestress force can be applied to the cross section. However, if additional prestressing force cannot be included, the beneficial effects are limited to the increase in allowable tensile stress at midspan. Since this increase is limited to the increase in the square root of the compressive strength, the incremental benefits decrease with each incremental increase in compressive strength.

A shallower section with a higher strength concrete can be more cost-effective than utilizing a deeper section with a lower strength concrete. Depending upon the premium for the higher strength concrete, the unit cost of the superstructure may be lower with the shallower section than with the deeper section. In addition, there will be other savings from the reduced substructure height. This concept is worthy of further study with regard to replacement of existing bridges.



1 ft = 0.305 m
1 ksi = 6.89 MPa

Figure 57. Optimum Cross Sections for Various Span Lengths and Concrete Strengths.

Effect of Strand Spacing and Size

Changes in strand spacing had little effect on the cost index for girders with a concrete compressive strength of 6,000 psi (41 MPa). However, when the concrete compressive strength was 12,000 psi (83 MPa), a decrease in strand spacing from 2 in to 1.5 in (51 to 38 mm) enabled the maximum span length to increase by about 22 ft (6.7 m) or 12 percent.

The use of 0.6-in (15.2-mm) diameter strands in place of 0.5-in (12.7-mm) diameter strands at a spacing and cover of 2 in (51 mm) showed no benefits for a concrete compressive strength of 6,000 psi (41 MPa). However, the effects became more beneficial at the higher concrete compressive strength levels. The use of 0.6-in (15.2-mm) diameter strand, instead of 0.5-in (12.7-mm) diameter strand, in a BT-72 with a concrete compressive strength of 12,000 psi (83 MPa) enabled the maximum span length to be increased by approximately 18 ft (5.5 m) or 10 percent.

The cost-effectiveness of using the Japanese strand with diameters of 17.8 and 21.8 mm was very dependent on the strand spacing that would be required with these strand diameters.

Effect of Strand Strength

The effect of increasing the strand strength from 270 ksi to 300 ksi (1860 to 2070 MPa) was dependent on concrete strength. At a concrete compressive strength of 6,000 psi (41 MPa), the benefits of the higher strength strand were small. At 12,000 psi (83 MPa), the span length of a BT-72 can be increased by about 10 ft (3.0 m) with the higher grade strand.

Effect of Modified Cross Sections

The effect of increasing the bottom flange thickness so that an additional row of prestressing strands can be added had little benefit when the girder concrete compressive strength was 6,000 psi (41 MPa) and was a small benefit when the concrete strength was 12,000 psi (83 MPa).

Effect of Post-Tensioning

The use of post-tensioning showed no benefits at a concrete compressive strength of 6,000 psi (41 MPa). At 12,000 psi (83 MPa), the span length of a BT-72 was increased by about 10 ft (3.0 m) with a small cost benefit.

Regional Differences

The cost analyses described in this report were based on average values for the costs of each material. However, it is recognized that fabrication and construction costs vary on a regional basis. In addition, construction practices vary throughout the country. Consequently, regional differences may influence the choice of optimum section.

RECOMMENDATIONS

In the near future, the industry should concentrate on the usage of concrete with specified compressive strengths up to 10,000 psi (69 MPa). For existing girder cross sections designed with 0.5-in (12.7-mm) diameter Grade 270 strands at 2-in (51-mm) centers and 2-in (51-mm) cover, the use of concrete with compressive strengths up to 10,000 psi (69 MPa) will allow longer span girders and, depending on the premium cost for the higher strength concrete, more economical structures. As a minimum, all highway departments should adopt 8,000-psi (55-MPa) compressive strength concrete as the normal design strength for longer span girders. It should be recognized that many precasters are already producing girders at this strength level.

To effectively utilize concretes with compressive strengths in excess of 10,000 psi (69 MPa), the industry must develop methods to apply additional prestressing force to the cross section. The following approaches in order of efficiency are recommended:

1. Use of 0.7-in (17.8-mm) diameter strand at 2-in (51-mm) centers.
2. Use of 0.5-in (12.7-mm) diameter strand at 1.5-in (38-mm) centers.
3. Use of 0.6-in (15.2-mm) diameter strand at 2-in (51-mm) centers.
4. Use of Grade 300 strand in place of Grade 270 strand with existing allowable strand spacings.
5. Increase bottom flange thickness by 2 in (51 mm) to allow an additional layer of strands.

However, additional research on the effect of strand spacing on transfer and development lengths is needed before these approaches can be implemented.

The PCI Bulb-Tee should continue to be considered as a national standard for span lengths from 80 to 200 ft (24.4 to 61.0 m). However, the WA 14/6 and CO G68/6 are very equivalent at span lengths up to 120 ft (36.6 m). For span lengths greater than 150 ft (45.7 m), the FL BT-72 and NU 1800 are slightly more economical.

This study has shown that girder spacing wider than 14 ft (4.3 m) may not be economical as the advantage of fewer girders is offset by the additional cost of the deck spanning a larger distance.

FUTURE RESEARCH

The analyses described in this report have ignored several aspects of structural design that need to be considered before higher strength concretes can be successfully utilized. In the process of the analyses, the following research needs have been identified.

Transfer and Development Lengths

The most important research need is a systematic investigation of transfer and development length. The major parameters of the investigation are strand size, strand spacing, strand strength and concrete strength.

Deflection, Lateral Stability, and Dynamic Characteristics

Longer span girders are going to have more short-term and long-term deflections than shorter girders. The effect of deflections on construction and serviceability needs to be evaluated. Calculated short-term deflections are dependent on the modulus of elasticity of the concrete. The appropriateness of existing equations for the modulus of elasticity should be determined. With longer spans and wider girder spacings, the potential for lateral instability before, during, and after construction increases and needs to be investigated. Longer span structures will also have different dynamic characteristics than shorter span structures. The effect on deck performance and impact factors should be examined.

Prestress Losses

The analyses described in this report were based on constant prestress losses. High-strength concretes undergo less creep per unit stress, are subjected to a higher unit stress, have a higher modulus of elasticity, and may have different amounts of shrinkage compared to lower strength concretes. The overall effect of creep, shrinkage and elastic shortening on total prestress losses for different cross sections needs to be evaluated.

Post-Tensioning

The concept of post-tensioning high-strength concrete girders needs to be further investigated. In particular, the concepts of post-tensioning in stages or a combination of pretensioning and post-tensioning should be studied.

Shear Strength

Selected designs included in this study were checked for shear using existing design equations. Research is needed to determine if existing design methods for shear in prestressed girders are applicable for concrete compressive strengths in excess of 10,000 psi (69 MPa).

Design Strength Age

For high-strength concretes, the building industry has specified design strengths at 56 and 90 days, rather than the traditional age of 28 days. The appropriateness of 28 days as the design strength age for prestressed concrete should be evaluated.

Long Span Decks

This investigation has confirmed previous research that the most economical structure is achieved with the least number of girders.[1,6] However, with wider girder spacings, a reinforced concrete deck may not be the most economical. Research into alternative deck systems for use with high-strength concrete prestressed girders is recommended.

APPENDIX A - GIRDER PROPERTIES

Appendix A contains cross-sectional properties of the standard bridge girders analyzed in this report. Standard sectional property notations have been used and 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_b = modulus of section for bottom concrete fiber

Efficiency factors ρ and α discussed in Chapter 2 have also been included. Table 2 contains a list of the property data.

Table 2. Girder Properties.

Agency	Girder Type	Depth (in)	Web Width (in)	Area (in^2)	Inertia (in^4)	y_t (in)	y_b (in)	S_t (in^3)	S_b (in^3)	ρ	α
CTL	BT-48	48	6	557	177,736	23.53	24.47	7,553	7,264	0.554	0.940
	BT-60	60	6	629	308,722	29.59	30.41	10,432	10,154	0.545	0.931
	BT-72	72	6	701	484,993	35.64	36.36	13,606	13,340	0.534	0.914
PCI	BT-54	54	6	659	268,077	26.37	27.63	10,166	9,702	0.558	0.943
	BT-63	63	6	713	392,638	30.82	32.12	12,715	12,224	0.556	0.942
	BT-72	72	6	767	545,894	35.40	36.60	15,421	14,915	0.549	0.934
AASHTO	Type VI	72	8	1,085	733,320	35.62	36.38	20,587	20,157	0.522	0.893
	Mod. Type VI	72	6	941	671,088	35.56	36.44	18,871	18,417	0.550	0.941
Washington	80/6	50	6	513	159,191	27.24	22.76	5,844	6,994	0.501	0.943
	100/6	58	6	591	256,560	30.01	27.99	8,549	9,166	0.517	0.925
	120/6	73.5	6	688	475,502	37.68	35.82	12,619	13,275	0.512	0.908
	14/6	73.5	6	736	534,037	35.30	38.20	15,122	13,985	0.538	0.894
Colorado	G54/6	54	6	631	242,592	27.33	26.67	8,877	9,095	0.527	0.924
	G68/6	68	6	701	426,575	33.99	34.01	12,548	12,544	0.526	0.911
Nebraska	1600	63	5.9	852	494,829	32.64	30.36	15,159	16,300	0.586	1.051
	1800	70.9	5.9	898	659,505	36.72	34.18	17,959	19,297	0.585	1.049
	2000	78.7	5.9	944	849,565	40.74	37.96	20,854	22,380	0.582	1.042
	2400	94.5	5.9	1,038	1,323,985	48.84	45.66	27,106	28,999	0.572	1.023
Florida	BT-54	54	6.5	785	311,765	28.11	25.89	11,091	12,042	0.546	0.983
	BT-63	63	6.5	843	458,521	32.88	30.12	13,945	15,223	0.549	0.992
	BT-72	72	6.5	901	638,672	37.64	34.36	16,968	18,588	0.548	0.991
Texas	U54A	54	10.2	1,022	379,857	30.12	23.90	12,612	15,895	0.516	0.996
	U54B	54	10.2	1,118	403,878	31.54	22.48	12,807	17,966	0.509	1.029

1 in = 25.4 mm

1 in^2 = 645 mm^2

1 in^3 = 16,390 mm^3

1 in^4 = 416,000 mm^4

APPENDIX B - COST DATA

INTRODUCTION

Many factors affect the costs of a particular bridge. These factors include span length, number of spans, accessibility to the construction site, distance between girder producer and site, distance between ready-mix concrete supplier and site, availability of experienced labor, and unit material costs. In the previous study, it was found that combining the above factors presented difficulties in assessing cost factors in different regions of the United States.[1,2] In addition, each highway agency used a different approach to itemize unit costs. From survey data, average costs for girder concrete, deck concrete, reinforcing steel, and prestressing strands were determined. The average cost included the cost of materials and labor. For girder concrete, the cost also included a factor for transportation and erection. These average costs were then reduced to relative costs per pound of in-place materials. The following relative unit costs for in-place materials (including labor) were used for the cost analysis.

Concrete (girders and deck)	-	1 unit/unit weight of concrete
Strands	-	8 units/unit weight of strands
Reinforcing Steel	-	9 units/unit weight of reinforcing
Epoxy-Coated Reinforcing Steel	-	12 units/unit weight of epoxy-coated reinforcing

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 the computer program BRIDGE. Top deck reinforcement can also be specified as uncoated bars.

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

In the current analysis, the relative unit costs were assumed to be the same as in the previous report.[1,2] However, it was recognized that the relative unit costs may have changed since the previous survey was conducted. It was also recognized that the relative cost index per square foot may be sensitive to the assumed relative costs of the individual materials. Consequently, a limited survey was conducted to obtain additional cost information.

SURVEY DATA

Information on the cost of concrete, strand, uncoated reinforcement, and epoxy-coated reinforcement was obtained from four precast concrete producers at widely different locations in the United States. The producers were also asked to supply any information relative to the premium costs that they would charge for higher strength concrete. Results of this survey expressed as relative costs are summarized in table 3. Data published by Zia and Geren are also included in the table as relative costs.[7,4] As can be seen, there is a wide range in the relative costs of the different materials. This range is partly due to the different ways in which estimators allocate the costs for labor, overhead, miscellaneous accessories per girder and shipping costs. Based on this data, it was determined that the range of the indicators of relative material costs were as follows:

Material	Measure	Minimum Ratio	Intermediate Ratio	Maximum Ratio
Concrete (girder)	unit/unit weight of girder concrete	1	1	1
Concrete (deck)	unit/unit weight of deck concrete	1	1	1
Strands	unit/unit weight of strand	8	15	30
Uncoated Reinforcement	unit/unit weight of reinforcement	9	25	25
Epoxy-Coated Reinforcement	unit/unit weight of epoxy-coated reinforcement	12	30	50

To determine the sensitivity of the cost index per square foot to the assumed relative unit costs, analyses were made using the relative unit costs listed above.

Data from the precasters indicated a range of premium costs that would be charged for higher strength concrete. To obtain a better measure of the relative costs of materials for use in high-strength concrete, a separate investigation was conducted. The mixed proportions of high-strength concretes available in the Chicago area have been published elsewhere.[14-16] These proportions were utilized to develop the cost of the concrete materials per cubic yard. The unit prices for the constituent materials used in the mixes were obtained from information published in *ENR* and by contacting admixture suppliers to determine the approximate cost of their materials. The costs per cubic yard of concrete obtained by this approach are shown in figure 58. It should be noted that these figures are for material costs alone and do not reflect the in-place cost of the concrete. On the assumption that the labor to deliver and place the concretes will be only slightly dependent on concrete strength, the relative premium costs of the higher strength concrete will be lower than those shown in figure 58. In-place costs for high-strength concrete have been published by ACI Committee 363 and these data are shown below.[11]

Table 3. Relative Cost of In-Place Materials.

Source	Strength, psi	Concrete*	Strand*	Uncoated Bars *	Epoxy-Coated Bars*
A	<10,000	1	15	27*	27**
A	<14,000	1.47	15	27*	27**
B	6,000	1	28	24	53
C	5,000	1.00	25	-	24
C	7,000	1.19	25	-	24
C	9,500	1.30	25	-	24
C	12,000	1.53	25	-	24
D	5,000	1.00	26	22**	22**
D	6,000	1.03	26	22**	22**
D	7,000	1.09	26	22**	22**
D	10,000	1.69	26	22**	22**
Zia(7)	6,000	1	30	19	-
Geren(4)	6,000	1	14	7**	7**

* The costs for each source are relative to a value of 1 for the concrete from the same source.

** Average cost of reinforcement.

1000 psi = 6.89 MPa

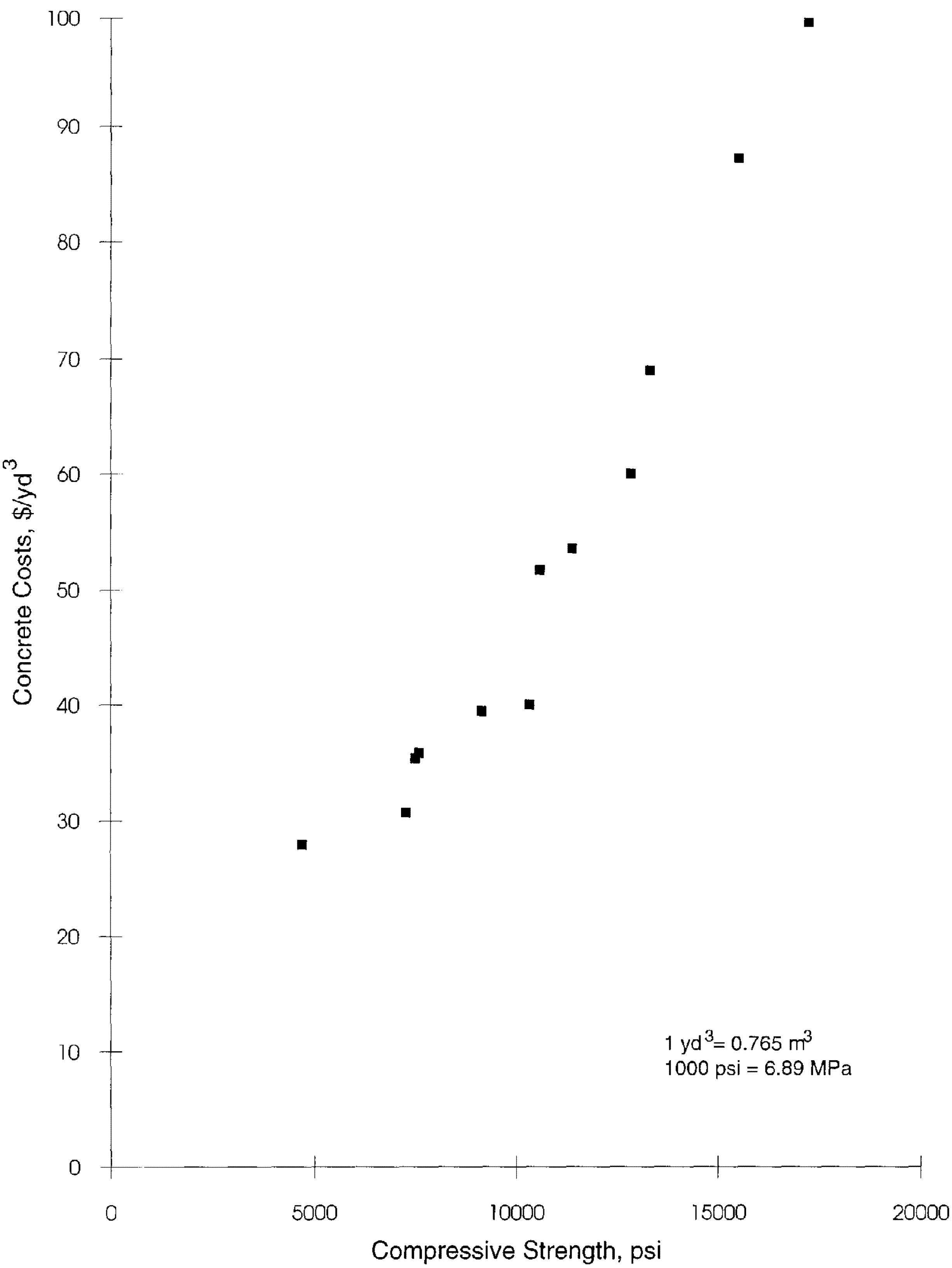


Figure 58. Premium Concrete Costs versus Compressive Strength.

Strength	\$/yd^3	\$/m^3	Relative Cost
7,000 psi (48 MPa)	80	105	1.00
9,000 psi (62 MPa)	85	111	1.06
11,000 psi (76 MPa)	104	136	1.30
14,000 psi (97 MPa)	129	169	1.61

In-place unit costs for material have also been published by Zia and they are as follows:(7)

Strength	\$/yd^3	\$/m^3	Relative Cost
6,000 psi (41 MPa)	75	98	1.00
8,000 psi (55 MPa)	82	107	1.09
10,000 psi (69 MPa)	95	124	1.27
12,000 psi (83 MPa)	112	146	1.49

Based on these data, it was decided to perform some analyses using the following ratios for premium costs of high-strength concrete to determine the sensitivity of the cost index per square foot to premium costs of higher strength concrete.

Strength	Minimum Ratio	Intermediate Ratio	Maximum Ratio
6,000 psi (41 MPa)	1.00	1.00	1.00
8,000 psi (55 MPa)	1.00	1.05	1.10
10,000 psi (69 MPa)	1.00	1.13	1.25
12,000 psi (83 MPa)	1.00	1.25	1.50

Results of the comparisons are reported in the section entitled **RELATIVE UNIT COSTS.**

APPENDIX C - THE COMPUTER PROGRAM "BRIDGE"

The computer program BRIDGE is a precast, prestressed bridge girder design program developed as part of a previous FHWA investigation. A complete description of the original program is contained in Appendix E of the final report for that study, *Optimized Sections for Major Prestressed Concrete Bridge Girders*.(1)

As part of this current FHWA investigation, the BRIDGE program was rewritten in BASIC for an IBM PC, modified to incorporate the current AASHTO Specifications and expanded in application.(10) Details of the new version are explained in this appendix. For completeness, much of the original documentation has been repeated below and modified accordingly for this new version. Program documentation, source listing, and sample problems follow.

PROGRAM DOCUMENTATION

Overview

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

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

Design procedures for the girder and deck are based on the 1992 AASHTO Specifications.(10) All computations are made for an interior girder with HS 20-44 loading.

The program is divided into eight solution steps, which consist of the following:

1. Geometric and material properties, and relative unit costs of materials are input.
2. Allowable material properties are computed.
3. Deck thickness and reinforcement are determined.
4. Non-composite and composite sectional properties are calculated.

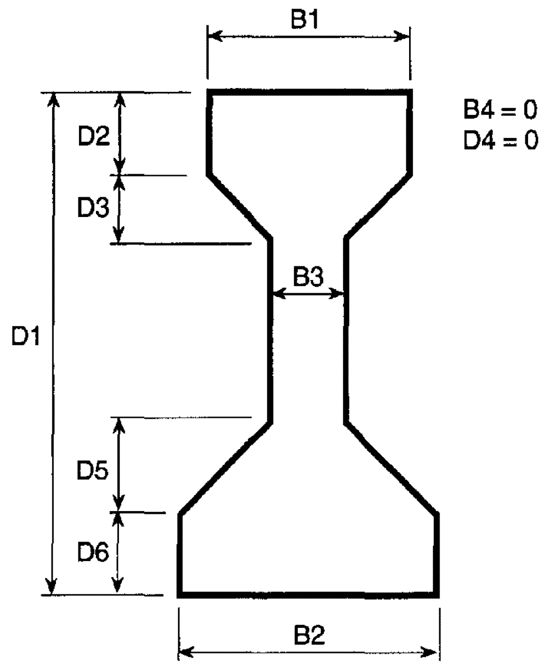
5. Design loads and moments are computed.
6. Number of strands required to satisfy service and strength conditions are determined.
7. Midspan deflections are computed.
8. Total and unit material quantities and cost index per unit surface area of bridge are calculated.

Each solution step is described below, including details of data input and output, design assumptions, capabilities, and limitations.

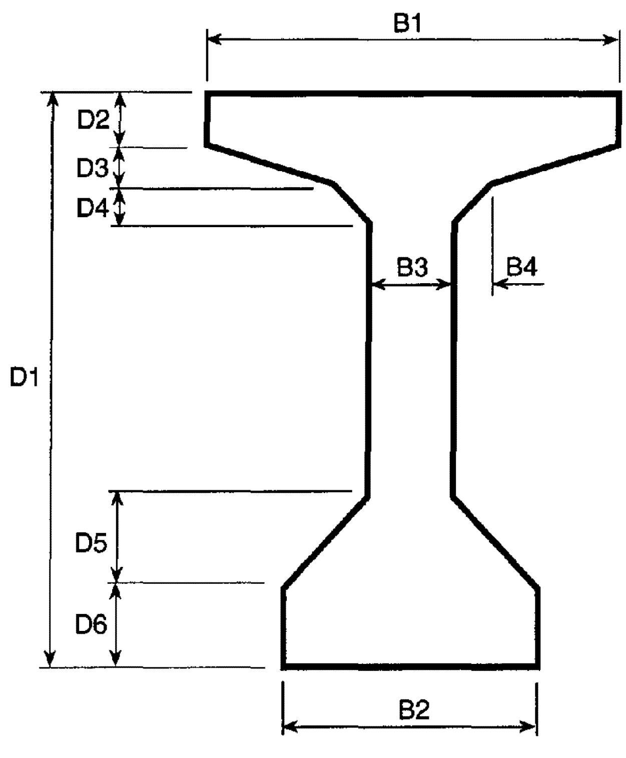
Solution Step 1 - Input Data

Input for the program consists of 38 variables that uniquely define a particular girder condition. Input is accomplished through DATA statements within Step 1. The required input variables are:

SL	=	Span length, center to center, of supports for simply supported girders, in feet. The program will accommodate spans ranging from 70 to 300 ft (21 to 92 m).
GS	=	Girder spacing, center to center, in feet. Maximum girder spacing within the program is such that the effective deck span cannot exceed 16 ft (4.9 m).
B1 - B4	=	Horizontal dimensions that define the girder cross section; refer to figure 59; in inches.
D1 - D6	=	Vertical dimensions that define the girder cross section; refer to figure 59; in inches.
CTC	=	Center-to-center strand spacing at midspan, in inches. The program assumes the same spacing in the vertical and horizontal directions. For bundled strands, CTC should be set equal to the nominal strand diameter.
CSC	=	Concrete surface to center of strand distance at midspan, in inches. This dimension reflects the amount of concrete cover in the bottom flange at midspan.



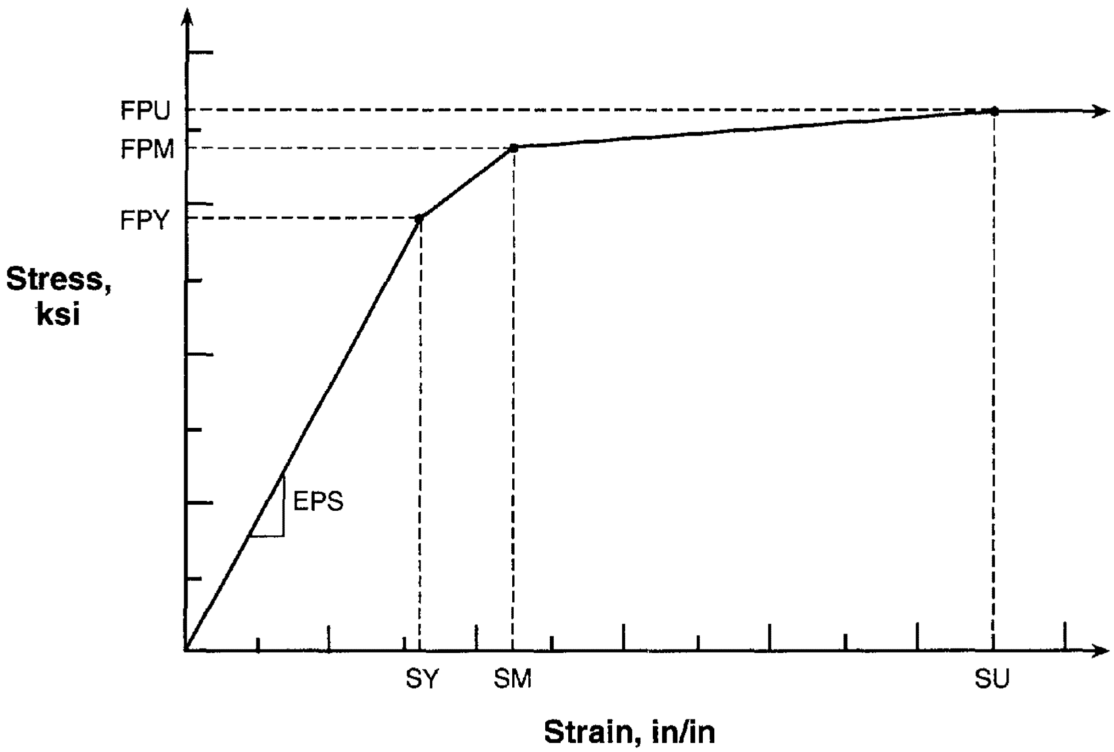
a) I-Shaped Section.



b) T-Shaped Section.

Figure 59. Dimensions Defining Girder Cross Section.

SWW	=	Number of strands in one row, within the web width at midspan. This variable is required only when the bottom flange is completely filled with prestressing strands and additional strands are required to satisfy stress or strength requirements.
FCP	=	Girder concrete compressive strength at 28 days, in psi.
FCPI	=	Girder concrete compressive strength at prestress transfer, in psi.
WC	=	Unit weight of girder concrete, in pcf.
WCD	=	Unit weight of deck concrete, in pcf.
WG	=	Unit weight of girder, in pcf, accounting for weight of concrete, reinforcing steel, and strands.
WD	=	Unit weight of deck, in pcf, accounting for weight of concrete and reinforcing steel.
FCD	=	Deck concrete compressive strength at 28 days, in psi.
FPY, FPU	=	Prestressing strand yield stress and ultimate strength, respectively, in psi, as shown in figure 60.
FPM	=	Stress level, in psi, which is positioned between FPY and FPU to form the trilinear stress-strain relationship of the prestressing strand, as shown in figure 60.
SY, SM, US	=	Strains corresponding to FPY, FPM, and FPU, respectively, as shown in figure 60.
ESP	=	Modulus of elasticity of prestressing strand, in psi, as shown in figure 60.
XLS	=	Total prestress losses, in psi.
ASTD	=	Nominal area of one prestressing strand, in square inches.
UST	=	Nominal prestressing strand weight, in lb/ft.
SRF	=	Stiffness reduction factor used to compute the girder deflections. This variable accounts for concrete creep and prestress losses.



1 in = 25.4 mm
1 ksi = 6.89 MPa

Figure 60. Trilinear Stress-Strain Characteristics of Strands.

RUCG, RUCD, RUS, RUR, RUE

= Relative unit costs for girder concrete, deck concrete, prestressing strand, deck bottom reinforcing, and deck top epoxy-coated reinforcing, respectively. If top deck reinforcing is not epoxy-coated, set RUE = RUR.

Solution Step 2 - Material Properties and Allowable Stresses

Various material properties and allowable stresses are calculated based on the input data from Step 1. Material properties include modulus of rupture for the girder concrete and moduli of elasticity for the girder and deck concretes. Allowable stresses include concrete stresses at transfer and service load conditions and strand effective prestress after losses. A check is made to verify that the effective prestress after losses is within the allowable AASHTO Specification range.[10] If the strand stress is not within the allowable range, the program is terminated at this point.

Solution Step 3 - Deck Thickness and Reinforcement

Determination of the cast-in-place slab (deck) thickness and reinforcement have been adopted from design aids prepared by the Washington State Department of Transportation.[17] The design aids were based on AASHTO and ACI guidelines. For the current study, the guidelines were checked and updated for compliance with the current AASHTO Specifications and ACI recommendations.[10,11] In addition, the design was extended from an effective deck span of 10 ft (3.05 m) to an effective deck span of 16 ft (4.9 m). If the effective deck span exceeds 16 ft (4.9 m), the program is terminated at this point. The deck design was 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.

Results of the design are shown in table 4. This data was subsequently stored within the BRIDGE program.

The effective deck span has also been updated to reflect the current AASHTO Specifications.[10] If the width/thickness ratio of the girder top flange is less than a value of

Table 4. Deck Design.*

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
10 to 11 inclusive	8-3/4	No. 6	7.5
11 to 12 inclusive	9	No. 7	10.0
12 to 13 inclusive	9-1/2	No. 7	10.0
13 to 14 inclusive	9-3/4	No. 7	9.5
14 to 15 inclusive	10	No. 7	9.5
15 to 16 inclusive	10-1/2	No. 7	9.5

1 in = 25.4 mm
1 ft = 0.305 m

NOTES:

* The table is based on Reference No. 17.

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

four, the effective deck span equals the clear span as shown in figure 61a. However, if the width/thickness ratio of the girder top flange equals or exceeds a value of four, the effective deck span equals the clear span plus one-half of the girder top flange as shown in figure 61b.

Solution Step 4 - Girder Section Properties

Section properties determined for the noncomposite (girder) and composite (girder/deck) member consist of the following:

1. Location of center of gravity.
2. Cross-sectional area.
3. Moment of inertia.
4. Section modulus for the top and bottom fibers.

For the composite section, the effective top flange width is calculated as the smallest of:

1. Girder span divided by four.
2. Girder spacing.
3. Twelve times the deck thickness plus the web width.

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

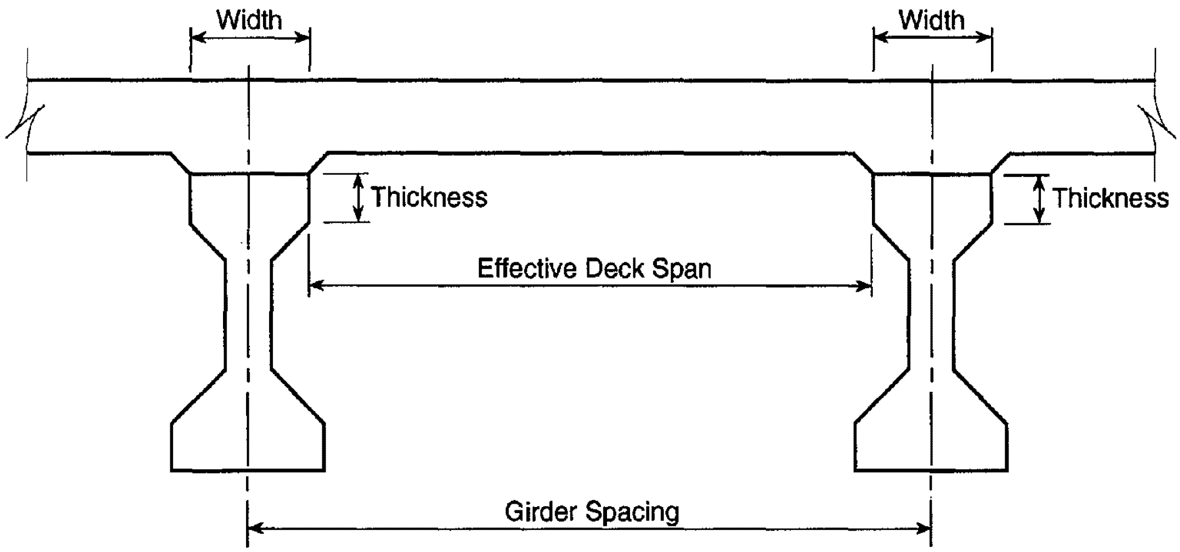
Solution Step 5 - Design Loads and Moments

Dead loads are based on the cross-sectional area of the girder and deck calculated in the previous step and unit weights specified in Step 1. Live load considered in the BRIDGE program is HS 20-44, with an impact factor based on the equation 50/(span length in feet + 125), but not to exceed 30 percent.

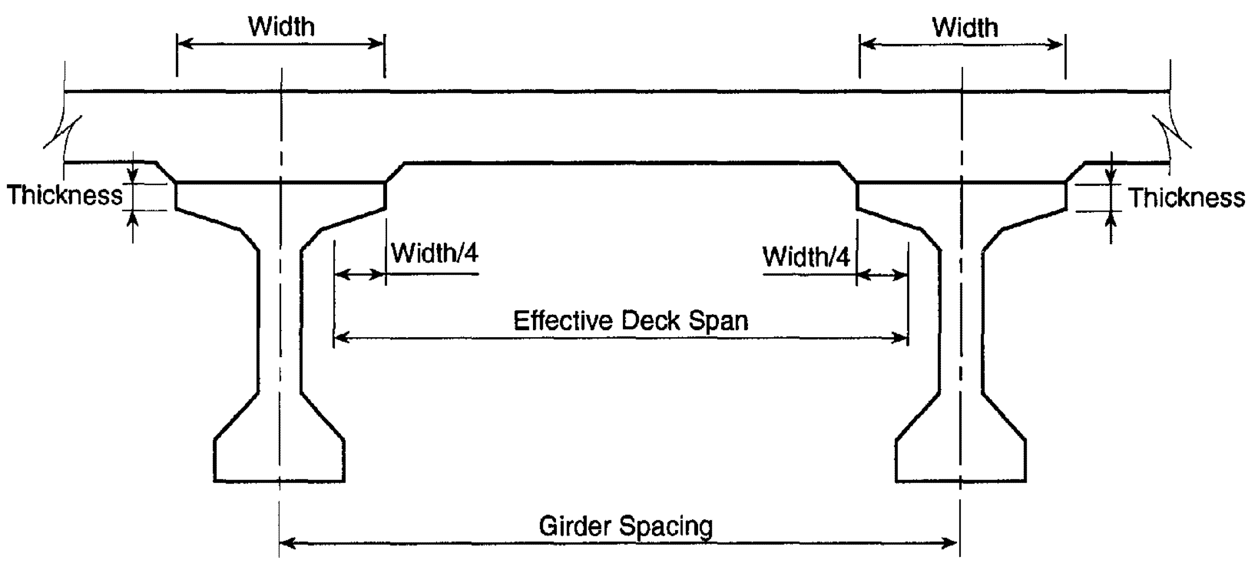
Midspan dead load and live load moments are computed for simple spans. Live load moments have been adopted from Appendix A of the AASHTO Specifications.(10) They are summarized in table 5. For intermediate spans, moments are computed through linear interpolation. These live load moments are for a 10-ft (3.0-m) lane width and must be proportioned to the actual girder spacing. The ultimate strength required is based on the equation 1.3 [Dead Load + 1.67 (Live Load + Impact)].

Solution 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



a) Width/Thickness Ratio < 4.



b) Width/Thickness Ratio ≥ 4.

Figure 61. Effective Deck Spans.

Table 5. HS 20-44 Bending Moments.[10]

Span (ft)	Bending Moment (ft-kip)
70	985.6
80	1164.9
90	1344.4
100	1524.0
110	1703.6
120	1883.3
130	2063.1
140	2242.8
150	2475.1
160	2768.0
170	3077.1
180	3402.1
190	3743.1
200	4100.0
220	4862.0
240	5688.0
260	6578.0
280	7532.0
300	8550.0

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

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 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 62. 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.

During the incremental analysis to determine the number of strands, initial and effective prestress levels are computed. 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.[18] The required flexural strength (M_u) is checked against the nominal flexural strength (M_n) times the strength reduction factor of 0.9. Minimum steel requirements of the AASHTO Specifications are also checked.[10] If these requirements are not satisfied, the number of strands is increased by one. The BRIDGE program limits strand placement to below the top core or kern of the section.

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

Solution Step 7 - Deflection at Midspan

The effect of prestress alone 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; due to

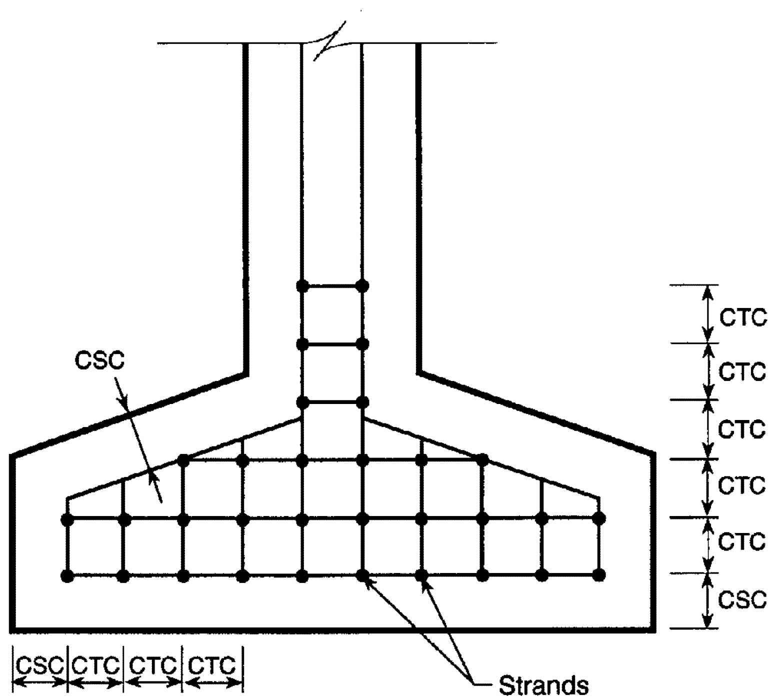


Figure 62. Prestressing Strand Layout.

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.[19] The stiffness reduction factor (SRF) is equal to 1/1.82 or 0.55.

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 the BRIDGE program, 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.[20]

Solution Step 8 - Cost Index

Cost index per unit surface area of bridge computed in the BRIDGE program 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 from Step 1.

SOURCE LISTING

The following pages contain a source listing of the BASIC version of the BRIDGE program for the IBM PC.

```
100 '
110 '
120 '    ***********************************************************
130 '    *                                                         *
140 '    *                   Program "BRIDGE"                      *
150 '    *      Structural Design of Prestressed Concrete Bridge Girders   *
160 '    *                                                         *
170 '    *      This program calculates the number of strands and the    *
180 '    *       cost index per unit surface area for a prestressed       *
190 '    *       concrete bridge girder.                            *
200 '    *                                                         *
210 '    ***********************************************************
220 '
230 '
240 KEY OFF
250 ' Set up array variables
260 DIM DECKDSG(15,4),X(3),A(7),Y(7),XJ(7),SLL(19),DM(19),NR(50)
270 ' Input values for deck design & HS20-44 loading
280 '  Deck design:
290 '   DECKDSG(I,1) = deck thickness for given effective slab span (Ith span)
300 '   DECKDSG(I,2) = required reinforcement size, i.e. 5 = No. 5 bar
310 '   DECKDSG(I,3) = reinforcement weight corresponding to DECKDSG(I,2)
320 '   DECKDSG(I,4) = required spacing of reinforcement
330 FOR I=1 TO 15
340  FOR J=1 TO 4
350   READ DECKDSG(I,J)
360  NEXT J
370 NEXT I
380  DATA  7,5,1.043,10,7,5,1.043,10,7,5,1.043,8.5,7,5,1.043,7.5,7,5,1.043,6.5
390  DATA  7,6,1.502,8,7.5,6,1.502,8,8,6,1.502,8,8.5,6,1.502,8,8.75,6,1.502,7.5
400  DATA  9,7,2.044,10,9.5,7,2.044,10,9.75,7,2.044,9.5,10,7,2.044,9.5
410 DATA  10.5,7,2.044,9.5
420 '  HS20-44 loading:
430 '   SLL = span length for particular HS20-44 live load moment
440 '   DM = HS20-44 live load moment per 10 ft lane width corresponding to SLL
450 FOR I=1 TO 19
460  READ SLL(I),DM(I)
470 NEXT I
480 DATA  70,985.6,80,1164.9,90,1344.4,100,1524.0,110,1703.6,120,1883.3
490 DATA  130,2063.1,140,2242.8,150,2475.1,160,2768.0,170,3077.1
500 DATA  180,3402.1,190,3743.1,200,4100.0,220,4862.0,240,5688.0
510 DATA 260,6578.0,280,7532.0,300,8550.0
520 '
530 ' ******************* STEP 1 - INPUT DATA *******************
540 '
550 ' Geometric Properties:
560 '  SL = girder span length
570 '  GS = girder spacing
580 '  B1,B2,B3,B4 = horizontal dimensions of girder cross section (in.)
590 '  D1,D2,D3,D4,D5,D6 = vertical dimensions of girder cross section (in.)
600 '  CTC = center to center distance of strands (in.)
610 '  CSC = concrete surface to center of strands (in.)
620 '  SWW = maximum number of strands within web width
630 '
```

```
640 ' Material Properties:
650 '  Concrete:
660 '   FCP = specified girder concrete strength (psi)
670 '   FCPI = concrete strength at prestress transfer (psi)
680 '   WC = unit weight of girder concrete (pcf)  "145"
690 '   WCD = unit weight of deck concrete (pcf)  "145"
700 '   WG = unit weight of girder (pcf)  "150"
710 '   WD = unit weight of deck (pcf)  "150"
720 '   FCD = specified concrete strength for deck (psi)
730 '  Strand:
740 '   FPY = specified yield stress of strand (psi)
750 '   FPM = specified strand stress level which is positioned between
760 '         FPY and FPU to form trilinear stress-strain curve (psi)
770 '   FPU = specified ultimate strength of strand (psi)
780 '   EPS = modulus of elasticity of strand (psi)
790 '   SY = strain corresponding to FPY
800 '   SM = strain corresponding to FPM
810 '   SU = strain corresponding to FPU
820 '   XLS = total prestress losses (psi)
830 '   ASTD = nominal area of strand (in^2)
840 '   UST = nominal weight of strand (lb/ft)
850 '
860 '   SRF = stiffness reduction factor due to creep and prestress
870 '          losses  "0.55"
880 '
890 ' Relative Unit Costs:
900 '  RUCG = relative unit cost of girder concrete (unit/lb)
910 '  RUCD = relative unit cost of deck concrete (unit/lb)
920 '  RUS = relative unit cost of strand (unit/lb)
930 '  RUR = relative unit cost of deck reinforcement (unit/lb)
940 '  RUE = relative unit cost of epoxy coated deck reinforcement;
950 '       if no epoxy coated bar is used, a value equal to RUR is to be
960 '       given for RUE
970 '
980 ' ********** PROGRAM INPUT STATEMENTS **********
990 '
1000 TITLE$=" FHWA SAMPLE PROBLEM NO. 1, AASHTO TYPE VI"
1010 OUTPUT=1
1020 READ SL,GS,B1,B2,B3,B4,D1,D2,D3,D4,D5,D6,CTC,CSC,SWW,FCP,FCPI,
WC,WCD,WG,WD
1030 READ FCD,FPY,FPM,FPU,EPS,SY,SM,SU,XLS
1040 READ ASTD,UST,SRF,RUCG,RUCD,RUS,RUR,RUE
1050 DATA 140,6,42,28,8,4,72,5,3,4,10,8,2,2,2,10000,7500,155,145,160,150
1060 DATA 4000,230000,255000,270000,28000000,0.008214,0.012,0.040,45000
1070 DATA 0.153,0.530,0.55,1,1,8,9,12
1080 '
1090 ' ********** PROGRAM INPUT STATEMENTS **********
1100 '
1110 ' ********************** END OF STEP 1 **********************
1120 '
1130 ' *************** STEP 2 - MATERIAL PROPERTIES ***************
1140 '
1150 ' Input:
1160 '  From Step 1:
```

```
1170 '   FCP,FCPI,FCD,WC,WCD,WG,WD,FPU,FPY,EPS,XLS
1180 ' Output:
1190 '  Concrete:
1200 '   FCCI = allowable compressive stress of girder at prestress
1210 '         transfer (psi)
1220 '   FCTI = allowable tensile stress of girder at prestress transfer (psi)
1230 '   FCC = allowable compressive stress of girder at service load (psi)
1240 '   FCT = allowable tensile stress of girder at service load (psi)
1250 '   FCR = modulus of rupture of girder concrete (psi)
1260 '   ECI = modulus of elasticity of girder at prestress transfer (psi)
1270 '   EC = modulus of elasticity of girder (psi)
1280 '   ECD = modulus of elasticity of deck concrete (psi)
1290 '   XNE = modulur ratio of deck concrete to girder concrete
1300 '  Strand:
1310 '   FPS = allowable stress at service load (psi)
1320 '   FSE = effective prestress at service load (psi)
1330 '
1340 ' Concrete properties
1350 FCCI=(.6)*FCPI
1360 FCTI=0
1370 FCC=(.4)*FCP
1380 FCT=6*SQR(FCP)
1390 FCR=7.5*SQR(FCP)
1400 ECI=33*(WC^1.5)*SQR(FCPI)
1410 EC=33*(WC^1.5)*SQR(FCP)
1420 ECD=33*(WCD^1.5)*SQR(FCD)
1430 XNE=ECD/EC
1440 ' Strand properties
1450 FPS=(.8)*FPY
1460 FSE=(.7)*FPU-XLS
1470 ' Check that effective prestress is within AASHTO specified range
1480 IF (.5*FPU) > FSE THEN GOTO 1490 ELSE GOTO 1520
1490  PRINT "EFFECTIVE PRESTRESS IS OUT OF AASHTO ALLOWABLE
RANGE"
1500  PRINT "PROGRAM TERMINATED"
1510  END
1520 IF FSE > FPS THEN GOTO 1530 ELSE GOTO 1560
1530  PRINT "EFFECTIVE PRESTRESS IS OUT OF AASHTO ALLOWABLE
RANGE"
1540  PRINT "PROGRAM TERMINATED"
1550  END
1560 '
1570 ' *********************** END OF STEP 2 ***********************
1580 '
1590 ' *********** STEP 3 - THICKNESS AND REINF. OF DECK ***********
1600 '
1610 ' Input:
1620 '  From Step 1:
1630 '   GS,B1
1640 ' Output:
1650 '  SE = effective span of deck (ft)
1660 '  TD = thickness of deck (in)
1670 '  DREINFNO = deck reinforcement bar number, i.e. 5 = #5 bar
1680 '  DREINFWT = deck reinforcement weight corresponding to DREINFNO (lb/ft)
```

```
1690 '  DREINFSP = deck reinforcement spacing (in)
1700 '
1710 ' Calculate eff. deck span in accordance with AASHTO 15th edition
1720 CLEARSPN=GS-(B1/12)
1730 IF (B1/D2) < 4 THEN SE=CLEARSPN ELSE SE=CLEARSPN+(B1/2)/12
1740 I=0
1750 FOR J=1 TO 15
1760  IF SE > J AND SE <= J+1 THEN I=J ELSE GOTO 1770
1770 NEXT J
1780 ' Test for effective slab span exceeding table limits
1790 IF I=0 THEN GOTO 1800 ELSE GOTO 1820
1800  PRINT "EFFECTIVE SLAB SPAN EXCEEDS 16 ft, PROGRAM TERMINATED"
1810  END
1820 TD=DECKDSG(I,1)
1830 DREINFNO=DECKDSG(I,2)
1840 DREINFWT=DECKDSG(I,3)
1850 DREINFSP=DECKDSG(I,4)
1860 '
1870 ' *********************** END OF STEP 3 ***********************
1880 '
1890 ' ************* STEP 4 - GIRDER SECTION PROPERTIES ************
1900 '
1910 ' Input:
1920 '  From Step 1:
1930 '    SL,GS,B1,B2,B3,B4,D1,D2,D3,D4,D5,D6
1940 '  From Step 2:
1950 '    XNE
1960 '  From Step 3:
1970 '    TD
1980 ' Output:
1990 '  AG = area of noncomposite section (in^2)
2000 '  AC = area of composite section (in^2)
2010 '  BE = effective top flange (deck) width (in) - composite
2020 '  YT = distance from centroid to top fiber (in) - noncomposite
2030 '  YB = distance from centroid to bottom fiber (in) - noncomposite
2040 '  YTC = distance from centroid to top fiber (in) - composite
2050 '  YBC = distance from centroid to bottom fiber (in) - composite
2060 '  XIG = moment of inertia of girder section (in^4) - noncomposite
2070 '  XIGC = moment of inertia of girder section (in^4) - composite
2080 '  ST = section modulus for top fiber (in^3) - noncomposite
2090 '  SB = section modulus for bottom fiber (in^3) - noncomposite
2100 '  STC = section modulus for top fiber (in^3) - composite
2110 '  SBC = section modulus for bottom fiber (in^3) - composite
2120 '
2130 ' Calculate effective width of top flange
2140 X(1)=SL*12/4
2150 X(2)=GS*12
2160 X(3)=(TD*12)+B3
2170 BE=X(3)
2180 FOR I=1 TO 2
2190  IF BE > X(I) THEN BE=X(I) ELSE GOTO 2200
2200 NEXT I
2210 ' Noncomposite section:
2220 '  A(I) = area of Ith element of section
```

```
2230 A(1)=B1*D2
2240 A(2)=(2*B4+B3)*D3
2250 A(3)=(B1-2*B4-B3)*D3/2
2260 A(4)=2*B4*D4/2
2270 A(5)=B3*(D1-D2-D3-D6)
2280 A(6)=(B2-B3)*D5/2
2290 A(7)=B2*D6
2300 '  Y(I) = distance from bottom fiber to centroid of Ith element
2310 Y(1)=D1-D2/2
2320 Y(2)=D1-D2-D3/2
2330 Y(3)=D1-D2-D3/3
2340 Y(4)=D1-D2-D3-D4/3
2350 Y(5)=(D1-D2-D3-D6)/2+D6
2360 Y(6)=D5/3+D6
2370 Y(7)=D6/2
2380 '  XJ(I) = moment of inertia of Ith element
2390 XJ(1)=B1*D2^3/12
2400 XJ(2)=(2*B4+B3)*D3^3/12
2410 XJ(3)=(B1-2*B4-B3)*D3^3/12
2420 XJ(4)=2*B4*D4^3/36
2430 XJ(5)=B3*(D1-D2-D3-D6)^3/12
2440 XJ(6)=(B2-B3)*D5^3/36
2450 XJ(7)=B2*D6^3/12
2460 '
2470 AG=0
2480 FOR I=1 TO 7
2490  AG=AG+A(I)
2500 NEXT I
2510 TJ=0
2520 FOR I=1 TO 7
2530  TJ=TJ+XJ(I)
2540 NEXT I
2550 YBB=0
2560 FOR I=1 TO 7
2570  YBB=YBB+A(I)*Y(I)
2580 NEXT I
2590 YB=YBB/AG
2600 YT=D1-YB
2610 XIGG=0
2620 FOR I=1 TO 7
2630  XIGG=XIGG+A(I)*(Y(I)-YB)^2
2640 NEXT I
2650 XIG=TJ+XIGG
2660 ST=XIG/YT
2670 SB=XIG/YB
2680 '
2690 ' Composite section:
2700 A(1)=XNE*BE*TD
2710 A(2)=AG
2720 Y(1)=D1+TD/2
2730 Y(2)=YB
2740 XJ(1)=XNE*BE*TD^3/12
2750 XJ(2)=XIG
2760 AC=A(1)+A(2)
```

```
2770 YBC=(A(1)*Y(1)+A(2)*Y(2))/AC
2780 YTC=D1-YBC
2790 XIGC=XJ(1)+XJ(2)+A(1)*(Y(1)-YBC)^2+A(2)*(Y(2)-YBC)^2
2800 STC=XIGC/YTC
2810 SBC=XIGC/YBC
2820 '
2830 ' *********************** END OF STEP 4 ***********************
2840 '
2850 ' ************* STEP 5 - DESIGN LOADS AND MOMENTS *************
2860 '
2870 ' Input:
2880 '  From Step 1:
2890 '    SL,GS,WG,WD
2900 '  From Step 3:
2910 '    TD
2920 '  From Step 4:
2930 '    AG
2940 ' Output:
2950 '  XMD = moment due to deck plus girder weight (ft-kip)
2960 '  XMDG = moment due to girder weight (ft-kip)
2970 '  XML = moment due to live load (ft-kip)
2980 '  XMU = factored total moment (ft-kip)
2990 '  XIP = impact load coefficient
3000 '  WUG = uniformly distributed load due to girder weight (kip/ft)
3010 '  WUD = uniformly distributed load due to deck weight (kip/ft)
3020 '
3030 ' Calculate design loads and moments
3040 XIP=50/(SL+125)
3050 IF XIP > .3 THEN XIP=.3 ELSE GOTO 3060
3060 WUG=WG*AG/(12^2)/1000
3070 WUD=WD*GS*TD/12/1000
3080 XMDG=WUG*SL^2/8
3090 XMD=XMDG+WUD*SL^2/8
3100 ' Check if girder span is within 70 ft to 300 ft range
3110 IF SL < SLL(1) OR SL > SLL(19) THEN GOTO 3120 ELSE GOTO 3140
3120  PRINT "GIRDER SPAN IS OUTSIDE OF ALLOWABLE RANGE, PROGRAM TERMINATED"
3130  END
3140 FOR I=2 TO 19
3150  IF SL <= SLL(I) AND SL >= SLL(I-1) THEN GOTO 3160 ELSE GOTO 3170
3160   XXM=(SL-SLL(I-1))/(SLL(I)-SLL(I-1))*(DM(I)-DM(I-1))+DM(I-1)
3170 NEXT I
3180 XML=(GS/10)*XXM
3190 XMU=1.3*(XMD+(1.67)*(XML)*(1+XIP))
3200 '
3210 ' *********************** END OF STEP 5 ***********************
3220 '
3230 ' ************ STEP 6 - REQUIRED NUMBER OF STRANDS ************
3240 '
3250 ' Substep 6A - Allowable Stresses Check
3260 ' Input:
3270 '  From Step 1:
3280 '    FPU,B2,B3,D5,D6,ASTD,CTC,CSC,SWW
3290 '  From Step 2:
```

```
3300 '   FCCI,FCTI,FCC,FCT,FSE
3310 '  From Step 4:
3320 '   AG,ST,SB,STC,SBC,YB
3330 '  From Step 5:
3340 '   XMD,XMDG,XML,XIP
3350 ' Output:
3360 '  NS = No. of strands required
3370 '  ET = distance from centroid of strands to centroid of girder
3380 '      section (in) - noncomposite
3390 '  AS = total area of strands required (in^2)
3400 '  FI = total initial prestressing force (kip)
3410 '  FS = total prestressing force at service load (kip)
3420 '  CE = distance from centroid of strands to bottom fiber (in)
3430 '  NR(I) = maximum no. of strands placed in Ith row (max. I=50).  The
3440 '        first row is located next to the bottom srface
3450 '
3460 ' Strand arrangement
3470 XA=(SWW-1)*CTC/2
3480 XB=B2/2-CSC
3490 FOR I=1 TO 50
3500  NR(I)=0
3510 NEXT I
3520 A1=-D5/(B2/2-B3/2)
3530 B=-A1/2*B2+D6-CSC*SQR((B2/2-B3/2)^2+D5^2)/(B2/2-B3/2)
3540 ' I = No. of rows
3550 I=0
3560 YBM=SB/AG+YB
3570 I=I+1
3580 YI=I
3590 Y1=CTC*(YI-1)+CSC
3600 IF Y1 > YBM THEN GOTO 3730 ELSE GOTO 3610
3610 IF I >= 50 GOTO 3730 ELSE GOTO 3620
3620 ' J = No. of columns within each row
3630 J=0
3640 J=J+1
3650 X2=J-1
3660 X1=CTC*X2/2
3670 IF X1 <= XA THEN GOTO 3640 ELSE GOTO 3680
3680 IF Y1 > (A1*X1+B) THEN GOTO 3710 ELSE GOTO 3690
3690 IF X1 > XB THEN GOTO 3710 ELSE GOTO 3700
3700 GOTO 3640
3710 NR(I)=J-1
3720 GOTO 3570
3730 '
3740 NS=0
3750 ' Start iteration with no. of strands increased by one each time
3760 '
3770 NS=NS+1
3780 '  Maximum number of strands set at 200
3790 IF NS > 200 THEN GOTO 3800 ELSE GOTO 3830
3800  PRINT "CANNOT SATISFY STRESS AND STRENGTH REQUIREMENTS"
3810  PRINT "PROGRAM TERMINATED"
3820  END
3830 XNS=NS
```

```
3840 AS=ASTD*XNS
3850 A2=0
3860 AY=0
3870 I=0
3880 NRT=0
3890 I=I+1
3900 IF NR(I) = 0 THEN 3910 ELSE 3940
3910  PRINT "CANNOT SATISFY STRESS AND STRENGTH REQUIREMENTS"
3920  PRINT "PROGRAM TERMINATED"
3930  END
3940 XI=I
3950 XN=NS-NRT
3960 NRT=NRT+NR(I)
3970 IF NS > NRT THEN GOTO 4020 ELSE GOTO 3980
3980 A2=A2+XN*ASTD
3990 AY=AY+XN*ASTD*(CSC+CTC*(XI-1))
4000 CE=AY/A2
4010 GOTO 4060
4020 XNR=NR(I)
4030 A2=A2+XNR*ASTD
4040 AY=AY+XNR*ASTD*(CSC+CTC*(XI-1))
4050 GOTO 3890
4060 '
4070 ET=YB-CE
4080 FI=.7*AS*FPU/1000
4090 FS=FSE*AS/1000
4100 ' Check initial stresses; "+" = compression, "-" = tension
4110 '  Top fiber stress
4120 Z1=-FCTI/1000
4130 Z2=FCCI/1000
4140 SIT=FI/AG-FI*ET/ST+XMDG*12/ST
4150 IF Z1 > SIT THEN GOTO 3760 ELSE GOTO 4160
4160 IF Z2 < SIT THEN GOTO 3760 ELSE GOTO 4170
4170 '  Bottom fiber stress
4180 SIB=FI/AG+FI*ET/SB-XMDG*12/SB
4190 IF Z1 > SIB THEN GOTO 3760 ELSE GOTO 4200
4200 IF Z2 < SIB THEN GOTO 3760 ELSE GOTO 4210
4210 ' Check service load stresses; "+" = compression, "-" = tension
4220 '  Top fiber stress
4230 Z1=-FCT/1000
4240 Z2=FCC/1000
4250 SST=FS/AG-FS*ET/ST+XMD*12/ST+XML*12/STC*(1+XIP)
4260 IF Z1 > SST THEN GOTO 3760 ELSE GOTO 4270
4270 IF Z2 < SST THEN GOTO 3760 ELSE GOTO 4280
4280 '  Bottom fiber stress
4290 SSB=FS/AG+FS*ET/SB-XMD*12/SB-XML*12/SBC*(1+XIP)
4300 IF Z1 > SSB THEN GOTO 3760 ELSE GOTO 4310
4310 IF Z2 < SSB THEN GOTO 3760 ELSE GOTO 4320
4320 '
4330 ' Substep 6B - Ultimate Positive Moment
4340 ' Input:
4350 '  From Step 1:
4360 '
B1,B2,B3,B4,D1,D2,D3,D4,D5,D6,FPU,FCD,FCP,EPS,FPY,FPM,SY,SM,SU
```

```
4370 '  From Step 2:
4380 '   FSE,FCR
4390 '  From Step 3:
4400 '   TD
4410 '  From Step 4:
4420 '   BE,AG,SB,SBC
4430 '  From Step 5:
4440 '   XMU,XMD
4450 '  From Substep 6A:
4460 '   AS,CE,FS,ET
4470 ' Output:
4480 '  XMCR = cracking moment (ft-kip)
4490 '  XMN = flexural design strength of composite section (ft-kip)
4500 '  AFSU = average stress in strands at ultimate moment (psi)
4510 '  DE = distance from extreme compression fiber to centroid of the
4520 '       strands (in) - composite section
4530 '  PA = strand ratio
4540 '  EPO = effective strain of the strand due to prestress only at service
4550 '        load condition
4560 '  EP1 = strain of the strands at ultimate moment excluding EPO
4570 '  EP2 = total strain of the strands at ultimate moment
4580 '  BP = equivalent width of web (in)
4590 '  BET1 = ratio of the depth of compression zone to the distance from
4600 '         extreme compression fiber to neutral axis
4610 '  CD = distance from extreme compression fiber to neutral axis (in)
4620 '  RIX = reinforcement index
4630 '  CDI = increment of CD (in)
4640 '  FCE = equivalent specified concrete strength = weighted average of
4650 '        girder concrete strength and deck concrete strength (psi)
4660 '  CBC = distance from extreme compression fiber to the centroid of
4670 '        compression stress block (in)
4680 '  AD = depth of equivalent rectangular compression stress block (in)
4690 '  TF = tension force in strands (lb)
4700 '  CF = compression force in compression stress block (lb)
4710 '
4720 ' Precalculated variables used in substep 6B
4730 G1=BE*TD
4740 G2=TD+D2
4750 G3=TD+D2+D3
4760 G4=TD+D2+D3+D4
4770 G5=B1*D2
4780 G6=(B1+2*B4+B3)*D3/2
4790 G7=(2*B3+2*B4)*D4/2
4800 G8=.85*FCD*TD*BE
4810 G9=.85*FCP*D2*B1
4820 G10=.85*FCP*(B1+2*B4+B3)*D3/2
4830 G11=.85*FCP*(2*B3+2*B4)*D4/2
4840 G12=TD/2
4850 G13=D2/2+TD
4860 G14=D3*(B1+4*B4+2*B3)/3/(B1+2*B4+B3)+TD+D2
4870 G15=D4*(2*B4+B3+2*B3)/3/(2*B4+B3+B3)+TD+D2+D3
4880 '
4890 ' Iteration process
4900 DE=TD+D1-CE
```

```
4910 EPO=FSE/EPS
4920 ' Initial values of CD and CDI
4930 CD=0
4940 CDI=1
4950 ' IPI = pointer for iteration (first iteration = 0, second = 1)
4960 IPI=0
4970 ' Start of ultimate moment loop
4980 '
4990 CD=CD+CDI
5000 '  Calculate AFSU and TF
5010 EP1=.003*(DE-CD)/CD
5020 EP2=EP1+EPO
5030 IF EP2 > SY THEN GOTO 5060 ELSE GOTO 5040
5040  AFSU=FPY*EP2/SY
5050  GOTO 5130
5060 IF EP2 > SM THEN GOTO 5090 ELSE GOTO 5070
5070  AFSU=FPY+(EP2-SY)/(SM-SY)*(FPM-FPY)
5080  GOTO 5130
5090 IF EP2 > SU THEN GOTO 5120 ELSE GOTO 5100
5100  AFSU=FPM+(EP2-SM)/(SU-SM)*(FPU-FPM)
5110  GOTO 5130
5120 AFSU=FPU
5130 TF=AS*AFSU
5140 '  Calculate FCE, BET1, and AD
5150 '   ADT = tentative value for AD (in^2)
5160 '   ADD = deck concrete portion of stress block area (in^2)
5170 '   AGG = girder concrete portion of stress block area (in^2)
5180 ADD=G1
5190 ADT=.85*CD
5200 IF ADT > TD THEN GOTO 5230 ELSE GOTO 5210
5210  AGG=0
5220  GOTO 5430
5230 IF ADT > G2 THEN GOTO 5260 ELSE GOTO 5240
5240  AGG=(ADT-TD)*B1
5250  GOTO 5430
5260 IF ADT > G3 THEN GOTO 5340 ELSE GOTO 5270
5270  Y1=G3-ADT
5280  Y3=D3
5290  X3=(B1-(2*B4+B3))/2
5300  X1=X3*Y1/Y3
5310  Y2=D3-Y1
5320  AGG=G5+(B1+2*X1+B3+2*B4)*Y2/2
5330  GOTO 5430
5340 IF ADT > G4 THEN GOTO 5420 ELSE GOTO 5350
5350  X3=B4
5360  Y3=D4
5370  Y1=G4-ADT
5380  X1=X3*Y1/Y3
5390  Y2=D4-Y1
5400  AGG=G5+G6+(2*B4+B3+2*X1+B3)*Y2/2
5410  GOTO 5430
5420 AGG=G5+G6+G7+B3*(ADT-G4)
5430 '
5440 FCE=(ADD*FCD+AGG*FCP)/(ADD+AGG)
```

```
5450 BET1=.85-.05*(FCE-4000)/1000
5460 IF BET1 < .65 THEN BET1=.65 ELSE GOTO 5470
5470 AD=BET1*CD
5480 '  Calculate CF,CBC,BP
5490 IF AD > TD THEN GOTO 5540 ELSE GOTO 5500
5500  CF=.85*FCD*AD*BE
5510  CBC=AD/2
5520  BP=BE
5530  GOTO 6130
5540 IF AD > G2 THEN GOTO 5630 ELSE GOTO 5550
5550  CF1=G8
5560  CF2=.85*FCP*(AD-TD)*B1
5570  CB1=G12
5580  CB2=(AD-TD)/2+TD
5590  CF=CF1+CF2
5600  CBC=(CF1*CB1+CF2*CB2)/CF
5610  BP=B1
5620  GOTO 6130
5630 IF AD > G3 THEN GOTO 5800 ELSE GOTO 5640
5640  Y3=D3
5650  Y1=G3-AD
5660  X3=(B1-(2*B4+B3))/2
5670  X1=X3*Y1/Y3
5680  Y2=D3-Y1
5690  XX=2*X1+B3+2*B4
5700  CF1=G8
5710  CF2=G9
5720  CF3=.85*FCP*(B1+XX)*Y2/2
5730  CB1=G12
5740  CB2=G13
5750  CB3=Y2*(B1+2*XX)/3/(B1+XX)+G2
5760  CF=CF1+CF2+CF3
5770  CBC=(CF1*CB1+CF2*CB2+CF3*CB3)/CF
5780  BP=XX
5790  GOTO 6130
5800 IF AD > G4 THEN GOTO 5990 ELSE GOTO 5810
5810  X3=B4
5820  Y3=D4
5830  Y1=G4-AD
5840  Y2=D4-Y1
5850  X1=X3*Y1/Y3
5860  XX=2*X1+B3
5870  CF1=G8
5880  CF2=G9
5890  CF3=G10
5900  CF4=.85*FCP*(2*B4+B3+2*X1+B3)*Y2/2
5910  CB1=G12
5920  CB2=G13
5930  CB3=G14
5940  CB4=Y2*(2*B4+B3+2*XX)/3/(2*B4+B3+XX)+G3
5950  CF=CF1+CF2+CF3+CF4
5960  CBC=(CF1*CB1+CF2*CB2+CF3*CB3+CF4*CB4)/CF
5970  BP=XX
5980  GOTO 6130
```

```
5990 '
6000 CF1=G8
6010 CF2=G9
6020 CF3=G10
6030 CF4=G11
6040 CF5=.85*FCP*(AD-G4)*B3
6050 CB1=G12
6060 CB2=G13
6070 CB3=G14
6080 CB4=G15
6090 CB5=(AD-G4)/2+G4
6100 CF=CF1+CF2+CF3+CF4+CF5
6110 CBC=(CF1*CB1+CF2*CB2+CF3*CB3+CF4*CB4+CF5*CB5)/CF
6120 BP=B3
6130 '
6140 IF TF > CF THEN GOTO 4980 ELSE GOTO 6150
6150 '  Check whether the first iteration or the second
6160 IF IPI = 1 THEN GOTO 6220 ELSE GOTO 6170
6170  IPI=1
6180  CD=CD-CDI
6190  CDI=.1
6200  GOTO 4980
6210 ' End of ultimate moment loop
6220 '
6230 ' After the second iteration
6240 XMNS=(CF+TF)/2*(DE-CBC)/12/1000
6250 XMN=.9*XMNS
6260 IF XMN < XMU THEN GOTO 3760 ELSE GOTO 6270
6270 ' Calculate cracking moment
6280 XMCR=SBC/12*(FS/AG+FS*ET/SB-XMD*12/SB+FCR/1000)
6290 IF XMNS < (1.2*XMCR) THEN GOTO 3760 ELSE GOTO 6300
6300 ' End of iteration
6310 '
6320 ' Check maximum reinforcement index
6330 IF AD > TD THEN GOTO 6370 ELSE GOTO 6340
6340  FCE=FCD
6350  PA=AS/BP/DE
6360  GOTO 6380
6370 PA=AD*.85*FCE/AFSU/DE
6380 RIX=PA*AFSU/FCE
6390 IF RIX > .3 THEN GOTO 6400 ELSE GOTO 6420
6400  PRINT "REINFORCEMENT INDEX EXCEEDS 0.3, PROGRAM
TERMINATED"
6410  END
6420 '
6430 ' *********************** END OF STEP 6 ***********************
6440 '
6450 ' ************** STEP 7 - DEFLECTIONS AT MIDSPAN **************
6460 '
6470 ' Input:
6480 '  From Step 1:
6490 '    SRF,SL
6500 '  From Step 2:
6510 '    EC,ECI
```

```
6520 '  From Step 4:
6530 '   XIG
6540 '  From Step 5:
6550 '   WUG,WUD
6560 '  From Substep 6A:
6570 '   FI,ET
6580 ' Output:
6590 '  DUP = upward deflection due to prestressing (straight strands) (in)
6600 '  DDG = downward deflection due to girder weight (in)
6610 '  CAMB = resultant camber at erection (in)
6620 '  DDD = downward deflection due to deck weight (in)
6630 '
6640 ' Calculate deflections
6650 DUP=.125*FI*ET*SL^2*12^2*1000/SRF/ECI/XIG
6660 DDG=5*WUG*SL^4*12^3*1000/SRF/384/ECI/XIG
6670 CAMB=DUP-DDG
6680 DDD=5*WUD*SL^4*12^3*1000/384/EC/XIG
6690 '
6700 ' *********************** END OF STEP 7 ***********************
6710 '
6720 ' ******* STEP 8 - COST PER UNIT SURFACE AREA OF BRIDGE *******
6730 '
6740 ' Input:
6750 '  From Step 1:
6760 '    SL,GS,WC,WCD,RUCG,RUCD,RUS,RUR,RUE,UST
6770 '  From Step 3:
6780 '   TD,DREINFWT,DREINFSP
6790 '  From Step 4:
6800 '   AG
6810 '  From Substep 6A:
6820 '   NS
6830 ' Output:
6840 '  TWS = total weight of strands per girder (lb)
6850 '  TWR = total weight of deck reinforcement per girder (lb)
6860 '  TWCG = total weight of girder concrete per girder (lb)
6870 '  TWCD = total weight of deck concrete per girder (lb)
6880 '  WUS = strand weight per unit surface area (psf)
6890 '  WUR = deck reinforcement weight per unit surface area (psf)
6900 '  WUCG = girder concrete weight per unit surface area (psf)
6910 '  WUCD = deck concrete weight per unit surface area (psf)
6920 '  CUS = cost of strand per unit surface area (unit/ft^2)
6930 '  CUR = cost of deck reinforcement per unit surface area (unit/ft^2)
6940 '  CUCG = cost of girder concrete per unit surface area (unit/ft^2)
6950 '  CUCD = cost of deck concrete per unit surface area (unit/ft^2)
6960 '  TUT = total cost index per unit surface area based on relative
6970 '        unit costs (unit/ft^2)
6980 '
6990 ' Cost calculations
7000 TWS=NS*SL*UST
7010 TWR=SL*12*GS*2*DREINFWT/DREINFSP
7020 TWCG=AG*SL*WC/12^2
7030 TWCD=TD*GS*SL*WCD/12
7040 WUS=TWS/SL/GS
7050 WUR=TWR/SL/GS
```

```
7060 WUCG=TWCG/SL/GS
7070 WUCD=TWCD/SL/GS
7080 CUS=RUS*WUS
7090 CUR=(RUR+RUE)/2*WUR
7100 CUCG=RUCG*WUCG
7110 CUCD=RUCD*WUCD
7120 TUT=CUS+CUR+CUCG+CUCD
7130 '
7140 ' *********************** END OF STEP 8 **********************
7150 '
7160 ' ********************* STEP 9 - OUTPUT *********************
7170 '
7180 ' Input:
7190 '  All previous steps
7200 ' Output:
7210 '  Either full output or brief output
7220 '
7230 IF OUTPUT = 0 THEN GOTO 9410 ELSE GOTO 7240
7240 '
7250 ' Full output
7260 LPRINT "    ******************************************************
****************************"
7270 LPRINT "    *
                                        *"
7280 LPRINT "    * BRIDGE - STRUCTURAL DESIGN OF PRESTRESSED
CONCRETE BRIDGE GIRDERS *"
7290 LPRINT "    *
                                        *"
7300 LPRINT "    ******************************************************
****************************"
7310 LPRINT " "
7320 LPRINT " "
7330 LPRINT "    TITLE:"TITLE$
7340 LPRINT " "
7350 LPRINT " "
7360 LPRINT "    INPUT DATA:"
7370 LPRINT " "
7380 LPRINT "      Geometric Properties:"
7390 LPRINT "        Girder span length (ft) = " USING "###.#";SL
7400 LPRINT "        Girder spacing (ft) = " USING "##.#";GS
7410 LPRINT " "
7420 LPRINT "        Horizontal dimensions of girder cross section (in):"
7430 LPRINT "          B1 ="B1"  B2 ="B2"  B3 ="B3"  B4 ="B4"
7440 LPRINT " "
7450 LPRINT "        Vertical dimensions of girder cross section (in):"
7460 LPRINT "          D1 ="D1"  D2 ="D2"  D3 ="D3"  D4 ="D4"  D5 ="D5"  D6 ="D6
7470 LPRINT " "
7480 LPRINT "        Center to center spacing of strands (in) = " USING "#.#";CTC
7490 LPRINT "        Concrete surface to center of strands (in) = " USING "#.#";CSC
7500 LPRINT "        Number of strands within web width = " USING "##";SWW
7510 LPRINT "        Nominal area of each strand (in^2) = " USING "#.###";ASTD
7520 LPRINT " "
7530 LPRINT "      Material Properties:"
7540 LPRINT "        Concrete:"
```

```
7550 LPRINT "       Girder concrete strength (psi) = " USING "#####.#";FCP
7560 LPRINT "       Girder concrete strength at prestress transfer (psi) = " USING
"#####.#";FCPI
7570 LPRINT "       Deck concrete strength (psi) = " USING "#####.#";FCD
7580 LPRINT " "
7590 LPRINT "       Unit weight of girder concrete (pcf) = " USING "###.#";WC
7600 LPRINT "       Unit weight of deck concrete (pcf) = " USING "###.#";WCD
7610 LPRINT "       Unit weight of girder (pcf) = " USING "###.#";WG
7620 LPRINT "       Unit weight of deck (pcf) = " USING "###.#";WD
7630 LPRINT " "
7640 LPRINT "     Strand:"
7650 LPRINT "       Tri-linear stress-strain curve of strand:"
7660 LPRINT " "
7670 LPRINT "                          stress(psi)   strain"
7680 LPRINT "         ultimate strength      "FPU"      "USING"#.####";SU
7690 LPRINT "         intermediate stress    "FPM"      "USING"#.####";SM
7700 LPRINT "         yield stress           "FPY"      "USING"#.####";SY
7710 LPRINT " "
7720 LPRINT "       Modulus of elasticity of strand (psi) = " USING "########.#";EPS
7730 LPRINT "       Total prestress losses (psi) = " USING "#####.#";XLS
7740 LPRINT " "
7750 LPRINT "     Girder Stiffness Reduction Factor = " USING "#.###";SRF
7760 LPRINT " "
7770 LPRINT "    Relative Cost Index (unit/lb):"
7780 LPRINT "      Girder concrete = " USING "##.#";RUCG
7790 LPRINT "      Deck concrete = " USING "##.#";RUCD
7800 LPRINT "      Strand = " USING "##.#";RUS
7810 LPRINT "      Deck reinforcement = " USING "##.#";RUR
7820 LPRINT "      Epoxy coated deck reinforcement = " USING "##.#";RUE
7830 LPRINT " "
7840 LPRINT " "
7850 LPRINT "   CALCULATED MATERIAL PROPERTIES:"
7860 LPRINT " "
7870 LPRINT "    Concrete:"
7880 LPRINT "      Allowable girder concrete stresses (psi):"
7890 LPRINT "        At prestress transfer:"
7900 LPRINT "          compressive stress = " USING "#####.#";FCCI
7910 LPRINT "          tensile stress = " USING "####.#";FCTI
7920 LPRINT "        At service load:"
7930 LPRINT "          compressive stress = " USING "#####.#";FCC
7940 LPRINT "          tensile stress = " USING "####.#";FCT
7950 LPRINT " "
7960 LPRINT "      Modulus of rupture of girder concrete (psi) =" USING
"####.#";FCR
7970 LPRINT " "
7980 LPRINT "      Modulus of elasticity:"
7990 LPRINT "        Girder concrete (psi) = " USING "#######.#";EC
8000 LPRINT "        Girder concrete at prestress transfer (psi) = " USING
"#######.#";ECI
8010 LPRINT "        Deck concrete (psi) = " USING "#######.#";ECD
8020 LPRINT " "
8030 LPRINT "      Modular ratio (deck to girder) = " USING "#.###";XNE
8040 LPRINT " "
8050 LPRINT "    Strand:"
```

```
8060 LPRINT "      Allowable stress at service load (psi) = " USING "######.#";FPS
8070 LPRINT "      Effective prestress at service load (psi) = " USING "######.#";FSE
8080 LPRINT " "
8090 LPRINT " "
8100 LPRINT "  DECK DESIGN:"
8110 LPRINT " "
8120 LPRINT "     Effective span of deck (ft) = " USING "##.##";SE
8130 LPRINT "     Thickness of deck (in) = " USING "##.##";TD
8140 LPRINT "     Spacing of No."DREINFNO"bars (in) = " USING "##.#";DREINFSP
8150 LPRINT " "
8160 LPRINT " "
8170 LPRINT "  SECTION PROPERTIES OF GIRDER:"
8180 LPRINT " "
8190 LPRINT "     Effective top flange (deck) width (in) = " USING "###.#";BE
8200 LPRINT " "
8210 LPRINT "     Noncomposite Section:"
8220 LPRINT "       Area (in^2) = " USING "####.#";AG
8230 LPRINT "       Distance from centroid to top fiber (in) = " USING "##.#";YT
8240 LPRINT "       Distance from centroid to bottom fiber (in) = " USING "##.#";YB
8250 LPRINT " "
8260 LPRINT "       Moment of inertia (in^4) = " USING "#######.#";XIG
8270 LPRINT "       Section modulus for top fiber (in^3) = " USING "######.#";ST
8280 LPRINT "       Section modulus for bottom fiber (in^3) = " USING "######.#";SB
8290 LPRINT " "
8300 LPRINT "     Composite Section:"
8310 LPRINT "       Area (in^2) = " USING "####.#";AC
8320 LPRINT "       Distance from centroid to top fiber (in) = " USING "##.#";YTC
8330 LPRINT "       Distance from centroid to bottom fiber (in) = " USING "##.#";YBC
8340 LPRINT " "
8350 LPRINT "       Moment of inertia (in^4) = " USING "#######.#";XIGC
8360 LPRINT "       Section modulus for top fiber (in^3) = " USING "######.#";STC
8370 LPRINT "       Section modulus for bottom fiber (in^3) = " USING
"######.#";SBC
8380 LPRINT " "
8390 LPRINT " "
8400 LPRINT "  DESIGN LOADS AND MOMENTS:"
8410 LPRINT " "
8420 LPRINT "     Impact load factor = " USING "#.###";XIP
8430 LPRINT " "
8440 LPRINT "     Design Loads:"
8450 LPRINT "       Uniform load due to girder weight (kip/ft) = " USING
"##.###";WUG
8460 LPRINT "       Uniform load due to deck weight (kip/ft) = " USING "##.###";WUD
8470 LPRINT " "
8480 LPRINT "     Design Moments:"
8490 LPRINT "       Moment due to girder weight (ft-kip) = " USING "#####.#";XMDG
8500 LPRINT "       Moment due to deck plus girder weight (ft-kip) = " USING
"#####.#";XMD
8510 LPRINT "       Moment due to live load (ft-kip) = " USING "#####.#";XML
8520 LPRINT "       Factored moment (ft-kip) = " USING "#####.#";XMU
8530 LPRINT " "
8540 LPRINT " "
8550 LPRINT " "
8560 LPRINT " "
```

```
8570 LPRINT " "
8580 LPRINT "   REQUIRED NUMBER OF PRESTRESSING STRANDS:"
8590 LPRINT " "
8600 LPRINT "     Strand Layout:"
8610 LPRINT "       Total No. of strands required = "NS
8620 LPRINT " "
8630 LPRINT "         Row     No. of strands per row"
8640 NRR=0
8650 FOR I=1 TO 50
8660  NRR=NRR+NR(I)
8670  IF NRR <= NS THEN 8680 ELSE 8690
8680   LPRINT "          "I"           "NR(I)
8690  IF NRR > NS AND (NRR-NR(I)) < NS THEN 8700 ELSE 8710
8700   LPRINT "          "I"           "NS-(NRR-NR(I))
8710 NEXT I
8720 LPRINT " "
8730 LPRINT "       Total area of strands required (in^2) = " USING "##.##";AS
8740 LPRINT " "
8750 LPRINT "     At Prestress Transfer:"
8760 LPRINT "       Prestressing force (kip) = " USING "#####.#";FI
8770 LPRINT "       Top fiber stress (psi) = " USING "####.#";SIT*1000
8780 LPRINT "       Bottom fiber stress (psi) = " USING "####.#";SIB*1000
8790 LPRINT " "
8800 LPRINT "     At Service Load:"
8810 LPRINT "       Prestressing force (kip) = " USING "#####.#";FS
8820 LPRINT "       Top fiber stress (psi) = " USING "####.#";SST*1000
8830 LPRINT "       Bottom fiber stress (psi) = " USING "####.#";SSB*1000
8840 LPRINT " "
8850 LPRINT "     Strains at Centroid of Strands:"
8860 LPRINT "       Strain due to prestress only at service load = " USING "#.####";EPO
8870 LPRINT "       Total strain at ultimate moment = " USING "#.####";EP2
8880 LPRINT " "
8890 LPRINT "     Geometric Parameters of Girder Section:"
8900 LPRINT "       Centroid of strands to centroid of noncomp. girder (in) = " USING
"###.#";ET
8910 LPRINT "       Centroid of strands to bottom fiber (in) = " USING "##.#";CE
8920 LPRINT "       Top fiber of comp. section to centroid of strands (in) = " USING
"###.#";DE
8930 LPRINT "       Top fiber of comp. section to neutral axis (in) = " USING
"###.#";CD
8940 LPRINT "       Ratio of stress block depth to neutral axis depth (in) = " USING
"#.##";BET1
8950 LPRINT "       Compression stress block depth (in) = " USING "##.##";AD
8960 LPRINT "       Top fiber to centroid of stress block, composite (in) = " USING
"##.##";CBC
8970 LPRINT "       Equivalent width of web, composite section (in) = " USING
"###.#";BP
8980 LPRINT " "
8990 LPRINT "     Average stress in strands at ultimate moment (psi) = " USING
"######.#";AFSU
9000 LPRINT "     Weighted average concrete strength (psi) = " USING "#####.#";FCE
9010 LPRINT "     Strand ratio = " USING "#.#####";PA
9020 LPRINT "     Reinforcement index = " USING "#.###";RIX
9030 LPRINT " "
```

```
9040 LPRINT "     Tensile force in strands (lb) = " USING "########.#";TF
9050 LPRINT "     Compressive force in stress block (lb) = " USING "########.#";CF
9060 LPRINT "     Flexural design strength of composite section (ft-kip) = " USING
"#####.#";XMN
9070 LPRINT "     Cracking moment of girder (ft-kip) = " USING "#####.#";XMCR
9080 LPRINT " "
9090 LPRINT " "
9100 LPRINT "   DEFLECTIONS AT MIDSPAN:"
9110 LPRINT " "
9120 LPRINT "     Upward deflection due to straight prestressing strands (in) = " USING
"##.##";DUP
9130 LPRINT "     Downward deflection due to girder weight (in) = " USING
"##.##";DDG
9140 LPRINT "     Resultant camber at erection (in) = " USING "##.##";CAMB
9150 LPRINT "     Downward deflection due to deck weight (in) = " USING
"##.##";DDD
9160 LPRINT " "
9170 LPRINT " "
9180 LPRINT "   COST INDEX PER UNIT SURFACE AREA OF BRIDGE:"
9190 LPRINT " "
9200 LPRINT "    Total Weight of Each Material per Girder:"
9210 LPRINT "       Strand (lb) = " USING "#####.#";TWS
9220 LPRINT "       Deck reinforcement (lb) = " USING "#####.#";TWR
9230 LPRINT "       Girder concrete (lb) = " USING "#######.#";TWCG
9240 LPRINT "       Deck concrete (lb) = " USING "######.#";TWCD
9250 LPRINT " "
9260 LPRINT "    Weight of Each Material per Unit Surface Area:"
9270 LPRINT "       Strand (psf) = " USING "##.##";WUS
9280 LPRINT "       Deck reinforcement (psf) = " USING "##.##";WUR
9290 LPRINT "       Girder concrete (psf) = " USING "####.##";WUCG
9300 LPRINT "       Deck concrete (psf) = " USING "###.##";WUCD
9310 LPRINT " "
9320 LPRINT "    Cost Index of Each Material per Unit Surface Area:"
9330 LPRINT "       Strand (unit/ft^2) = " USING "###.##";CUS
9340 LPRINT "       Deck reinforcement (unit/ft^2) = " USING "###.##";CUR
9350 LPRINT "       Girder concrete (unit/ft^2) = " USING "####.##";CUCG
9360 LPRINT "       Deck concrete (unit/ft^2) = " USING "####.##";WUCD
9370 LPRINT " "
9380 LPRINT "    Total Cost Index per Unit Surface Area of Bridge:"
9390 LPRINT "       Total relative unit cost (unit/ft^2) = " USING "####.##";TUT
9400 '
9410 '
9420 '*********************** END OF STEP 9 **********************
```

SAMPLE PROBLEMS

Output for two sample problems is presented in the following pages. Sample Problem 1 is the design of an AASHTO Type VI girder, while Sample Problem 2 is the design of a PCI Bulb-Tee girder. Both examples are for spans of 140 ft (42.7 m), girder spacing of 6.0 ft (1.80 m), and a 28-day girder concrete compressive strength of 10,000 psi (69.0 MPa).

Input for the problems consists of READ and DATA statements within Step 1 of the program, lines 1020 through 1070. For each example, the program lines correspond to the following:

Sample Problem 1 - AASHTO Type VI

```
READ  SL, GS, B1, B2, B3, B4, D1, D2, D3, D4, D5, D6, CTC, CSC, SWW,
      FCP, FCPI, WC, WD, WCD, WG, WD

READ  FCD, FPY, FPM, FPU, EPS, SY, SM, SU, XLS

READ  ASTD, UST, SRF, RUCG, RUCD, RUS, RUR, RUE

DATA  140, 6, 42, 28, 8, 4, 72, 5, 3, 4, 10, 8, 2, 2, 2, 10000, 7500, 155, 145,
      160, 150

DATA  4000, 230000, 255000, 270000, 28000000, 0.008214, 0.012, 0.040, 45000

DATA  0.153, 0.530, 0.55, 1, 1, 8, 9,12
```

Sample Problem 2 - PCI Bulb-Tee

```
READ  SL, GS, B1, B2, B3, B4, D1, D2, D3, D4, D5, D6, CTC, CSC, SWW,
      FCP, FCPI, WC, WD, WCD, WG, WD

READ  FCD, FPY, FPM, FPU, EPS, SY, SM, SU, XLS

READ  ASTD, UST, SRF, RUCG, RUCD, RUS, RUR, RUE

DATA  140, 6, 42, 26, 6, 2, 72, 3.5, 2, 2, 4.5, 6, 2, 2, 2, 10000, 7500, 155, 145,
      160, 150

DATA  4000, 230000, 255000, 270000, 28000000, 0.008214, 0.012, 0.040, 45000

DATA  0.153, 0.530, 0.55, 1, 1, 8, 9,12
```

```
*********************************************************************
*                                                                   *
* BRIDGE - STRUCTURAL DESIGN OF PRESTRESSED CONCRETE BRIDGE GIRDERS *
*                                                                   *
*********************************************************************


TITLE: FHWA SAMPLE PROBLEM NO. 1, AASHTO TYPE VI


INPUT DATA:

  Geometric Properties:
    Girder span length (ft) = 140.0
    Girder spacing (ft) =  6.0

    Horizontal dimensions of girder cross section (in):
      B1 = 42   B2 = 28   B3 = 8   B4 = 4

    Vertical dimensions of girder cross section (in):
      D1 = 72   D2 = 5   D3 = 3   D4 = 4   D5 = 10   D6 = 8

    Center to center spacing of strands (in) = 2.0
    Concrete surface to center of strands (in) = 2.0
    Number of strands within web width =  2
    Nominal area of each strand (in^2) = 0.153

  Material Properties:
    Concrete:
      Girder concrete strength (psi) = 10000.0
      Girder concrete strength at prestress transfer (psi) =  7500.0
      Deck concrete strength (psi) =  4000.0

      Unit weight of girder concrete (pcf) = 155.0
      Unit weight of deck concrete (pcf) = 145.0
      Unit weight of girder (pcf) = 160.0
      Unit weight of deck (pcf) = 150.0

    Strand:
      Tri-linear stress-strain curve of strand:

                                  stress(psi)   strain
        ultimate strength          270000       0.0400
        intermediate stress        255000       0.0120
        yield stress               230000       0.0082

      Modulus of elasticity of strand (psi) = 28000000.0
      Total prestress losses (psi) = 45000.0

  Girder Stiffness Reduction Factor = 0.550

  Relative Cost Index (unit/lb):
    Girder concrete =  1.0
    Deck concrete =  1.0
    Strand =  8.0
    Deck reinforcement =  9.0
    Epoxy coated deck reinforcement = 12.0


CALCULATED MATERIAL PROPERTIES:

  Concrete:
    Allowable girder concrete stresses (psi):
      At prestress transfer:
        compressive stress =  4500.0
        tensile stress =    0.0
```

```
      At service load:
        compressive stress =  4000.0
        tensile stress =  600.0

    Modulus of rupture of girder concrete (psi) = 750.0

    Modulus of elasticity:
      Girder concrete (psi) = 6368123.0
      Girder concrete at prestress transfer (psi) = 5514956.0
      Deck concrete (psi) = 3644146.0

    Modular ratio (deck to girder) = 0.572

  Strand:
    Allowable stress at service load (psi) = 184000.0
    Effective prestress at service load (psi) = 144000.0


DECK DESIGN:

  Effective span of deck (ft) =  4.25
  Thickness of deck (in) =  7.00
  Spacing of No. 5 bars (in) =  7.5


SECTION PROPERTIES OF GIRDER:

  Effective top flange (deck) width (in) =  72.0

  Noncomposite Section:
    Area (in^2) = 1085.0
    Distance from centroid to top fiber (in) = 35.6
    Distance from centroid to bottom fiber (in) = 36.4

    Moment of inertia (in^4) =  733359.3
    Section modulus for top fiber (in^3) =  20588.8
    Section modulus for bottom fiber (in^3) =  20158.0

  Composite Section:
    Area (in^2) = 1373.4
    Distance from centroid to top fiber (in) = 27.4
    Distance from centroid to bottom fiber (in) = 44.6

    Moment of inertia (in^4) = 1083217.0
    Section modulus for top fiber (in^3) =  39527.1
    Section modulus for bottom fiber (in^3) =  24289.8


DESIGN LOADS AND MOMENTS:

  Impact load factor = 0.189

  Design Loads:
    Uniform load due to girder weight (kip/ft) =  1.206
    Uniform load due to deck weight (kip/ft) =  0.525

  Design Moments:
    Moment due to girder weight (ft-kip) =  2953.6
    Moment due to deck plus girder weight (ft-kip) =  4239.9
    Moment due to live load (ft-kip) =  1345.7
    Factored moment (ft-kip) =  8984.5
```

REQUIRED NUMBER OF PRESTRESSING STRANDS:

Strand Layout:
Total No. of strands required = 50

Row	No. of strands per row
1	13
2	13
3	13
4	11

Total area of strands required (in^2) = 7.65

At Prestress Transfer:
Prestressing force (kip) = 1445.8
Top fiber stress (psi) = 841.9
Bottom fiber stress (psi) = 1833.7

At Service Load:
Prestressing force (kip) = 1101.6
Top fiber stress (psi) = 2286.6
Bottom fiber stress (psi) = -577.5

Strains at Centroid of Strands:
Strain due to prestress only at service load = 0.0051
Total strain at ultimate moment = 0.0255

Geometric Parameters of Girder Section:
Centroid of strands to centroid of noncomp. girder (in) = 31.5
Centroid of strands to bottom fiber (in) = 4.9
Top fiber of comp. section to centroid of strands (in) = 74.1
Top fiber of comp. section to neutral axis (in) = 9.5
Ratio of stress block depth to neutral axis depth (in) = 0.83
Compression stress block depth (in) = 7.84
Top fiber to centroid of stress block, composite (in) = 4.08
Equivalent width of web, composite section (in) = 42.0

Average stress in strands at ultimate moment (psi) = 262258.5
Weighted average concrete strength (psi) = 4493.3
Strand ratio = 0.00154
Reinforcement index = 0.090

Tensile force in strands (lb) = 2006277.0
Compressive force in stress block (lb) = 2013723.0
Flexural design strength of composite section (ft-kip) = 10557.9
Cracking moment of girder (ft-kip) = 1948.8

DEFLECTIONS AT MIDSPAN:

Upward deflection due to straight prestressing strands (in) = 7.22
Downward deflection due to girder weight (in) = 4.68
Resultant camber at erection (in) = 2.54
Downward deflection due to deck weight (in) = 0.97

COST INDEX PER UNIT SURFACE AREA OF BRIDGE:

Total Weight of Each Material per Girder:
Strand (lb) = 3710.0
Deck reinforcement (lb) = 2803.6
Girder concrete (lb) = 163503.5
Deck concrete (lb) = 71050.0

Weight of Each Material per Unit Surface Area:
Strand (psf) = 4.42

```
  Deck reinforcement (psf) =  3.34
  Girder concrete (psf) =  194.65
  Deck concrete (psf) =  84.58

Cost Index of Each Material per Unit Surface Area:
  Strand (unit/ft^2) =  35.33
  Deck reinforcement (unit/ft^2) =  35.04
  Girder concrete (unit/ft^2) =  194.65
  Deck concrete (unit/ft^2) =   84.58

Total Cost Index per Unit Surface Area of Bridge:
  Total relative unit cost (unit/ft^2) =  349.61
```

```
*********************************************************************
*                                                                   *
* BRIDGE - STRUCTURAL DESIGN OF PRESTRESSED CONCRETE BRIDGE GIRDERS *
*                                                                   *
*********************************************************************


TITLE: FHWA SAMPLE PROBLEM NO. 2, PCI BT-72


INPUT DATA:

  Geometric Properties:
    Girder span length (ft) = 140.0
    Girder spacing (ft) =  6.0

    Horizontal dimensions of girder cross section (in):
      B1 = 42   B2 = 26   B3 = 6   B4 = 2

    Vertical dimensions of girder cross section (in):
      D1 = 72   D2 = 3.5   D3 = 2   D4 = 2   D5 = 4.5   D6 = 6

    Center to center spacing of strands (in) = 2.0
    Concrete surface to center of strands (in) = 2.0
    Number of strands within web width =  2
    Nominal area of each strand (in^2) = 0.153

  Material Properties:
    Concrete:
      Girder concrete strength (psi) = 10000.0
      Girder concrete strength at prestress transfer (psi) =  7500.0
      Deck concrete strength (psi) =  4000.0

      Unit weight of girder concrete (pcf) = 155.0
      Unit weight of deck concrete (pcf) = 145.0
      Unit weight of girder (pcf) = 160.0
      Unit weight of deck (pcf) = 150.0

    Strand:
      Tri-linear stress-strain curve of strand:

                                   stress(psi)   strain
        ultimate strength           270000        0.0400
        intermediate stress         255000        0.0120
        yield stress                230000        0.0082

      Modulus of elasticity of strand (psi) = 28000000.0
      Total prestress losses (psi) = 45000.0

  Girder Stiffness Reduction Factor = 0.550

  Relative Cost Index (unit/lb):
    Girder concrete =  1.0
    Deck concrete =  1.0
    Strand =  8.0
    Deck reinforcement =  9.0
    Epoxy coated deck reinforcement = 12.0


CALCULATED MATERIAL PROPERTIES:

  Concrete:
    Allowable girder concrete stresses (psi):
      At prestress transfer:
        compressive stress =  4500.0
        tensile stress =    0.0
```

```
      At service load:
        compressive stress =  4000.0
        tensile stress =  600.0

    Modulus of rupture of girder concrete (psi) = 750.0

    Modulus of elasticity:
      Girder concrete (psi) = 6368123.0
      Girder concrete at prestress transfer (psi) = 5514956.0
      Deck concrete (psi) = 3644146.0

    Modular ratio (deck to girder) = 0.572

  Strand:
    Allowable stress at service load (psi) = 184000.0
    Effective prestress at service load (psi) = 144000.0


DECK DESIGN:

  Effective span of deck (ft) =  4.25
  Thickness of deck (in) =  7.00
  Spacing of No. 5 bars (in) =  7.5


SECTION PROPERTIES OF GIRDER:

  Effective top flange (deck) width (in) =  72.0

  Noncomposite Section:
    Area (in^2) =  767.0
    Distance from centroid to top fiber (in) = 35.4
    Distance from centroid to bottom fiber (in) = 36.6

    Moment of inertia (in^4) =  545871.5
    Section modulus for top fiber (in^3) =  15421.7
    Section modulus for bottom fiber (in^3) =  14913.0

  Composite Section:
    Area (in^2) = 1055.4
    Distance from centroid to top fiber (in) = 24.8
    Distance from centroid to bottom fiber (in) = 47.2

    Moment of inertia (in^4) =  864155.9
    Section modulus for top fiber (in^3) =  34891.2
    Section modulus for bottom fiber (in^3) =  18295.7


DESIGN LOADS AND MOMENTS:

  Impact load factor = 0.189

  Design Loads:
    Uniform load due to girder weight (kip/ft) =  0.852
    Uniform load due to deck weight (kip/ft) =  0.525

  Design Moments:
    Moment due to girder weight (ft-kip) =  2087.9
    Moment due to deck plus girder weight (ft-kip) =  3374.2
    Moment due to live load (ft-kip) =  1345.7
    Factored moment (ft-kip) =  7859.1
```

REQUIRED NUMBER OF PRESTRESSING STRANDS:

Strand Layout:
Total No. of strands required = 43

Row	No. of strands per row
1	12
2	12
3	9
4	4
5	2
6	2
7	2

Total area of strands required (in^2) = 6.58

At Prestress Transfer:
Prestressing force (kip) = 1243.4
Top fiber stress (psi) = 725.8
Bottom fiber stress (psi) = 2547.1

At Service Load:
Prestressing force (kip) = 947.4
Top fiber stress (psi) = 2490.8
Bottom fiber stress (psi) = -543.6

Strains at Centroid of Strands:
Strain due to prestress only at service load = 0.0051
Total strain at ultimate moment = 0.0284

Geometric Parameters of Girder Section:
Centroid of strands to centroid of noncomp. girder (in) = 31.3
Centroid of strands to bottom fiber (in) = 5.3
Top fiber of comp. section to centroid of strands (in) = 73.7
Top fiber of comp. section to neutral axis (in) = 8.4
Ratio of stress block depth to neutral axis depth (in) = 0.85
Compression stress block depth (in) = 7.11
Top fiber to centroid of stress block, composite (in) = 3.58
Equivalent width of web, composite section (in) = 42.0

Average stress in strands at ultimate moment (psi) = 263810.8
Weighted average concrete strength (psi) = 4069.2
Strand ratio = 0.00127
Reinforcement index = 0.082

Tensile force in strands (lb) = 1735611.0
Compressive force in stress block (lb) = 1753206.0
Flexural design strength of composite section (ft-kip) = 9167.4
Cracking moment of girder (ft-kip) = 1914.3

DEFLECTIONS AT MIDSPAN:

Upward deflection due to straight prestressing strands (in) = 8.28
Downward deflection due to girder weight (in) = 4.45
Resultant camber at erection (in) = 3.83
Downward deflection due to deck weight (in) = 1.31

COST INDEX PER UNIT SURFACE AREA OF BRIDGE:

Total Weight of Each Material per Girder:
Strand (lb) = 3190.6
Deck reinforcement (lb) = 2803.6
Girder concrete (lb) = 115582.6
Deck concrete (lb) = 71050.0

```
Weight of Each Material per Unit Surface Area:
  Strand (psf) =  3.80
  Deck reinforcement (psf) =  3.34
  Girder concrete (psf) =  137.60
  Deck concrete (psf) =  84.58

Cost Index of Each Material per Unit Surface Area:
  Strand (unit/ft^2) =  30.39
  Deck reinforcement (unit/ft^2) =  35.04
  Girder concrete (unit/ft^2) =  137.60
  Deck concrete (unit/ft^2) =   84.58

Total Cost Index per Unit Surface Area of Bridge:
  Total relative unit cost (unit/ft^2) =  287.61
```

APPENDIX D - COST CHARTS

This appendix contains cost charts for the following girder cross sections:

1. BT-54.
2. BT-72.
3. AASHTO Type VI.
4. Washington Modified 14/6.
5. Colorado Modified G68/6.
6. Nebraska 1800.
7. Florida BT-72.
8. Texas U54B.

These cost charts consist of optimum cost curves at 6,000, 8,000, 10,000, and 12,000 psi (41, 55, 69, and 83 MPa).

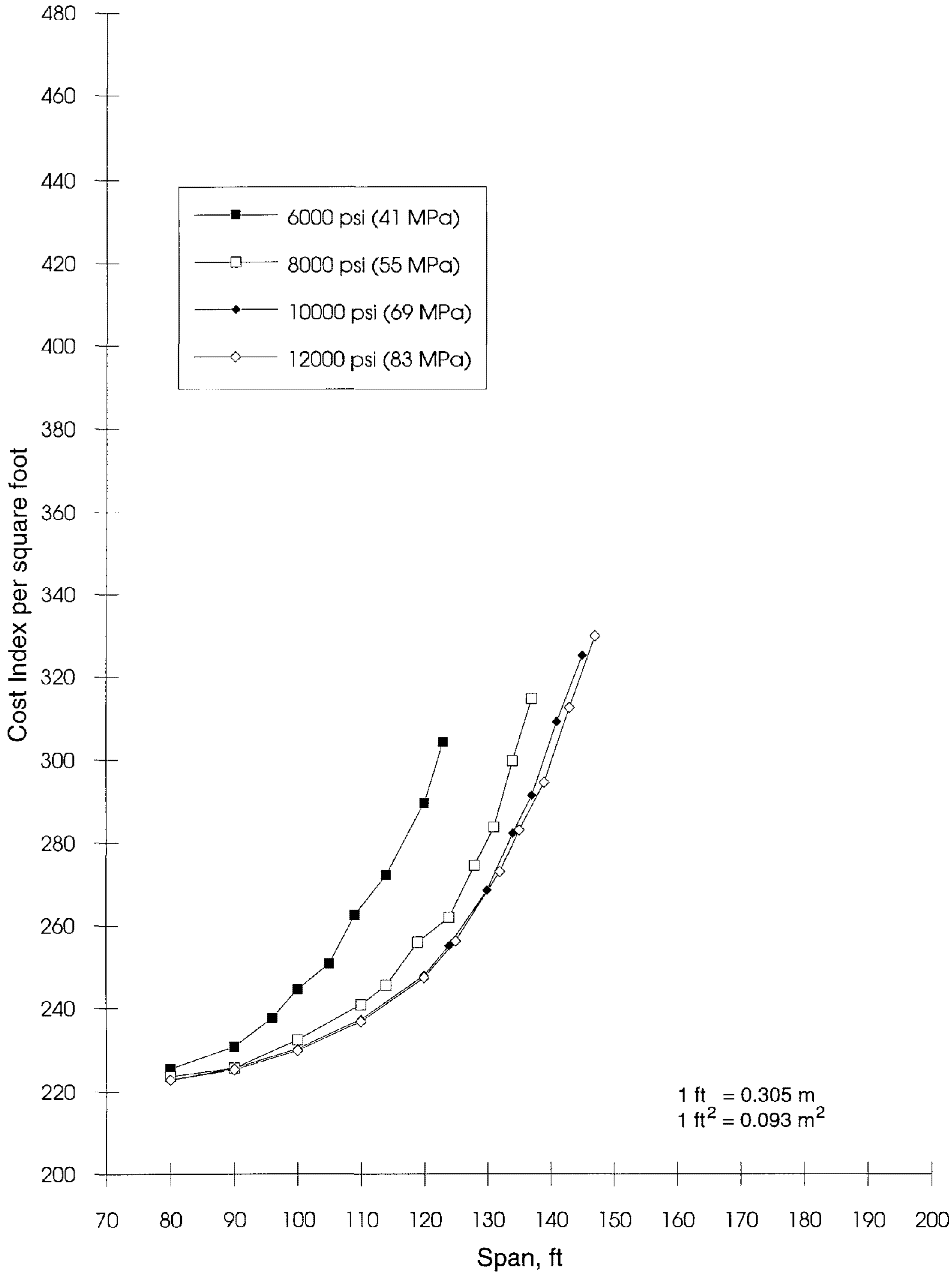


Figure 63. Cost Chart for a BT-54.

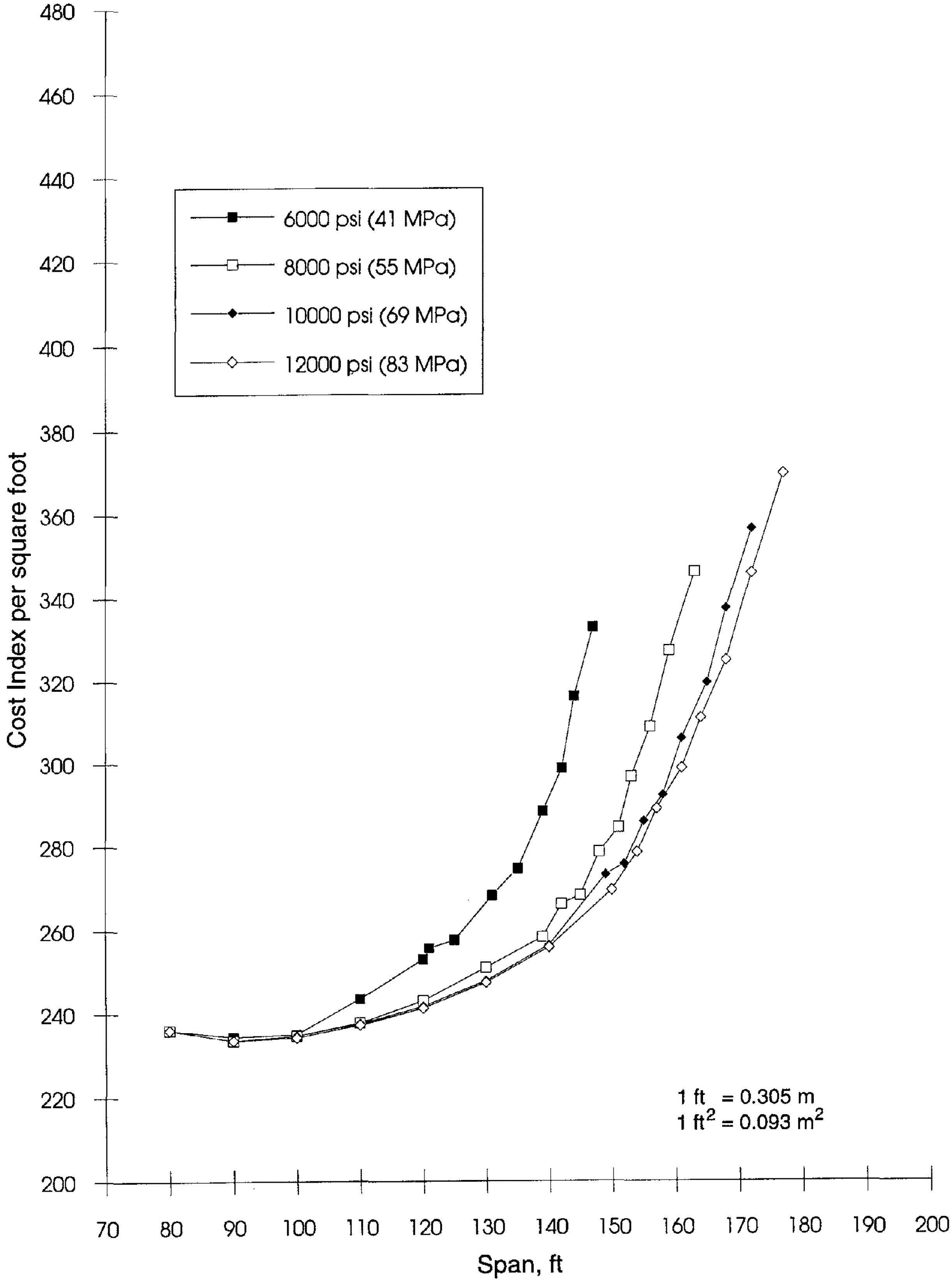


Figure 64. Cost Chart for a BT-72.

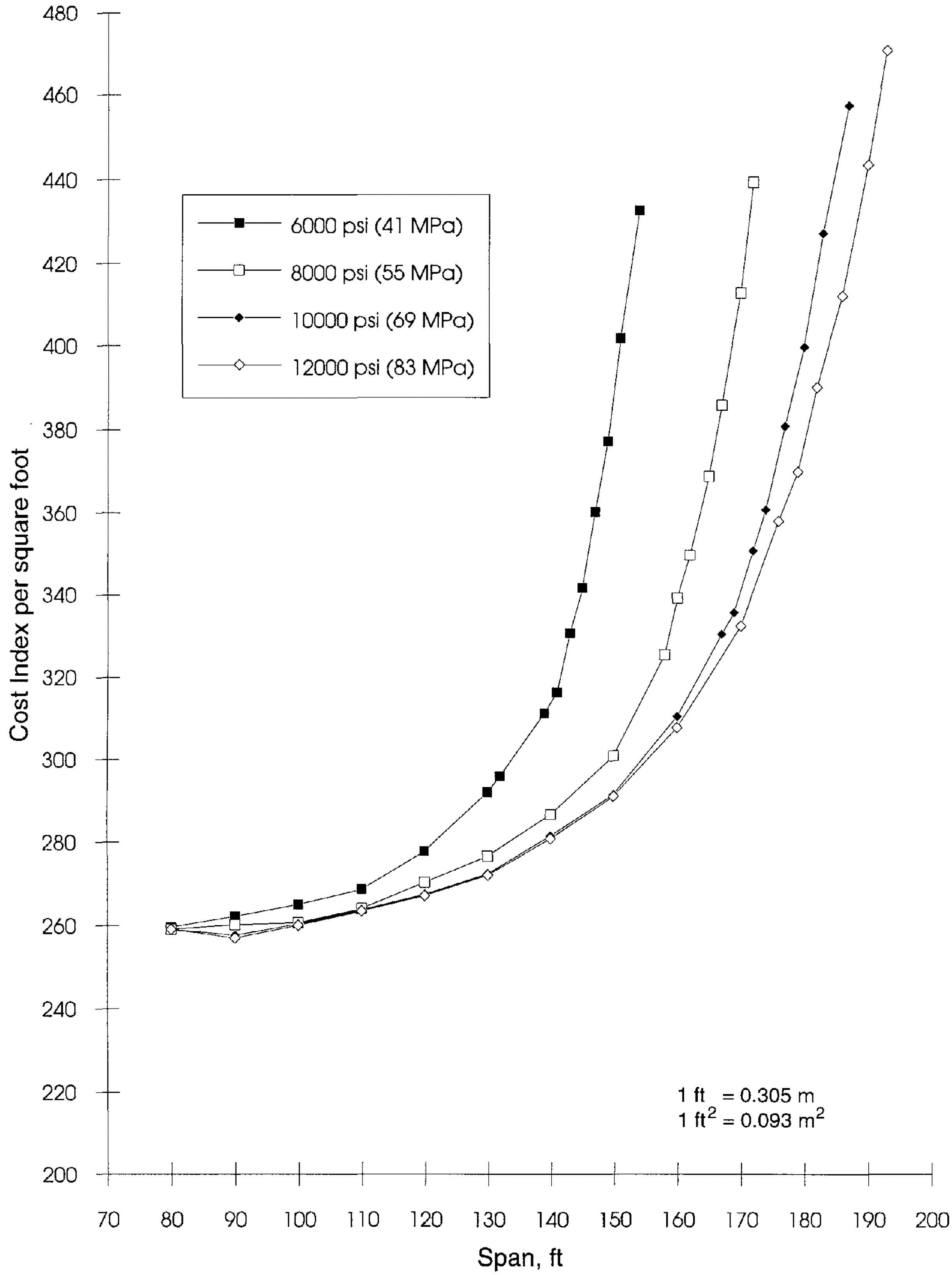


Figure 65. Cost Chart for an AASHTO Type VI.

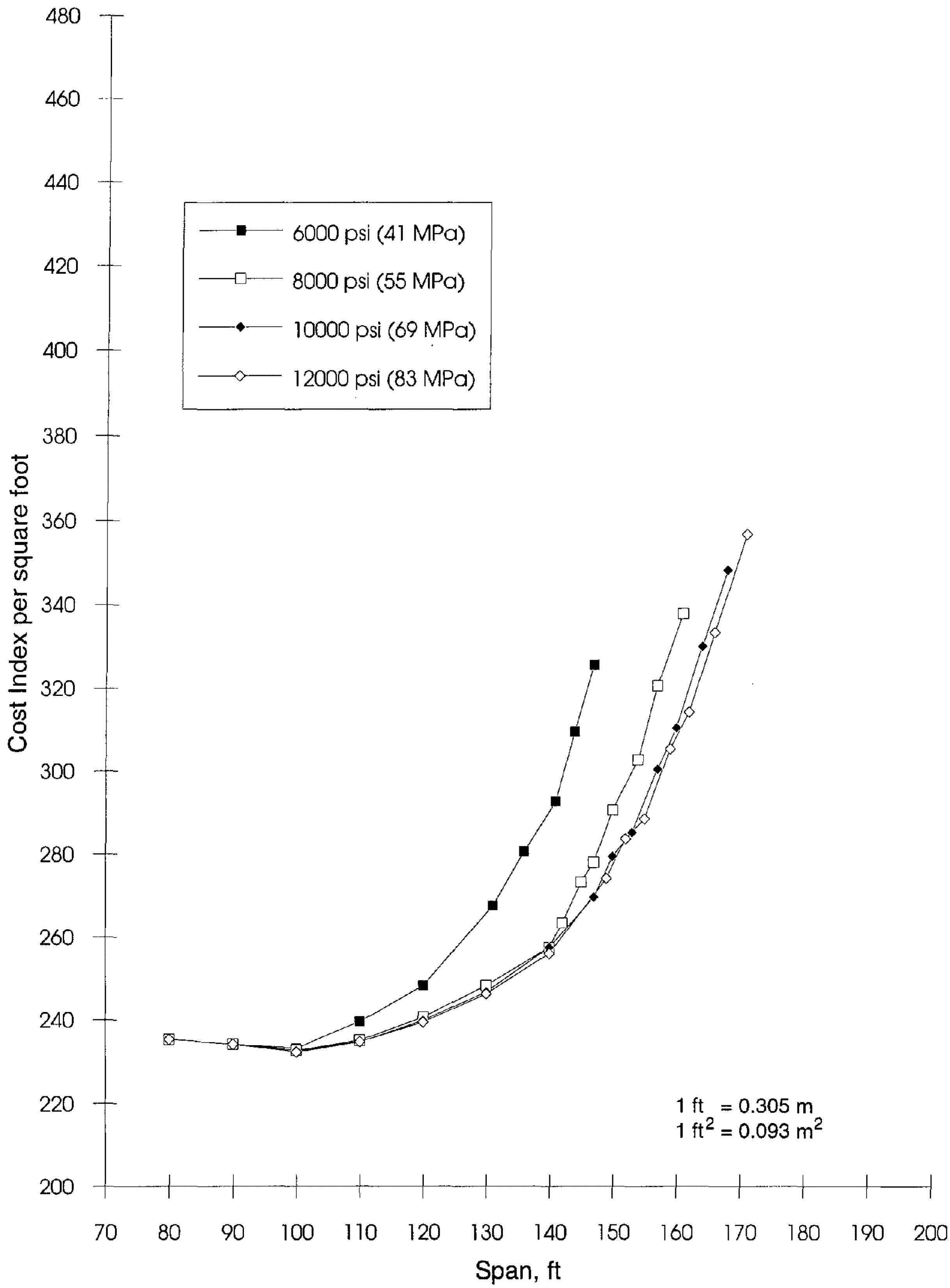


Figure 66. Cost Chart for a Washington Modified 14/6.

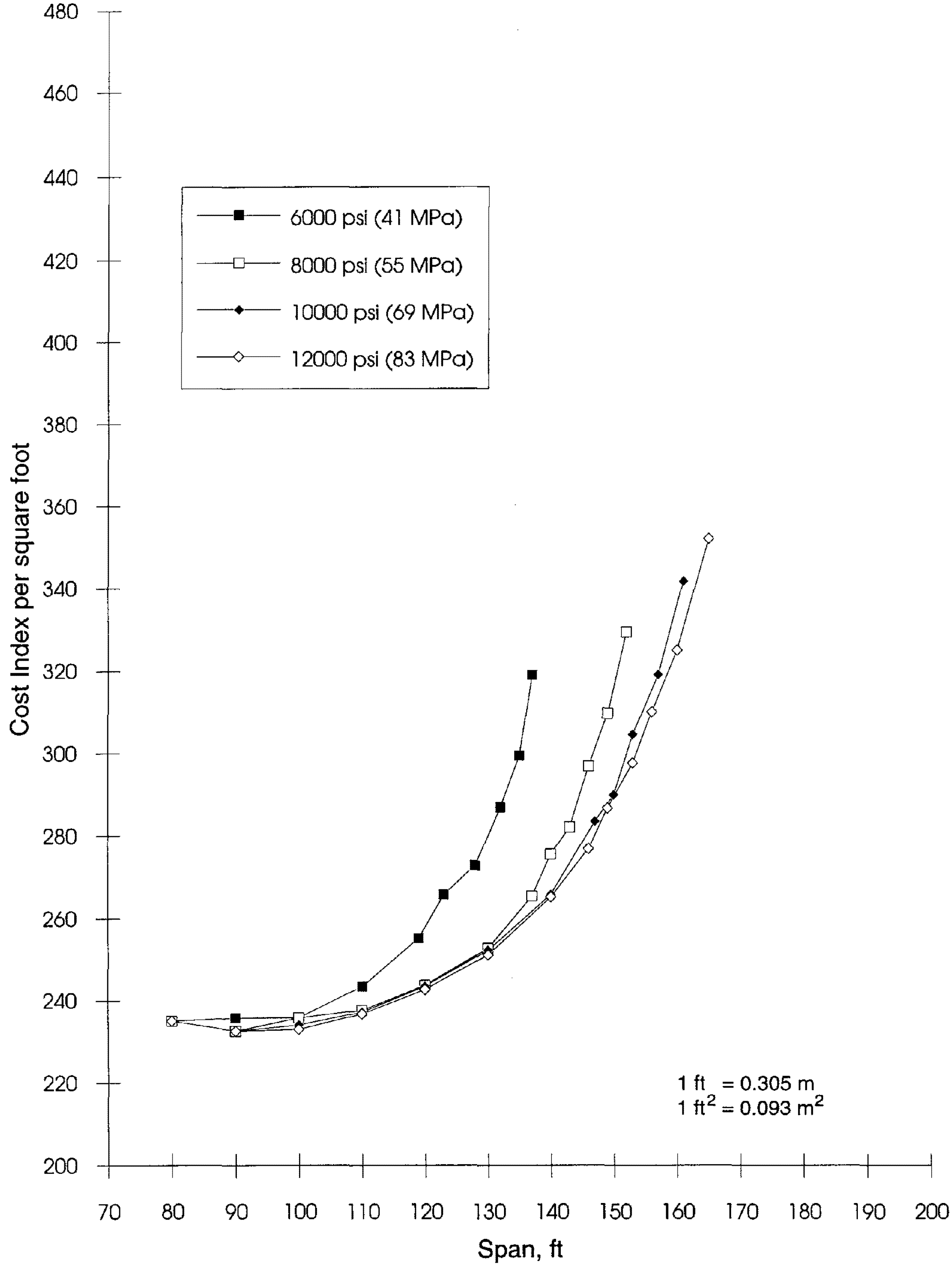


Figure 67. Cost Chart for a Colorado Modified G68/6.

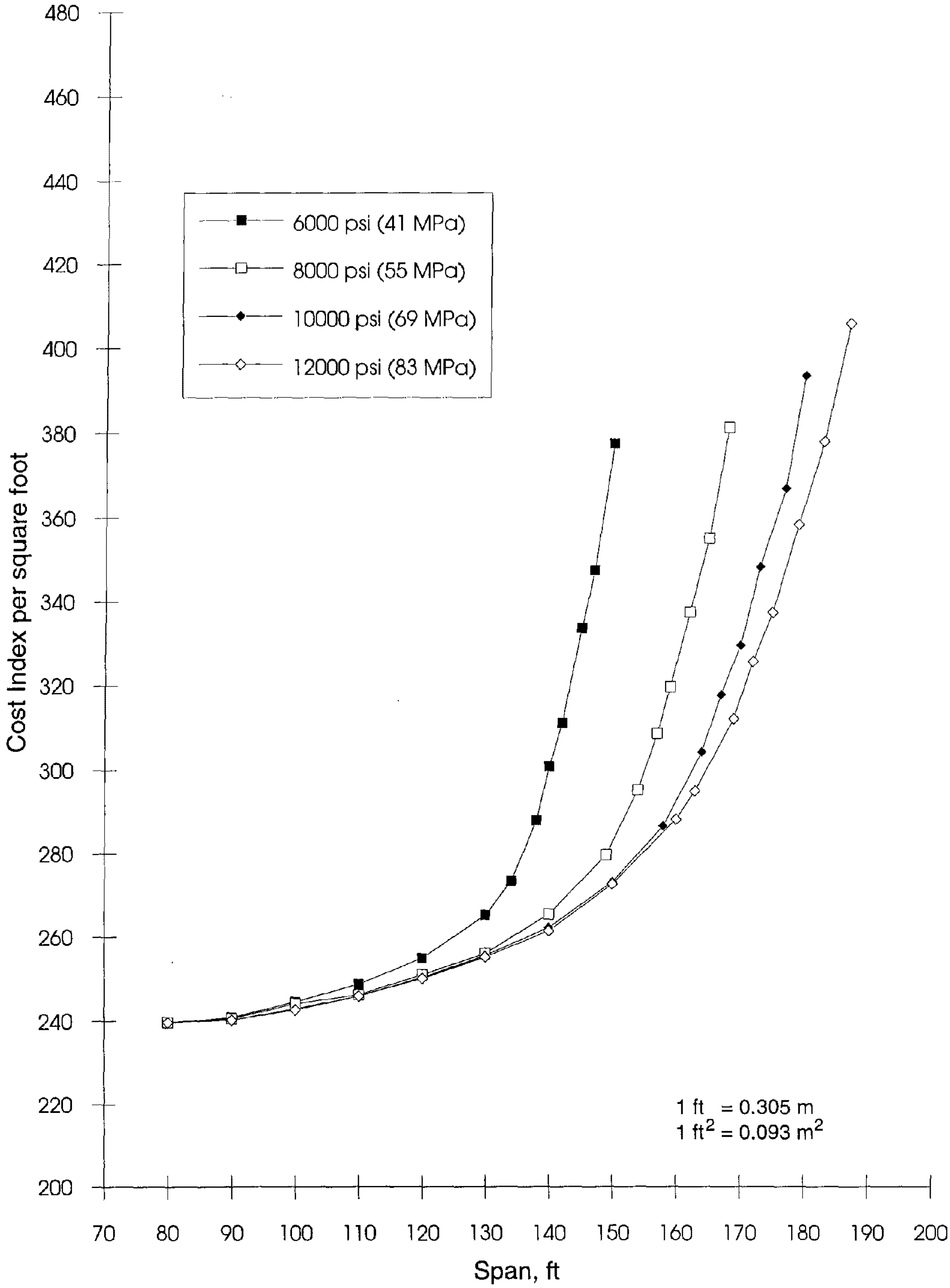


Figure 68. Cost Chart for a Nebraska 1800.

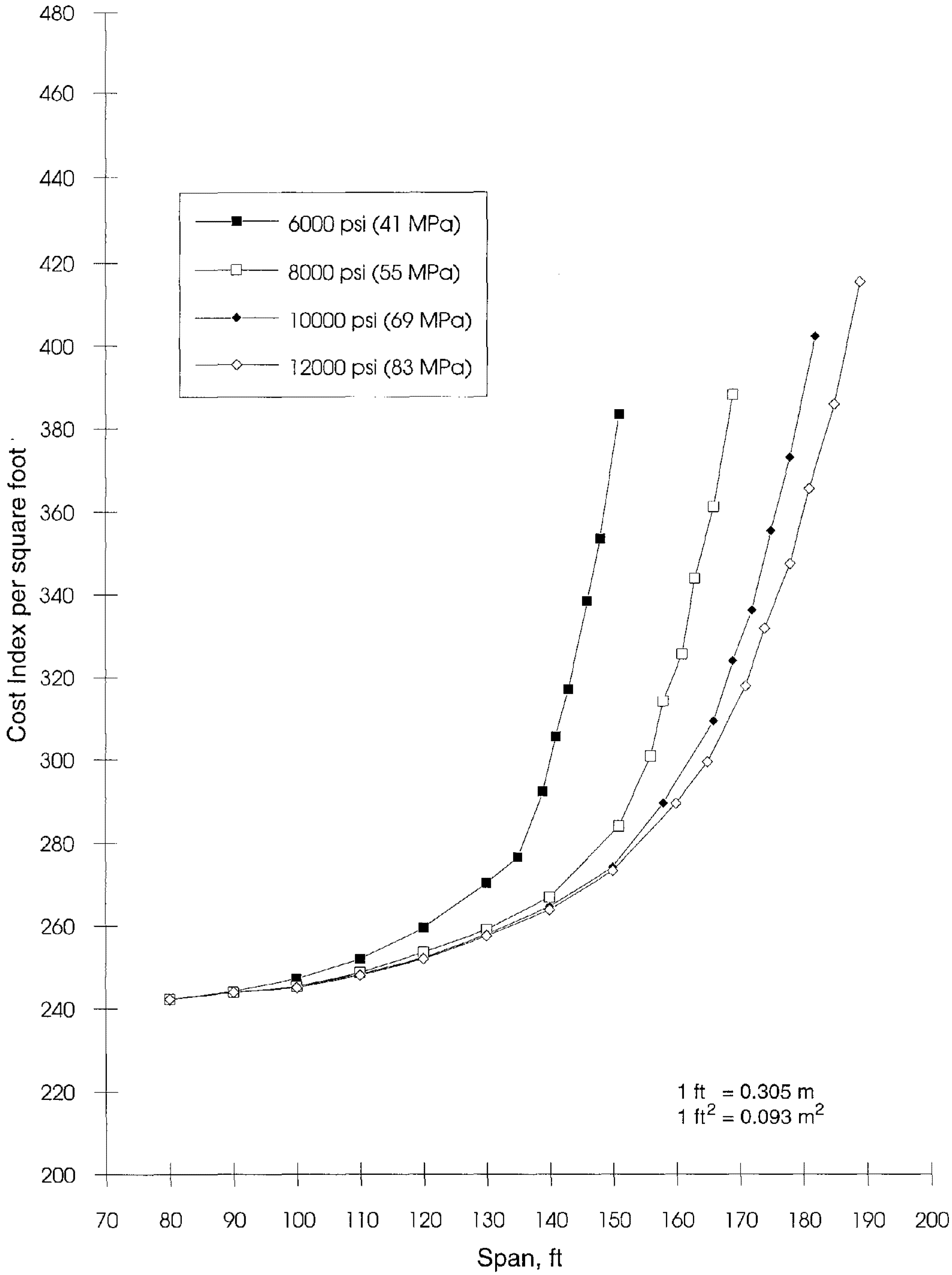


Figure 69. Cost Chart for a Florida BT-72.

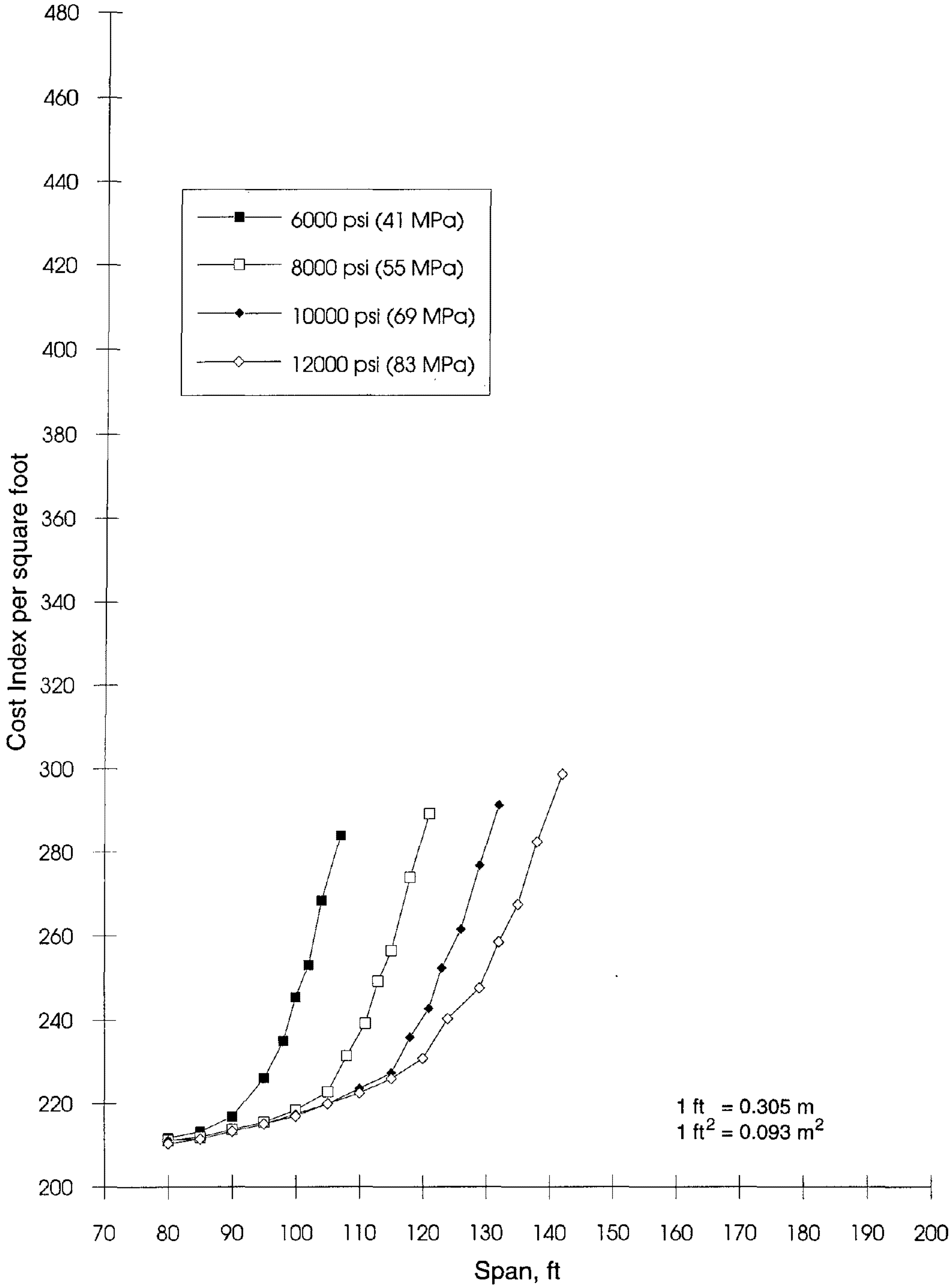


Figure 70. Cost Chart for a Texas U54B.

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