Optimized Sections for High-Strength Concrete Bridge Girders—Effect of Deck Concrete Strength

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FOREWORD

For more than 25 years, concretes with compressive strengths in excess of 41 megapascals (MPa) (6,000 pounds per square inch (psi)) have been used in the construction of columns of highrise buildings. While the availability of high-strength concretes was limited initially to a few geographic locations, opportunities to use these concretes at more locations across the United States have arisen. Although the technology to produce higher-strength concretes has developed primarily within the ready-mix concrete industry for use in buildings, the same technology can be applied in the use of concretes for bridge girders and bridge decks.

The durability of concrete bridge decks has been a concern for many years, and numerous strategies to improve the performance of bridge decks have been undertaken. Many of the factors that enable a durable concrete to be produced also result in a high-strength concrete. Consequently, if a concrete for a bridge deck is designed to be durable, it will probably also have a high compressive strength. This report contains an evaluation of the effect of high-performance concrete on the cost and structural performance of bridges constructed with high-performance concrete bridge decks and high-strength concrete girders. Several areas with the potential for improved structural performance through the use of high-performance concretes are investigated. This report should also assist designers and owners in recognizing that the use of high-performance concrete in bridges has advantages beyond those of improving durability.

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16. Abstract

This report contains an evaluation of the effect of high-performance concrete on the cost and structural performance of bridges constructed with high-performance concrete bridge decks and high-strength concrete girders. Bridge designers and owners are the main audience.

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PREFACE

For over 25 years, concretes with specified compressive strengths in excess of 41 MPa (6,000 psi) have been used in the construction of columns of highrise buildings. While the availability of the high-strength concretes was limited initially to a few geographic locations, opportunities have developed to use these concretes at more locations across the United States. As these opportunities have developed, material 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 41 MPa (6,000 psi) for bridge girders 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 41 MPa (6,000 psi). 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 for specific projects in a manner similar to the development in the building industry.

In parallel with an increased interest in the use of high-strength concretes in bridge girders, the use of high-performance concretes in bridge decks has also been receiving increased attention as a means of improving durability. High-performance concretes provide higher resistance to chloride penetration, higher resistance to deicer scaling, less damage from freezing and thawing, higher wear resistance, and less cracking. Many of the methods used to increase the durability of concrete result in a concrete that has a higher compressive strength. However, the higher concrete strength is rarely considered because the design of prestressed girders is controlled by service load stresses caused by dead load, live load, and impact.

This report contains an evaluation of the effect of high-performance concrete on the cost and structural performance of bridges constructed with high-performance concrete bridge decks and high-strength concrete girders. Several areas with the potential for improved structural performance through the use of high-performance concretes are investigated. This report should assist designers and owners in recognizing that the use of high-performance concrete in bridges has advantages beyond those of improving durability.

The research described in this report was sponsored by the Federal Highway Administration as part of their program to encourage the greater use of high-performance concretes in bridges. The program includes analytical and experimental research as well as showcase projects. The authors believe that high-performance concrete represents a technology with great potential for improving the infrastructure of the highway system.

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 yard	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 degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		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	35.314	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 degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		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 March 2003)

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CHAPTER 1. INTRODUCTION

BACKGROUND

For more than 25 years, concretes with compressive strengths in excess of 41 megapascals (MPa) (6,000 pounds per square inch (psi)) have been used in the construction of columns of highrise buildings.[1] While the availability of high-strength concretes was limited initially to a few geographic locations, opportunities to use these concretes at more locations across the United States have arisen. With the increase of such opportunities, material producers have accepted the challenge to manufacture concretes with higher compressive strengths. Although the technology to produce higher-strength concretes has developed primarily within the ready-mix concrete industry for use in buildings, the same technology can be applied in the use of concretes for bridge girders and bridge decks.

For precast, prestressed concrete bridge girders, compressive strengths in excess of 41 MPa (6,000 psi) have rarely been specified. However, strengths at release have frequently controlled the concrete mix design so that actual strengths at 28 days are often in excess of 41 MPa (6,000 psi). In recent years, a strong interest in using concrete with higher compressive strengths for bridge applications has emerged at a few geographic locations in a manner similar to the developments in the building industry. Several research studies have addressed the application of high-strength concrete in bridge girders and have identified the potential benefits of this approach (e.g., the use of fewer girders per cross section, longer span lengths, and more economical structures). (See references 2–6.)

The durability of concrete bridge decks has been a concern for many years, and numerous strategies to improve the performance of bridge decks have been undertaken. These include the use of greater cover to the reinforcing steel, the use of epoxy-coated reinforcing steel, the use of special admixtures in concrete to reduce permeability, and the use of sealers to reduce the penetration of chlorides into the concrete. Most codes and specifications now recognize that a more durable concrete can be achieved through the use of a low water-to-cementitious-material ratio, appropriate air entrainment, and appropriate cementitious materials to produce a low permeability concrete. These concretes are now becoming known as high-performance concretes where high-performance includes durability and ease of placement as well as strength. Many of the factors that enable a durable concrete to be produced also result in a high-strength concrete. Consequently, if a concrete for a bridge deck is designed to be durable, it will probably also have a high compressive strength. This research program was initiated, therefore, to investigate the cost and structural advantages of using high-performance concretes in bridge decks.

OPTIMIZED CROSS SECTIONS FOR BRIDGE GIRDERS

In the early applications of prestressed concrete, designers developed their own ideas of the "best" girder cross section to use. As a result, each bridge used a different girder shape, making it impossible to reuse girder formwork on subsequent contracts. Girder shapes were subsequently standardized in the interest of improving economy of construction which, in turn, led to the development of the standard American Association of State Highway and Transportation Officials–Prestressed Concrete Institute (AASHTO–PCI) sections for bridge girders. Girder types I through IV were developed in the late 1950s, and types V and VI in the 1960s.

Adoption of the AASHTO standard bridge girders simplified design practice and led to the 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, individual States again developed their own standard sections for improved efficiency and economy. In l980, the Federal Highway Administration (FHWA) initiated an investigation to identify new optimized sections for major prestressed concrete girders.

In an FHWA study published in 1982, Construction Technology Laboratories, Inc. identified the Bulb-Tee, Washington, and Colorado girders as the most structurally efficient sections.[7,8] A cost effectiveness analysis recommended the use of the Bulb-Tee girder (with a 152-millimeter (mm) (6-inch) web) as a national standard for precast, prestressed concrete bridge girders in the United States for span lengths ranging from 24 to 43 meters (m) (80 to 140 feet (ft)).

Subsequently, the PCI Committee on Concrete Bridges developed a modified section for use as a national standard.[9] The modifications resulted in a slightly heavier section that was easier to produce and handle. This cross section was later adopted by several States and is identified as the Bulb-Tee (BT-72) in this report. Several other versions of the Bulb-Tee have also been developed in different geographic locations.[10,11]

A recently completed report for the FHWA entitled *Optimized Sections for High-Strength Concrete Bridge Girders* concluded that the use of existing girder cross sections with concrete compressive strengths up to 69 MPa (10,000 psi) will allow longer span lengths and more economical structures.[6] In order for concrete with compressive strengths in excess of 69 MPa (10,000 psi) to be used effectively, additional prestressing forces must be applied to the cross section. Report conclusions were based on analyses performed using the computer program BRIDGE which determines relative unit costs and maximum span lengths for different prestressed concrete bridge designs. All analyses were based on the assumption that concrete in the deck had a compressive strength of 28 MPa (4,000 psi) and that the prestress losses have a constant value of 310 MPa (45,000 psi). However, these assumptions may not reflect true behavior and current trends in the usage of high-performance concretes.

HIGH-PERFORMANCE CONCRETE IN BRIDGE DECKS

The American Concrete Institute (ACI) has defined high-performance concrete as concrete meeting special performance and uniformity requirements that cannot always be achieved routinely using only conventional constituents and normal mixing, placing, and curing practices.[12] These requirements may involve enhancements of the following:

- Ease of placement and compaction without segregation.
- Long-term mechanical properties.
- Early age strength.
- Toughness.
- Volume stability.
- Long life in severe environments.

For bridge decks, high-performance concrete needs to have enhanced long-term mechanical properties, enhanced toughness, and long life in severe environments. Volume stability is also desirable. From a construction standpoint, ease of placement and compaction without segregation is also essential. Therefore, a concrete to be used for a durable and long-lasting bridge deck needs to meet all the requirements of a high-performance concrete as defined by ACI.

In the Strategic Highway Research Program (SHRP) C-205, high-performance concrete for pavements and bridges was defined as concrete with the following characteristics:

- A maximum water–cementitious material ratio of 0.35.
- A minimum durability factor of 80 percent as determined by ASTM C666 Method A.
- A minimum strength criteria of:
    1. 21 MPa (3,000 psi) within 4 hours of placement.
    2. 34 MPa (5,000 psi) within 24 hours.
    3. 69 MPa (10,000 psi) within 28 days.[13]

For bridge decks, items 1 and 2, as defined by SHRP, are clearly essential and desirable for long-term durability performance. However, the use of a water-to-cementitious-material ratio of 0.35 will result in a concrete compressive strength at 28 days well in excess of the 28 MPa (4,000 psi) that is often specified for today's bridge decks and could well be in the range of 41 to 55 MPa (6,000 to 8,000 psi).

The use of high-performance concretes in bridge decks is receiving an increased amount of attention as a means of improving durability. High-performance concretes provide higher resistance to chloride penetration, higher resistance to deicer scaling, less damage from freezing and thawing, higher wear resistance, and less cracking. Many of the methods used to increase the durability of concrete result in a concrete that has a higher compressive strength. However, the higher concrete strength is rarely considered because the design of long-span prestressed girders is controlled by service load stresses caused by dead load, live load, and impact.

In a recent project for the Louisiana Transportation Research Center, four full-size, prestressed concrete girders were tested to destruction in flexure.[14] While the specified strength of concrete in the girders was 69 MPa (10,000 psi), the bridge design for these girders required concrete with a compressive strength of only 29 MPa (4,200 psi) in the deck. However, analyses of the cross section indicated that failure in flexure would occur by crushing of the deck concrete. Consequently, a concrete compressive strength of 41 MPa (6,000 psi) was specified for the deck to ensure flexural failure by fracture of the strands. Although the flexural strength of the section was reached when the strands fractured, evaluation of the test results indicated that the section was close to failure by crushing the concrete deck even when 41 MPa (6,000 psi) concrete was used in the deck.

It was noted in a previous report that as girder concrete strength increased, a point of diminishing benefits was reached.[6] The primary cause of these diminishing returns is decreasing strand eccentricity. Once strands have to be placed within the web, the efficiency of additional strands decreases rapidly. Finally, a point is reached where no more space is available for additional strands. The only benefit, therefore, is an increase in the concrete tensile strength. Another factor contributing to the reduced benefits was the deck concrete strength. In calculating the composite section properties, transformed girder–deck section was used. As girder strength increased and deck strength remained constant, the composite section properties decreased with a corresponding increase in service load stresses in the girder for the same span length and girder spacing. The use of a high-performance concrete in the deck with a higher modulus of elasticity will result in an increase in the composite section properties.

EFFECT OF HIGH-STRENGTH CONCRETE ON PRESTRESS LOSSES

When compared with the properties of conventional strength concretes, high-strength concrete has a higher modulus of elasticity, a higher tensile strength, and reduced creep. The higher modulus of elasticity results in less elastic shortening in prestressed concrete girders at time of release for the same stress level. This reduction in shortening may be offset by the use of higher prestress levels with high-strength concrete. The higher tensile strength does not have a direct effect on prestress losses but allows high-strength girders to be designed for a higher permissible tensile stress. This, in turn, results in higher service load design moments.

Creep per unit stress of high-strength concrete is lower than the creep for conventional strength concretes.[13] Thus, the direct substitution of a high-strength concrete in place of a lower strength concrete will result in less prestress losses. However, the utilization of a higher level of prestress will offset the reduction in creep per unit stress. The magnitude of the net result will depend on the reduction in creep per unit stress and the increase in stress level.

Prior to casting the deck, prestress losses depend on the properties of the girder concrete alone. After the deck is cast, prestress losses depend on the properties of the deck concrete as well as the girder concrete. When the deck concrete has a strength significantly lower than the strength of the girder concrete, the deck concrete may have a major influence on the magnitude of prestress losses in the bridge. Consequently, the utilization of a higher-strength concrete in the bridge deck can be beneficial in reducing prestress losses. A reduction in prestress losses means that for the same amount of initial prestress, a greater force is available for design at service load. Since the amount of force available at service load controls the design of long-span girders, reduced prestress losses will be beneficial in the more effective utilization of high-strength concrete.

Another factor related to elastic shortening, creep, and prestress losses is the change in camber. Girders produced with high-strength concrete are likely to have less initial camber at release and less change in camber. Bridges produced with high-performance concretes in the decks and girders may undergo less deflection changes after the deck is cast.

OBJECTIVES AND SCOPE

Based on the above background, the objectives of the research were to evaluate the following:

- Effect of using high-performance concretes in the deck on the cost per unit area.
- Effect of using high-performance concretes in the deck and girders on flexural strength and ductility.
- Effect of high-performance concretes in the deck and girders on prestress losses and long-term deflections.

The objectives were accomplished in three separate tasks. Each task used a different research approach as described in the following chapters.

For each task, the analyses were based on a PCI Bulb-Tee (BT-72) cross section with a depth of 1.83 m (72 inches). In the previous investigation, the PCI Bulb-Tee was identified as the most cost-effective cross section for span lengths up to 45.7 m (150 ft) at all concrete compressive strength levels.(6) For span lengths greater than 45.7 m (150 ft) and for all concrete compressive strength levels, the Florida BT-72 and Nebraska NU-1800 were the most cost effective. In the present investigation, some analyses were also performed using the FL BT-72. The cross sectional dimensions of the BT-72 and FL BT-72 are shown in figures 1 and 2, respectively.

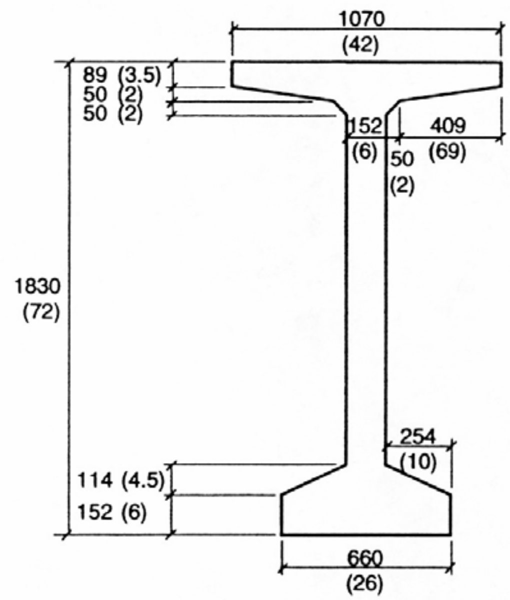

Figure 1. Cross section of girder analyzed – PCI Bulb-Tee (BT-72).
All dimensions are in millimeters (inches).

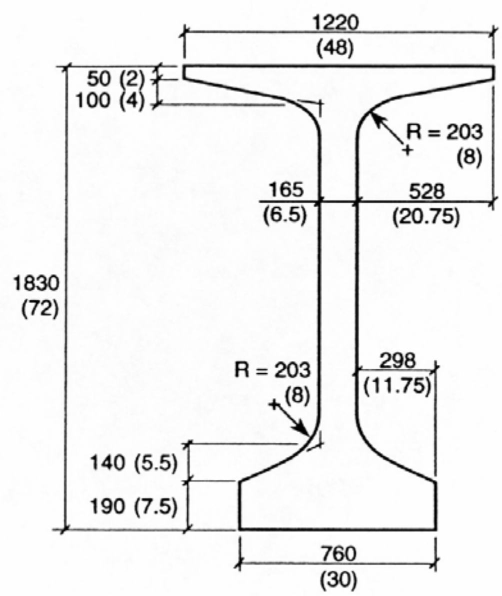

Figure 2. Cross section of girder analyzed – Florida Bulb-Tee (FL BT-72).
All dimensions are in millimeters (inches).

CHAPTER 2. TASK 1: COST ANALYSES OF HIGH-PERFORMANCE CONCRETE IN BRIDGE DECKS

RESEARCH APPROACH

Analyses to evaluate the effect of using high performance concrete in bridge decks on the cost per unit area were performed using a computer program called BRIDGE.

Computer Program BRIDGE

The computer program BRIDGE was written as part of a previous investigation for the Optimized Sections for Precast, Prestressed Bridge Girders and was later revised.[6,7] 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, the required number of prestressing strands, and the cost index per unit surface area of bridge deck. It also provides section properties, moments, stress levels, and deflections. Comparisons are made based on relative costs for simply supported spans. A complete description of BRIDGE is given in reference 6.

The following variables were used in the analyses using BRIDGE:

- Girder concrete strength: Concrete strength of the girders at 28 days was varied from 41 MPa (6,000 psi) upward in increments of 14 MPa (2,000 psi) to 83 MPa (12,000 psi); release strength was taken as 75 percent of the 28-day strength.
- Deck concrete strength: Strength of the deck concrete was varied from 28 MPa (4,000 psi) upwards in increments of 14 MPa (2,000 psi) to 69 MPa (10,000 psi).
- Unit weights of concrete were taken as 2.32, 2.37, 2.42, 2.48, and 2.5 megagrams per cubic meter (Mg/m^3) (145, 148, 151, 155 and 156 lb/ft^3, respectively) for concrete compressive strengths of 28, 41, 55, 69 and 83 MPa (4,000, 6,000, 8,000, 10,000, and 12,000 psi, respectively).
- Girder cross section: Analyses were made for the BT-72 and the FL BT-72.
- Span length: Spans in excess of 24.4 m (80 ft) up to a maximum possible span length were considered.
- Girder spacing: No maximum spacing was placed on the girders; effective deck span was not allowed to exceed 4.9 m (16 ft); minimum spacing corresponded to the flanges of the two girders touching each other.
- Deck thickness: Deck thickness varied with girder spacing according to a predetermined design; minimum deck thickness was 190 mm (7.5 inches).
- Relative unit costs of materials: Two sets of relative unit costs as discussed later were used.

The complete combination of variables is defined in tables 1 and 2. It is recognized that the use of a deck concrete strength of 69 MPa (10,000 psi) in combination with a girder concrete strength of 41 MPa (6,000 psi) is unlikely to occur. However, the complete combination of variables has been included for comparison purposes.

Table 1. Task 1 variables (SI units).

Girder Section	Girder Strength (MPa)	Deck Strength (MPa)	Girder Concrete Premium	Deck Concrete Premium
BT-72	41	28, 41, 55, 69	No	No
BT-72	55	28, 41, 55, 69	No	No
BT-72	69	28, 41, 55, 69	No	No
BT-72	83	28, 41, 55, 69	No	No
BT-72	41	28, 41, 55, 69	No	Yes
BT-72	55	28, 41, 55, 69	Yes	Yes
BT-72	69	28, 41, 55, 69	Yes	Yes
BT-72	83	28, 41, 55, 69	Yes	Yes
FL BT-72	41	28, 41, 55, 69	No	No
FL BT-72	83	28, 41, 55, 69	No	No
FL BT-72	41	28, 41, 55, 69	No	Yes
FL BT-72	83	28, 41, 55, 69	Yes	Yes

Table 2. Task 1 variables (English units).

Girder Section	Girder Strength (psi)	Deck Strength (ksi)	Girder Concrete Premium	Deck Concrete Premium
BT-72	6,000	4, 6, 8, 10	No	No
BT-72	8,000	4, 6, 8, 10	No	No
BT-72	10,000	4, 6, 8, 10	No	No
BT-72	12,000	4, 6, 8, 10	No	No
BT-72	6,000	4, 6, 8, 10	No	Yes
BT-72	8,000	4, 6, 8, 10	Yes	Yes
BT-72	10,000	4, 6, 8, 10	Yes	Yes
BT-72	12,000	4, 6, 8, 10	Yes	Yes
FL BT-72	6,000	4, 6, 8, 10	No	No
FL BT-72	12,000	4, 6, 8, 10	No	No
FL BT-72	6,000	4, 6, 8, 10	No	Yes
FL BT-72	12,000	4, 6, 8, 10	Yes	Yes

The following default assumptions in BRIDGE were used:

- Design conforms to AASHTO specifications.
- Live load consists of HS 20-44 loading.
- Girders are simply supported.
- Design is based on a typical interior girder.
- 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.
- Strands are Grade 270 with a 12.7 mm (0.5 inch) diameter and have an idealized, trilinear stress–strain curve.
- Strands are spaced at 51-mm (2-inch) centers (minimum spacing of 51-mm (2-inches) from concrete surface to center of the strand).
- Total prestress losses are constant and equal 310 MPa (45,000 psi). Separate analyses for prestress losses are discussed in chapter 4 of this report.
- Relative unit costs of materials and labor are constant for each cost analysis.
- Cost analysis comparisons are for the precast girder and a cast-in-place deck only. Costs of substructure and approach fills are not considered.
- Design is based on flexural strength at midspan. It is assumed that the compressive and tensile stressed 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 by debonding some strands at the ends of the girders.

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.

The computer program BRIDGE determines cast-in-place deck thickness and reinforcement from design aids prepared by the Washington State Department of Transportation.(15) Designs aids were based on AASHTO and ACI guidelines.(16,17) For effective slab spans less than 2.1 m (7 ft), the design aids use a slab thickness of 175 mm (7 inches). However, for this project the FHWA requested that the minimum slab thickness be increased to 190 mm (7½ inches). Slab reinforcement was then determined based on the new minimum slab thickness. Results of the design are shown in table 3 with the SI equivalents provided in table 4. These data as English units were stored within BRIDGE.

Table 3. Deck design[15] (English units).

Effective Slab Span (ft)	Slab Thickness (inches)	Slab Reinforcement*	
		Bar Size	Spacing (inches)
1 to 3 inclusive	7 ½	No. 5	11.0
3 to 4 inclusive	7 ½	No. 5	9.5
4 to 5 inclusive	7 ½	No. 5	8.5
5 to 6 inclusive	7 ½	No. 5	7.0
6 to 7 inclusive	7 ½	No. 6	9.0
7 to 8 inclusive	7 ½	No. 6	8.0
8 to 9 inclusive	8	No. 6	8.0
9 to 10 inclusive	8 ½	No. 6	8.0
10 to 11 inclusive	8 ¾	No. 6	7.5
11 to 12 inclusive	9	No. 7	10.0
12 to 13 inclusive	9 ½	No. 7	10.0
13 to 14 inclusive	9 ¾	No. 7	9.5
14 to 15 inclusive	10	No. 7	9.5
15 to 16 inclusive	10 ½	No. 7	9.5

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

Table 4. Deck design[15] (SI units).

Effective Slab Span (m)	Slab Thickness (mm)	Slab Reinforcement*	
		Bar Size	Spacing (mm)
0.3 to 0.9 inclusive	190	15 M	280
0.9 to 1.2 inclusive	190	15 M	240
1.2 to 1.5 inclusive	190	15 M	215
1.5 to 1.8 inclusive	190	15 M	180
1.8 to 2.1 inclusive	190	20 M	230
2.1 to 2.4 inclusive	190	20 M	205
2.4 to 2.7 inclusive	200	20 M	215
2.7 to 3.0 inclusive	215	20 M	215
3.0 to 3.4 inclusive	225	20 M	200
3.4 to 3.7 inclusive	230	20 M	195
3.7 to 4.0 inclusive	240	20 M	195
4.0 to 4.3 inclusive	250	20 M	190
4.3 to 4.6 inclusive	255	20 M	190
4.6 to 4.9 inclusive	265	20 M	190

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

Relative Costs

The material weight for girder concrete, deck concrete, strands, reinforcing steel, and epoxy-coated reinforcing steel is also calculated by BRIDGE. The relative cost of materials is then determined as the product of material weight and relative unit cost. The summation of the relative cost of materials is then divided by deck area to give the cost index per unit area. In the previous investigation, analyses were made to determine the effect of the premium cost for higher-strength girder concretes on the cost per unit area.[6] Based on a limited survey of industry, the ratios in table 5 were assumed for the premium cost for higher-strength concrete used in the girders.

Table 5. Ratios for high-strength concrete.

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

For the current investigation, the premium costs for higher-strength concrete in the girders were assumed to be those shown as intermediate ratio.

In the previous investigation, the deck concrete strength was assumed to be 28 MPa (4,000 psi) and the relative cost of the deck concrete was the same as that of the 41-MPa (6,000-psi) girder concrete.[(6)]

To investigate the costs of using high-strength concrete in bridge decks, it was necessary to determine the relative in-place costs for different strength concretes. Analyses of the data in the previous report indicated that the cost based on materials alone for a 69-MPa (10,000-psi) concrete would be about 72 percent higher than that of a 28-MPa (4,000-psi) concrete.[(6)] 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 higher-strength concrete in place will be lower than these numbers. In-place costs for high-strength concrete have been published by ACI Committee 363 and these data are shown in table 6.[(1)]

Table 6. In-place costs as per ACI Committee 363.

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

Based on these data, it was decided to use two sets of ratios for premium costs of high-strength concrete in the decks to determine the sensitivity of the cost index per unit area to the premium costs. The selected ratios are in table 7.

Table 7. Selected ratios for cost index per unit area to premium costs.

Deck Concrete Strength	No Premium Ratio*	Premium Ratio*
28 MPa (4,000 psi)	1.00	1.00
41 MPa (6,000 psi)	1.00	1.05
55 MPa (8,000 psi)	1.00	1.13
69 MPa (10,000 psi)	1.00	1.25

* Costs are relative to 41-MPa (6,000-psi) girder concrete.

Analyses were performed for both the no premium ratio and the premium ratio.

EFFECTS OF CONCRETE STRENGTH ONLY

The computer program BRIDGE determines the deck thickness and reinforcement based on a design aid prepared by the Washington State Department of Transportation.[15] The design aid is based on AASHTO and ACI guidelines and uses a concrete compressive strength at 28 days of 28 MPa (4,000 psi).[16,17] The design aid is based on the distribution of loads and design of concrete slabs given in the AASHTO Standard Specifications for main reinforcement perpendicular to traffic.[16] Discussions with several bridge engineers indicated that many States use design aids similar to the one incorporated into BRIDGE.

In the AASHTO design of concrete slabs, deck thickness and reinforcement are calculated to resist the bending moments caused by the dead load of the slab and the live load moment for the selected concentrated wheel load. In flexural design for under-reinforced sections, the concrete compressive strength has little effect on the flexural strength of the section. Consequently, the use of high-strength concrete in the deck will not impact the flexural strength of the deck. Based on this design approach, the deck thickness and the amount of reinforcement required to resist a given bending moment will not decrease as higher-strength concretes are used. Therefore, no revision was made to the design aid for use with higher-strength concretes in the decks. In addition, a minimum slab thickness of 190 mm (7.5 inches) was requested by FHWA. This controlled the deck thickness for effective span lengths up to 2.4 m (8 ft). Thus the use of higher-strength concrete in bridge decks will not reduce the thickness of the deck or the weight of the superstructure. In fact, the higher-strength concretes in the deck will slightly increase the dead load of the superstructure because of the higher density of the higher-strength concretes.

The computer program BRIDGE was used to perform cost efficiency analyses for various strengths of concrete in the girders and bridge decks. A sample cost chart for a BT-72 is shown in figure 3. The figure shows the cost index per unit surface area of the bridge deck versus span length for various girder spacings. The "optimum cost curve" is obtained where the end points of each individual cost curve are joined as shown by the dashed line in figure 3. This optimum cost curve indicates the least cost index for a particular span and varies as a function of girder spacing (see figure 3). As reported in previous investigations, for a given span, the cost index per unit area of a bridge deck decreases as girder spacing increases.[6,7] In all optimum cost curves used in this report, the girder spacing is allowed to vary to obtain the optimum cost.

Optimum cost curves are generated for a constant girder and deck concrete strength. The cost chart in figure 3 is for girder concrete strength at 28 days of 41 MPa (6,000 psi) and a 28-day concrete strength for the deck of 28 MPa (4,000 psi). Additional optimum cost curves can be generated for other deck concrete strengths for the same girder cross section and the same girder concrete strength. Figure 4 is a plot of the optimum cost curves for a BT-72 with a girder concrete strength of 41 MPa (6,000 psi) and deck concrete strengths of 28, 41, 55, and 69 MPa (4,000, 6,000, 8,000, and 10,000 psi, respectively). The data in figure 4 are based on the assumption that there is no cost premium for the higher-strength concrete. This figure illustrates that, at the shorter span lengths, there are no cost advantages or disadvantages in using the higher-strength concrete in the bridge deck. However, at the longer span lengths a slight advantage is achieved in a reduction in cost for the same span length. Alternatively, the ability to achieve a slightly longer span length is obtained. For the same girder cross section, these advantages occur because of the higher moment of inertia of the composite cross section that is achieved with the higher-strength concretes in the deck. In addition, the increase in the moment of inertia of the composite section will result in less live load deflection.

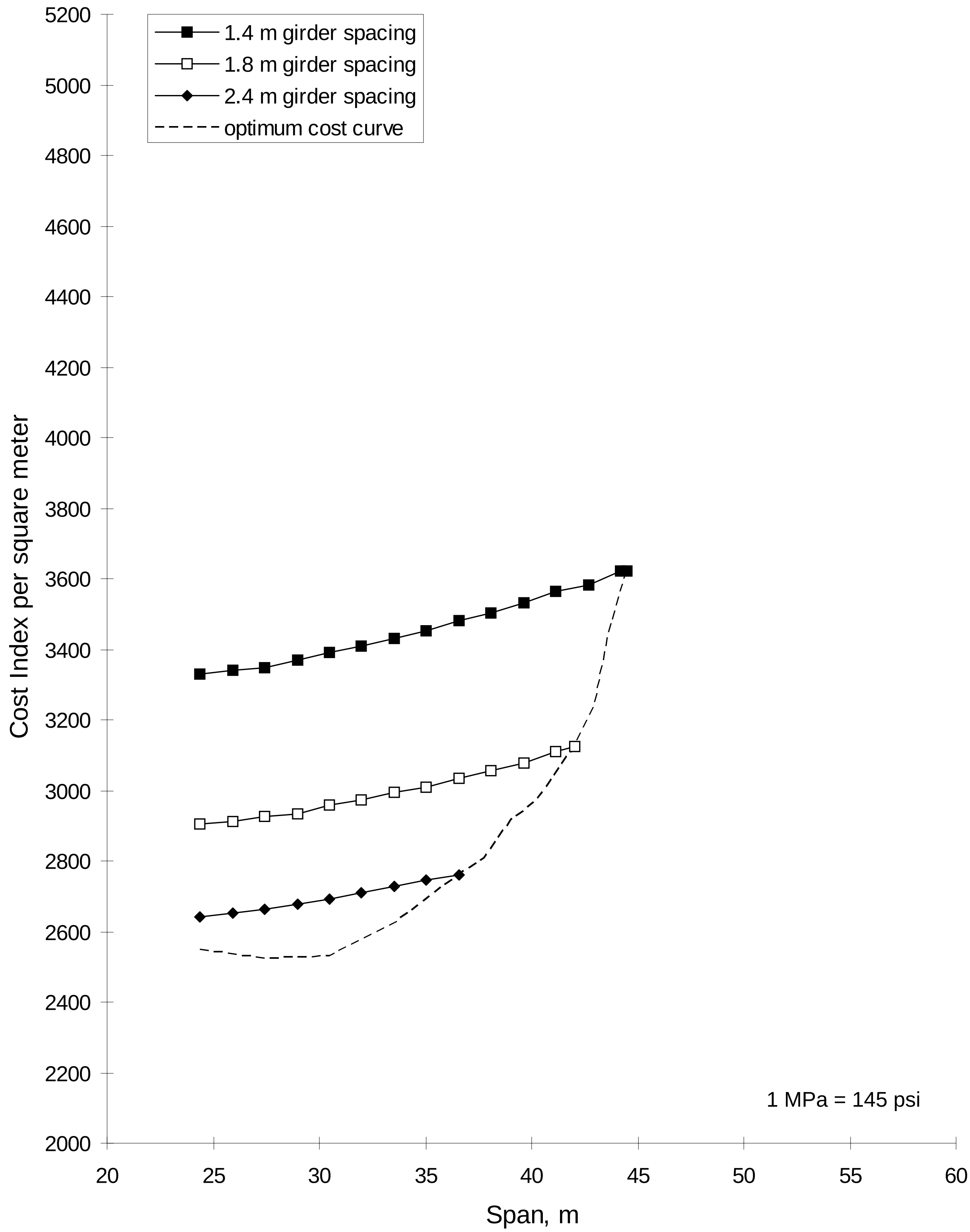

Figure 3. Cost chart for a BT-72, 41 MPa.

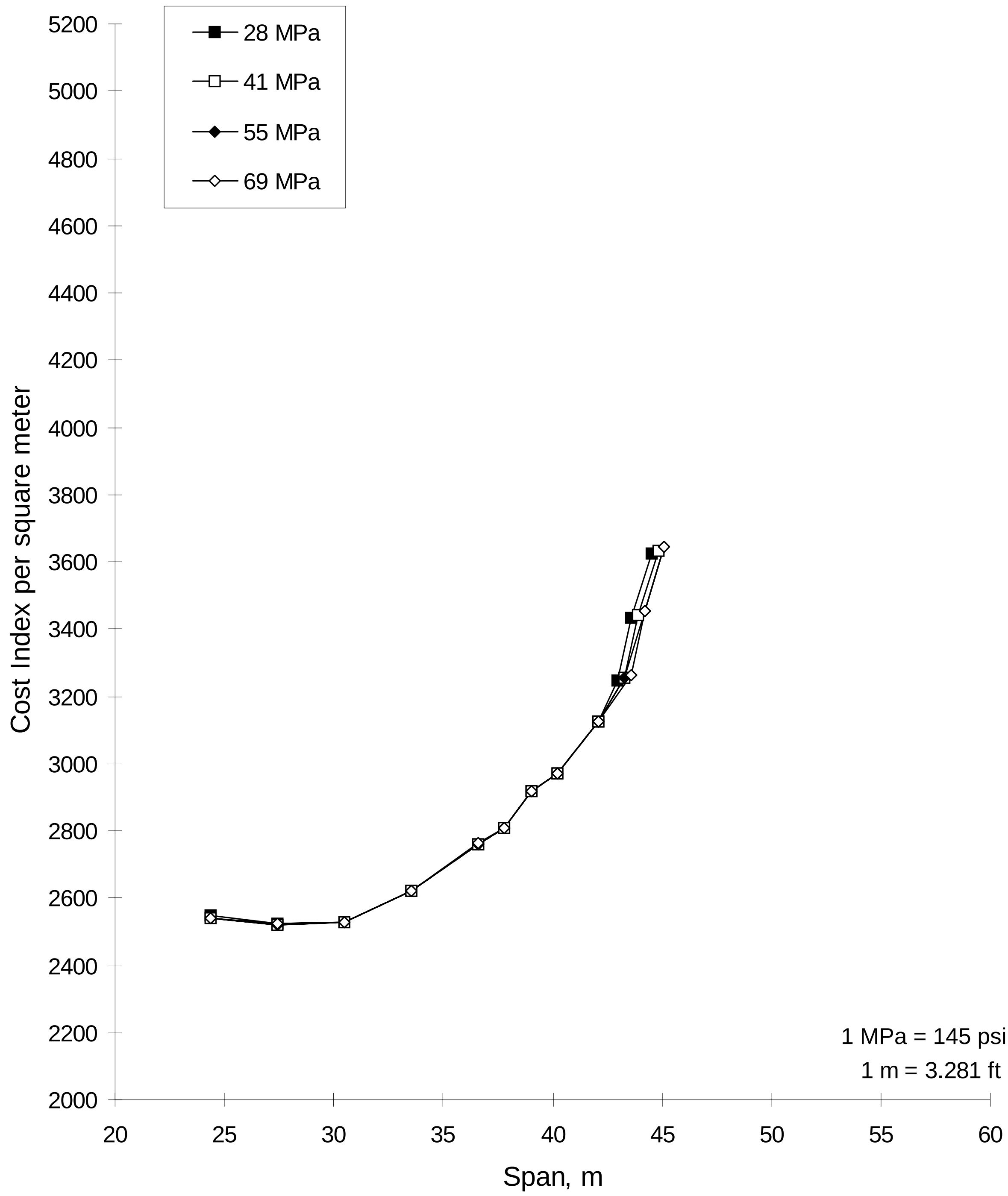

Figure 4. Optimum cost curves for a BT-72, 41 MPa.

A comparison of optimum cost curves for a BT-72 girder with a girder concrete strength of 83 MPa (12,000 psi) and deck concrete strengths of 28, 41, 55, and 69 MPa (4,000, 6,000, 8,000, and 10,000 psi, respectively) and no cost premium is shown in figure 5. In this combination, there are no real cost advantages or disadvantages with the use of the higher-strength concrete in the deck. Similar data for a BT-72 girder with concrete strengths of 55 MPa (8,000 psi) and 69 MPa (10,000 psi) and no cost premium are shown in figures 6 and 7, respectively.

Figure 8 shows a cost comparison for the BT-72 with girder compressive strengths of 41, 55, 69, and 83 MPa (6,000, 8,000, 10,000, and 12,000 psi, respectively) and deck concrete strengths of 28 and 69 MPa (4,000 and 10,000 psi, respectively). This figure illustrates the benefits and limitations of using higher-strength concrete with no cost premiums in precast, prestressed bridge girders. At shorter span lengths there are no benefits realized by using the higher-strength concretes. However, at longer span lengths, it is more economical to use the higher-strength concrete in the girders. The higher-strength concrete in the girders results in larger prestressing forces and, consequently, greater girder spacings for a given span length, thus reducing unit costs. These data have been confirmed in previous investigations.[6,7] For the very long span lengths, it is possible only to design for these lengths using the higher-strength concretes. Figure 8 also indicates another important point: the diminishing returns associated with the use of high-strength concrete, the primary cause of which is decreasing strand eccentricity. Once strands are placed within the web, the efficiency of the cross section begins to decrease rapidly. The incremental benefit of each succeeding strand decreases when sufficient room within the flange does not exist. Once additional prestressing force cannot be induced in the girder, the beneficial effects are limited to the increase in concrete tensile strength (which increases only as the square root of the compressive strength).[1]

Optimum cost curves for a FL BT-72 with girder concrete strengths of 41 and 83 MPa (6,000 and 12,000 psi, respectively) and varying deck concrete strengths are shown in figure 9. The data represent the cost index per unit area when there is no cost premium for the higher-strength concrete. This figure shows a pattern of results similar to that shown in figure 8 for the BT-72. In a previous investigation, the FL BT-72 was found to be more cost effective than the BT-72 for span lengths greater than 46 m (150 ft). These data show the same results.

EFFECTS OF CONCRETE COSTS

It is reasonable to expect the payment of a premium for the use of high-strength concrete in bridge decks. This premium results from the increased cost of materials to be used in the concrete, the inexperience of bridge contractors in placing and finishing these concretes, and the necessity of proper curing procedures. Analyses were therefore made using the cost premiums indicated in table 8.

Table 8. Relative premium costs of high-strength concretes.

Girder		Deck Strength (MPa (psi))			
Strength (MPa (psi))	**Cost**	**28 (4,000)**	**41 (6,000)**	**55 (8,000)**	**69 (10,000)**
41 (6,000)	1.00	1.00	1.05	1.13	1.25
55 (8,000)	1.05	1.00	1.05	1.13	1.25
69 (10,000)	1.13	1.00	1.05	1.13	1.25
83 (12,000)	1.25	1.00	1.05	1.13	1.25

Optimum cost curves for a BT-72 with girder concrete strengths of 41, 55, 69, and 83 MPa (6,000, 8,000, 10,000, and 12,000 psi, respectively) and varying deck concrete strengths are shown in figures 10, 11, 12, and 13, respectively. Optimum cost curves for a FL BT-72 with girder concrete strength of 41 and 83 MPa (6,000 and 12,000 psi, respectively) are shown in figures 14 and 15, respectively. These figures indicate that at span lengths of 24 m (80 ft), increasing the deck concrete strength from 28 MPa to 69 MPa (4,000 to 10,000 psi) results in an increase in the cost per unit area of approximately 10 percent for both the Bulb-Tee and the Florida Bulb-Tee. At span lengths of 44.5 m (146 ft) the percentage of increase is approximately 8 percent for both strengths of girder concrete. At the maximum span lengths achievable with the BT-72 and the FL BT-72 with a girder concrete strength of 83 MPa (12,000 psi), the increase is approximately 5 percent.

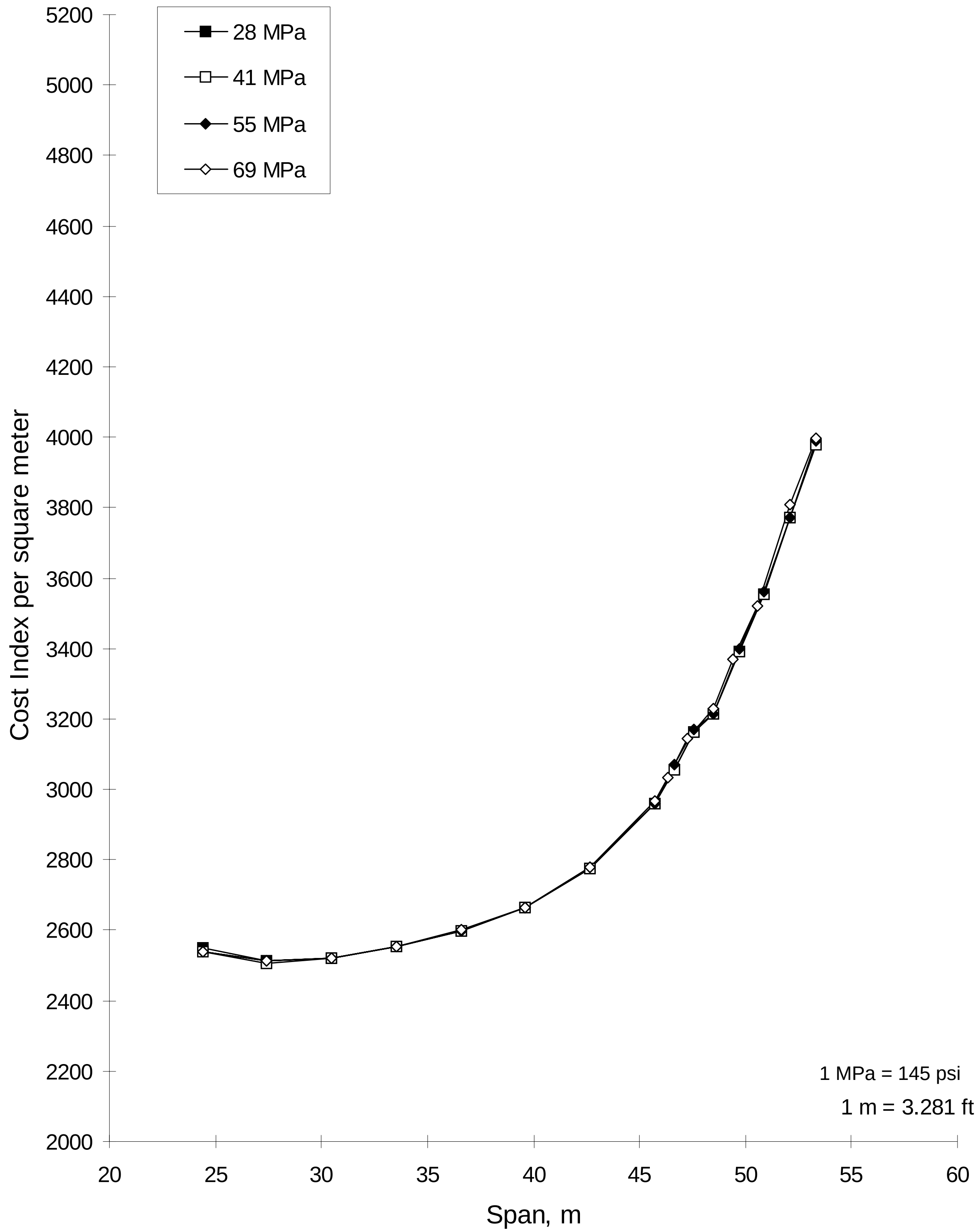

Figure 5. Optimum cost curves for a BT-72, 83 MPa.

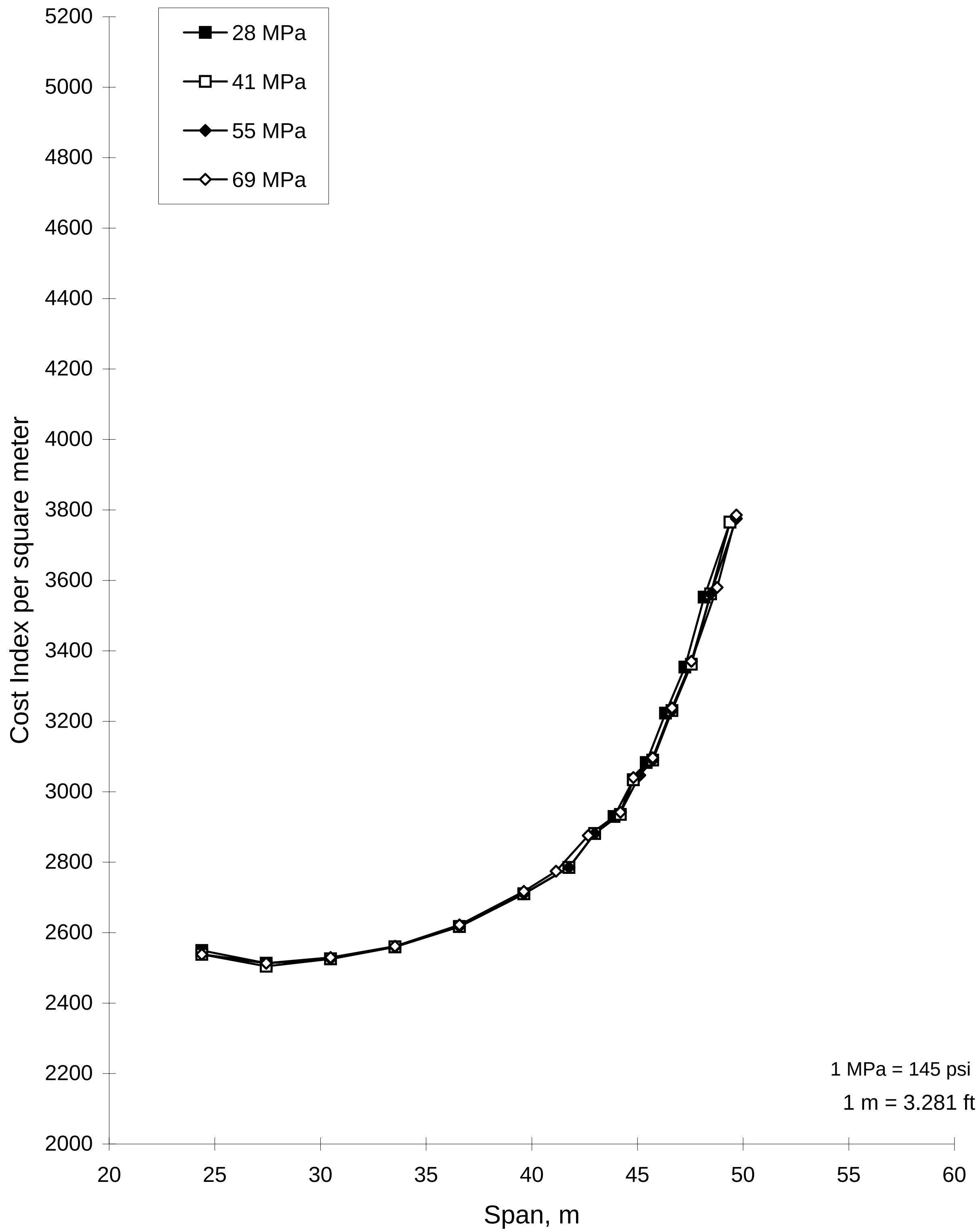

Figure 6. Optimum cost curves for a BT-72, 55 MPa.

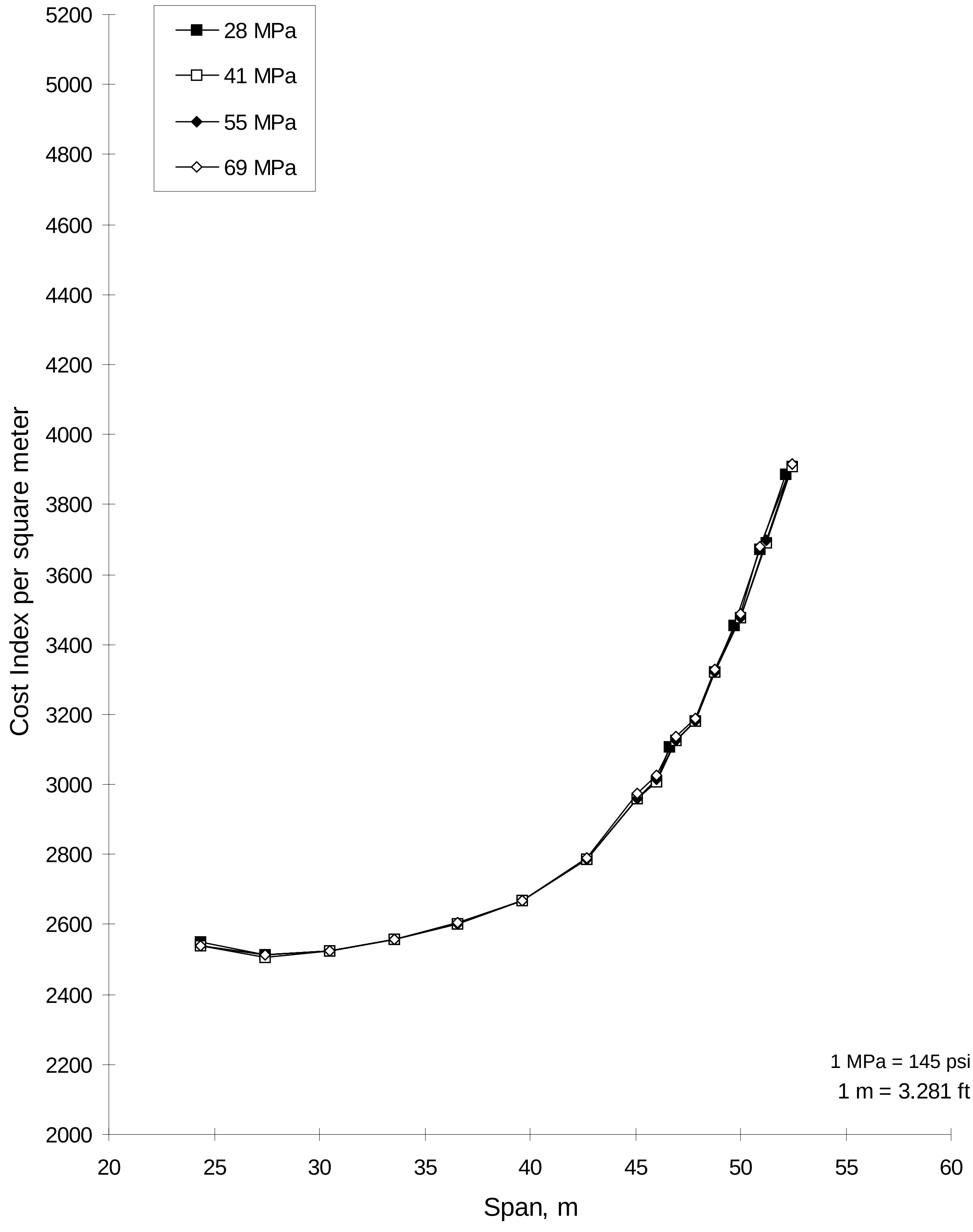

Figure 7. Optimum cost curves for a BT-72, 69 MPa.

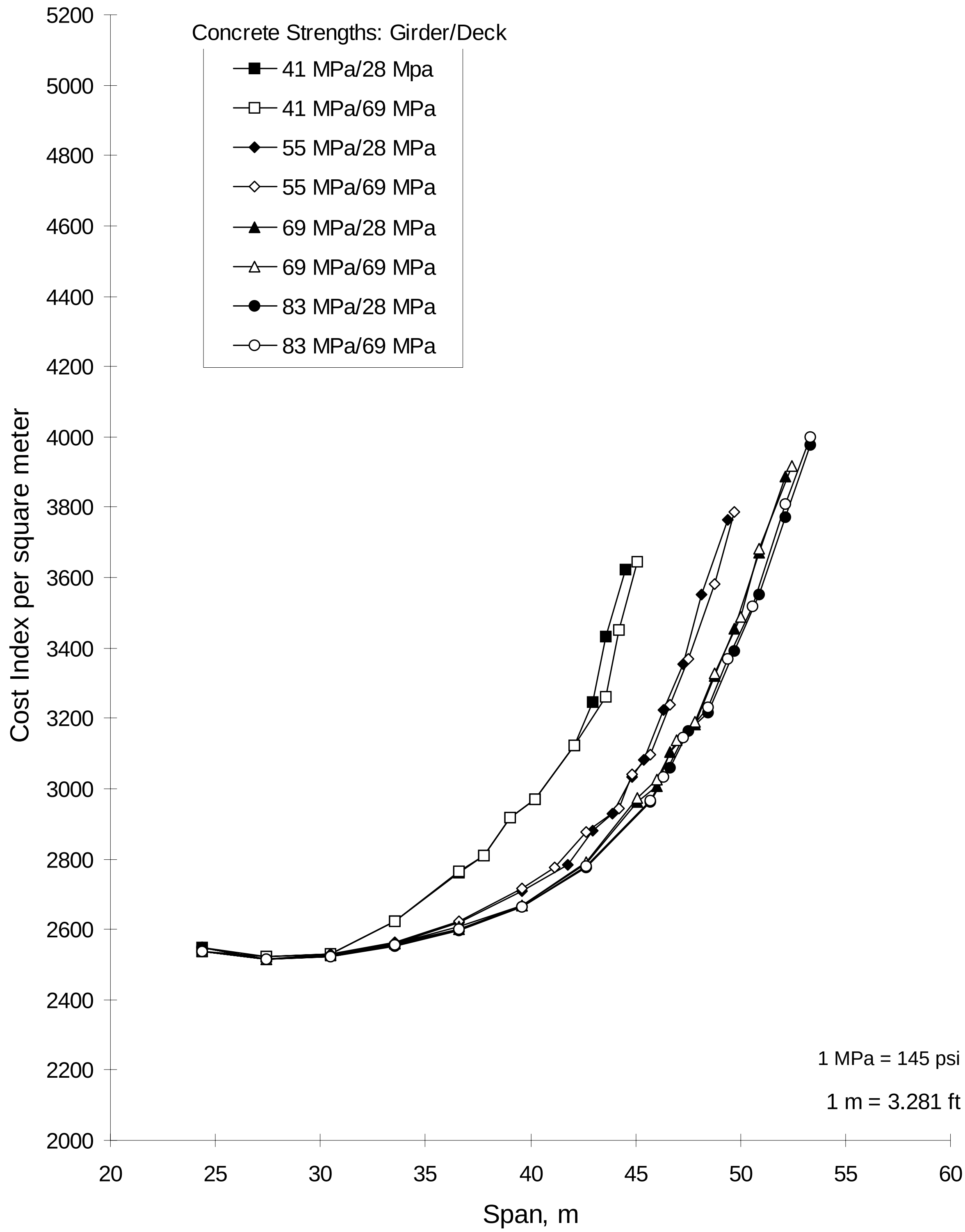

Figure 8. Comparison of optimum cost curves for a BT-72 with varying concrete strengths.

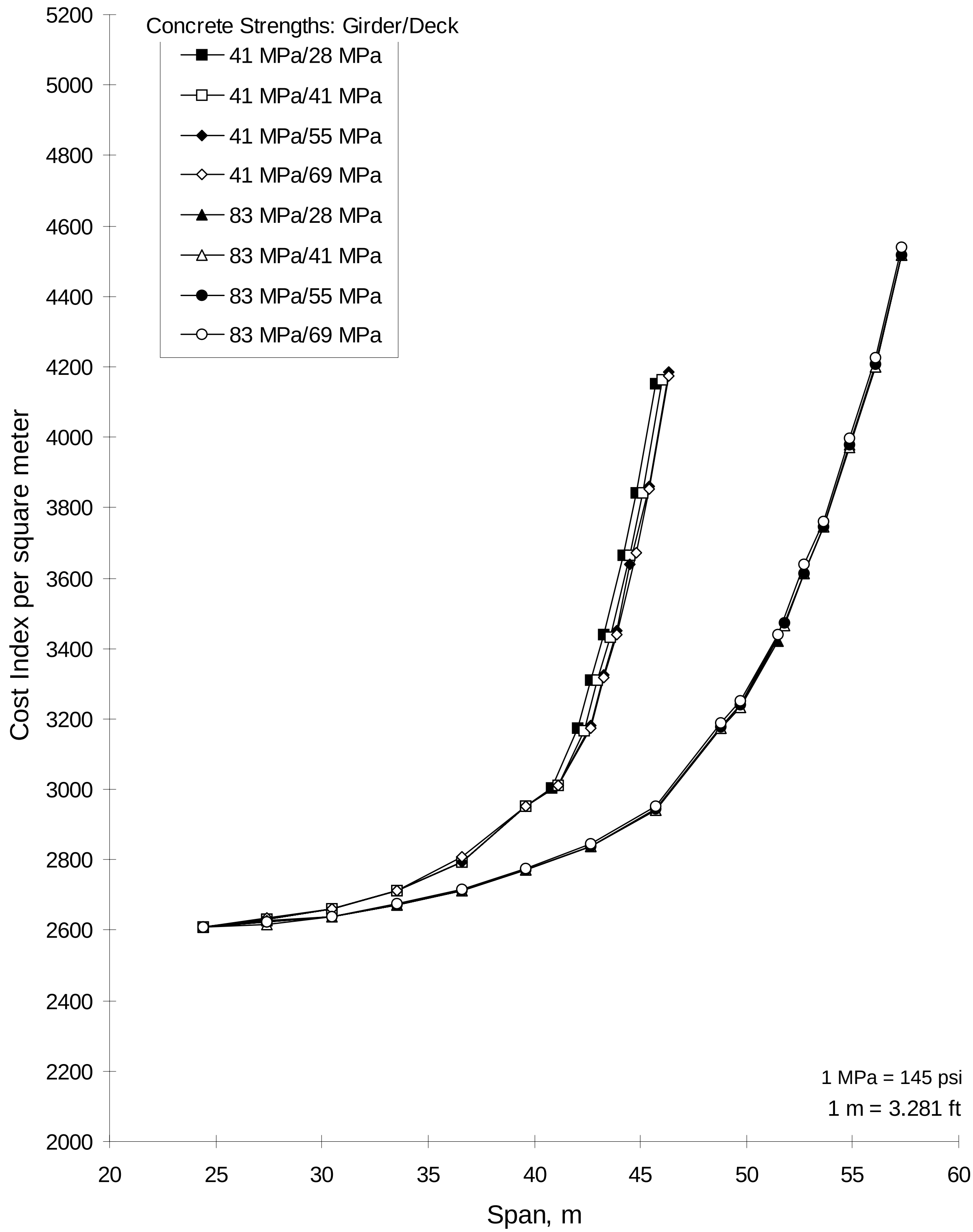

Figure 9. Comparison of optimum cost curves for a FL BT-72 with varying concrete strengths.

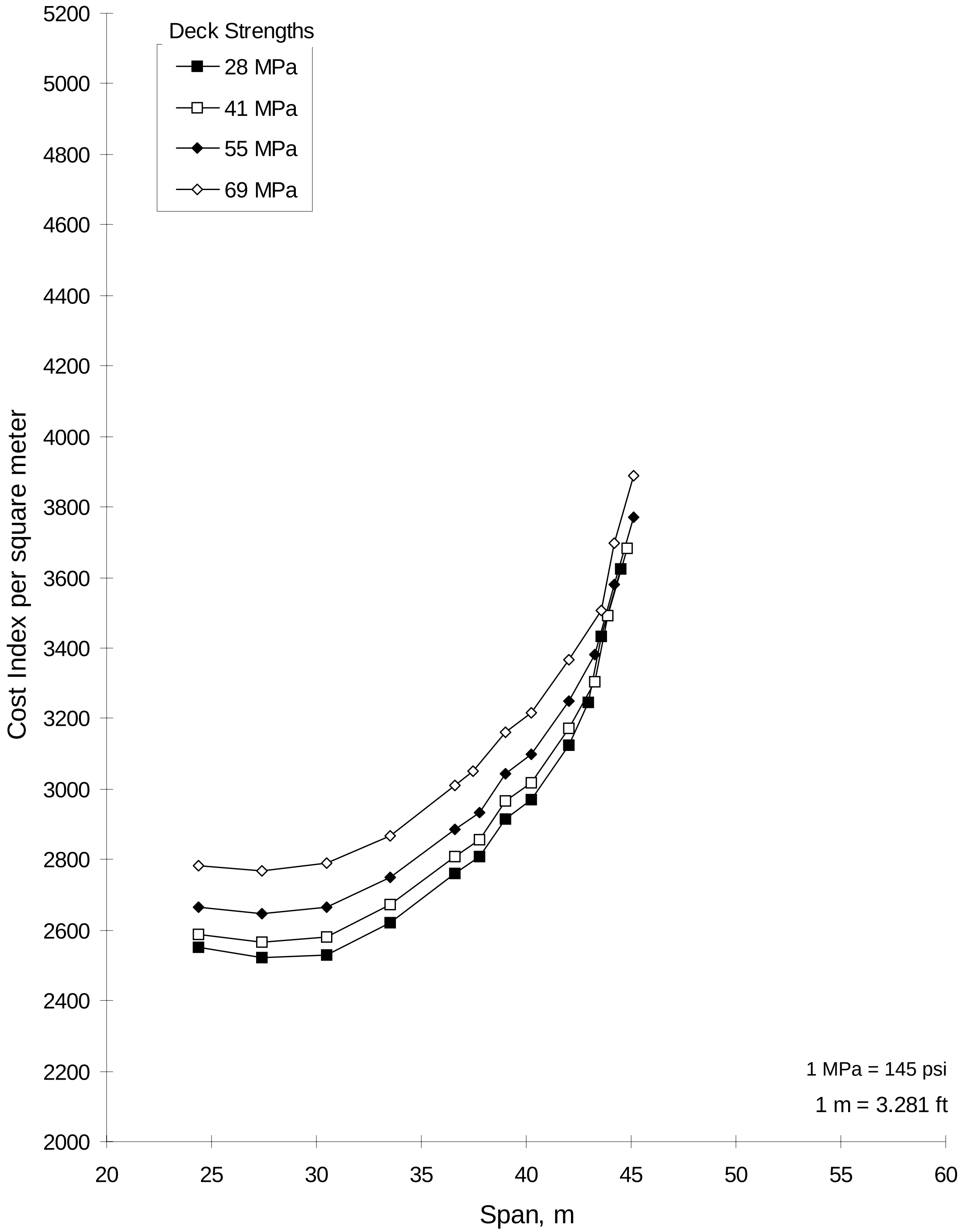

Figure 10. Optimum cost curves for a BT-72, 41 MPa with cost premium.

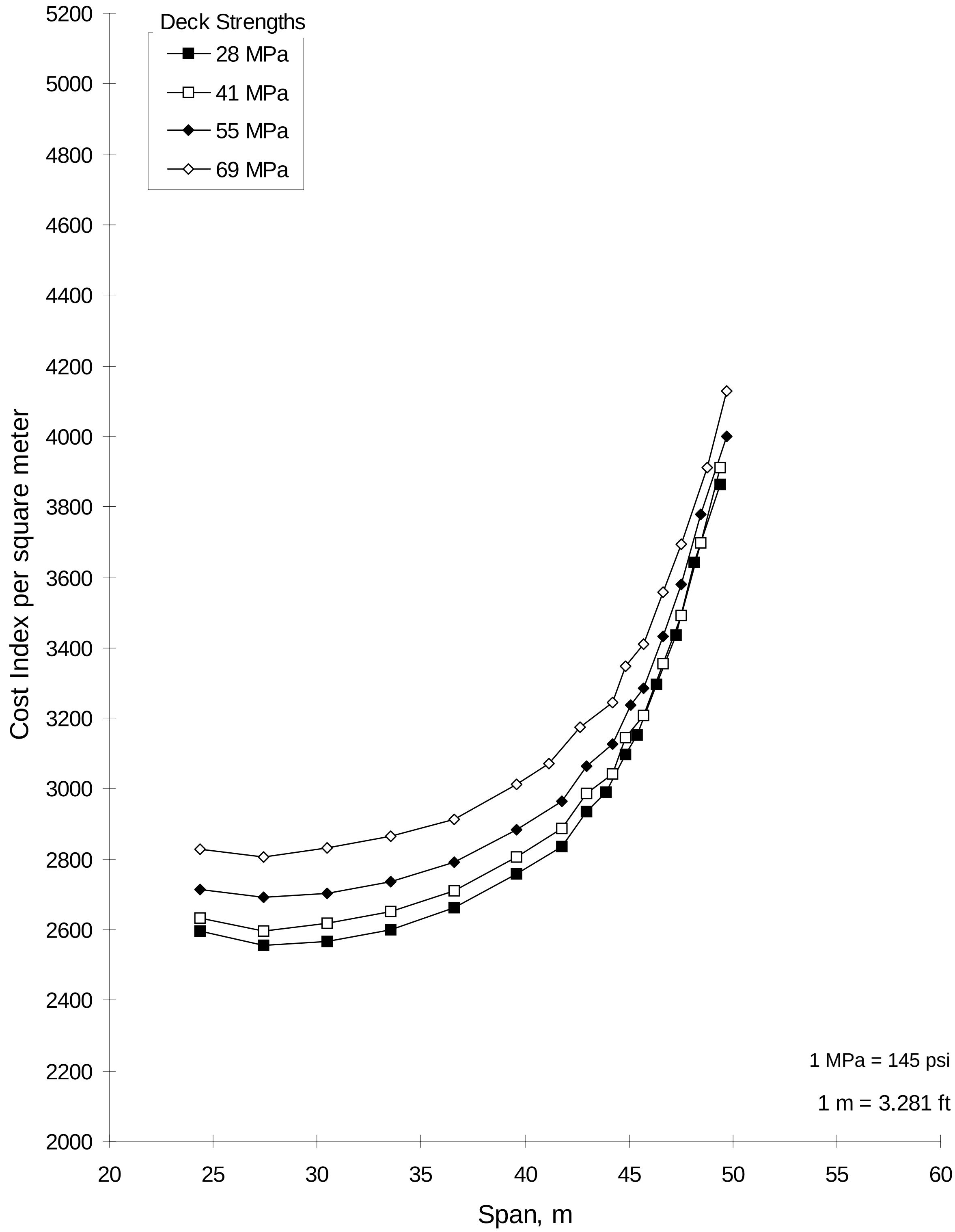

Figure 11. Optimum cost curves for a BT-72, 55 MPa with cost premium.

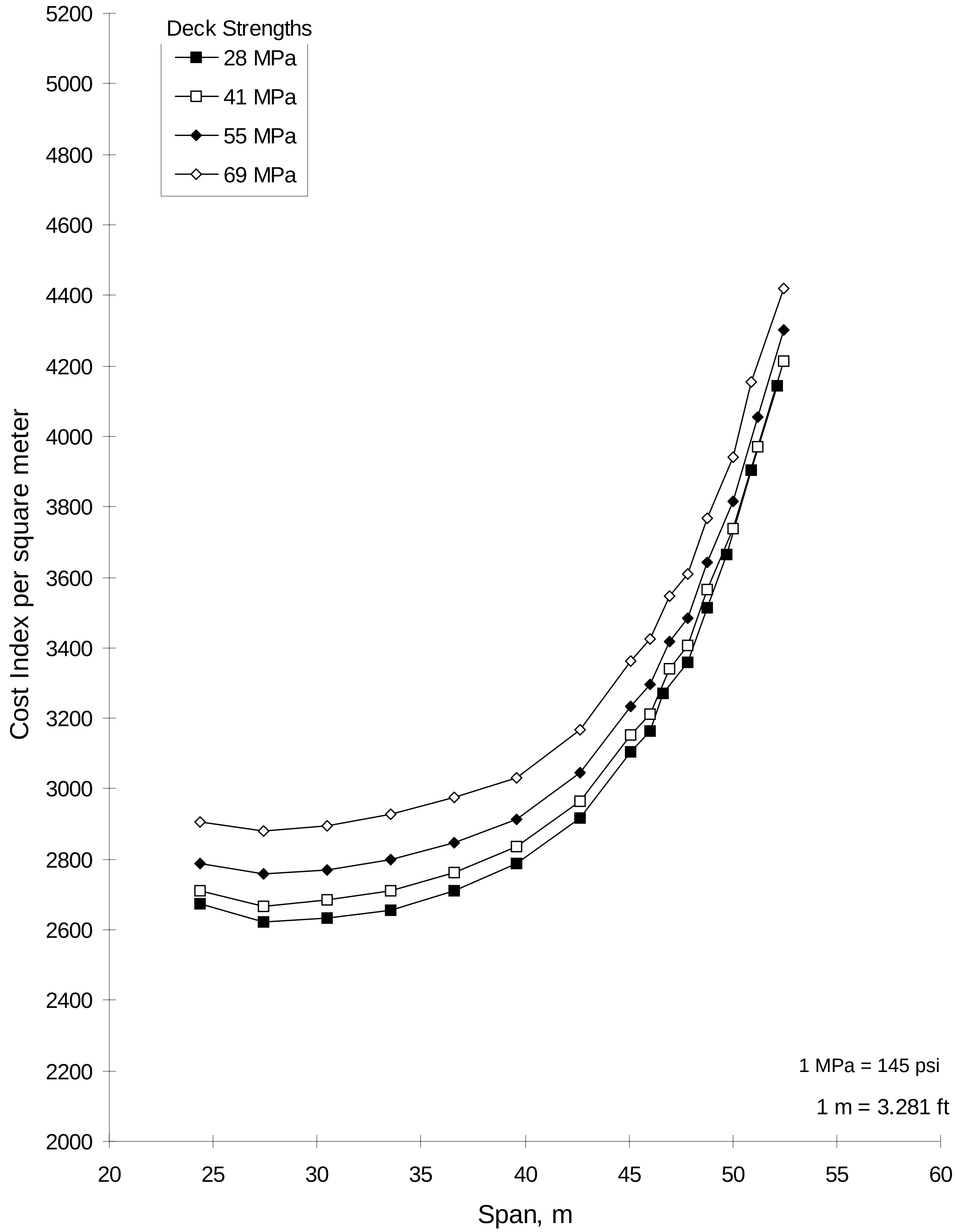

Figure 12. Optimum cost curves for a BT-72, 69 MPa with cost premium.

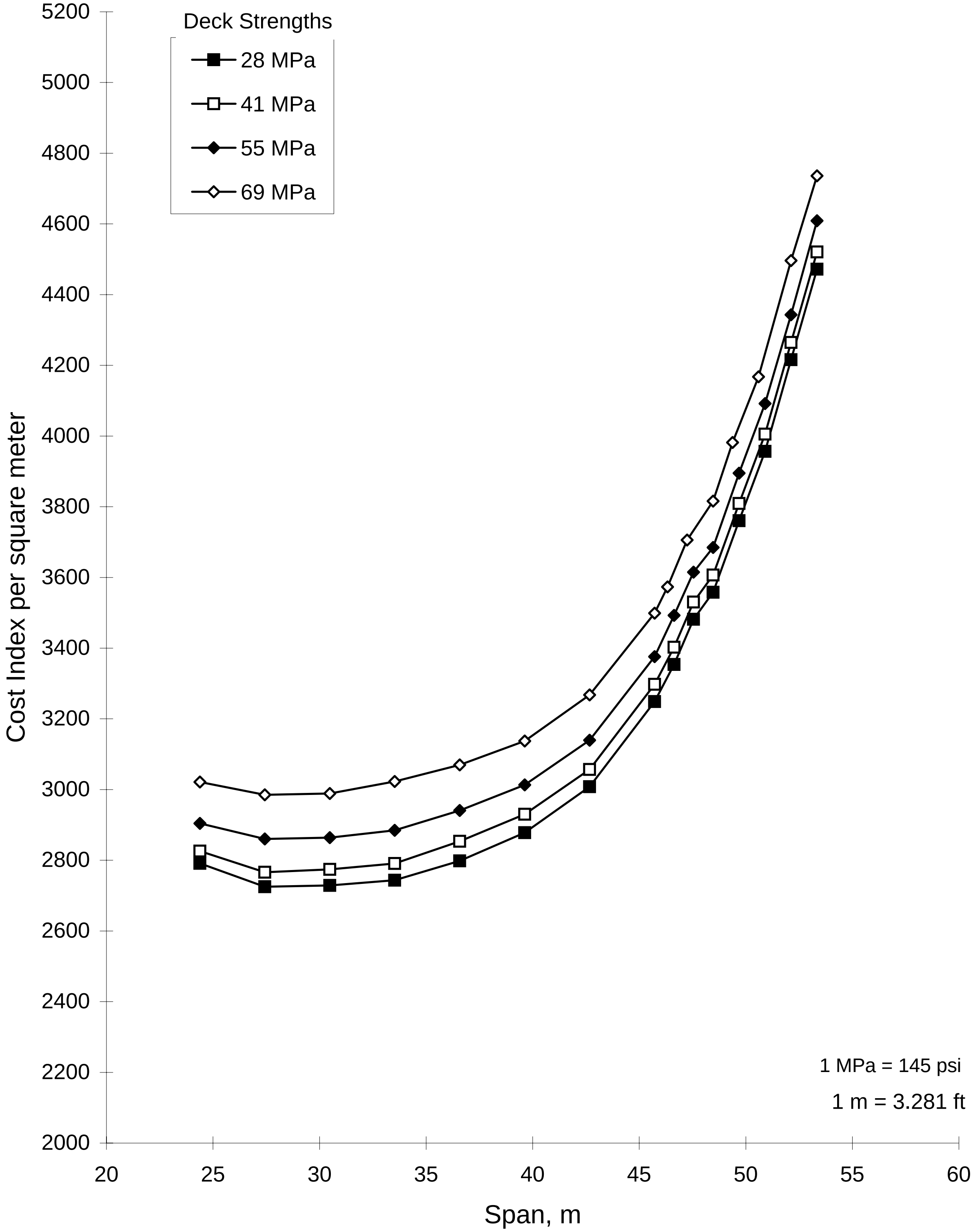

Figure 13. Optimum cost curves for a BT-72, 83 MPa with cost premium.

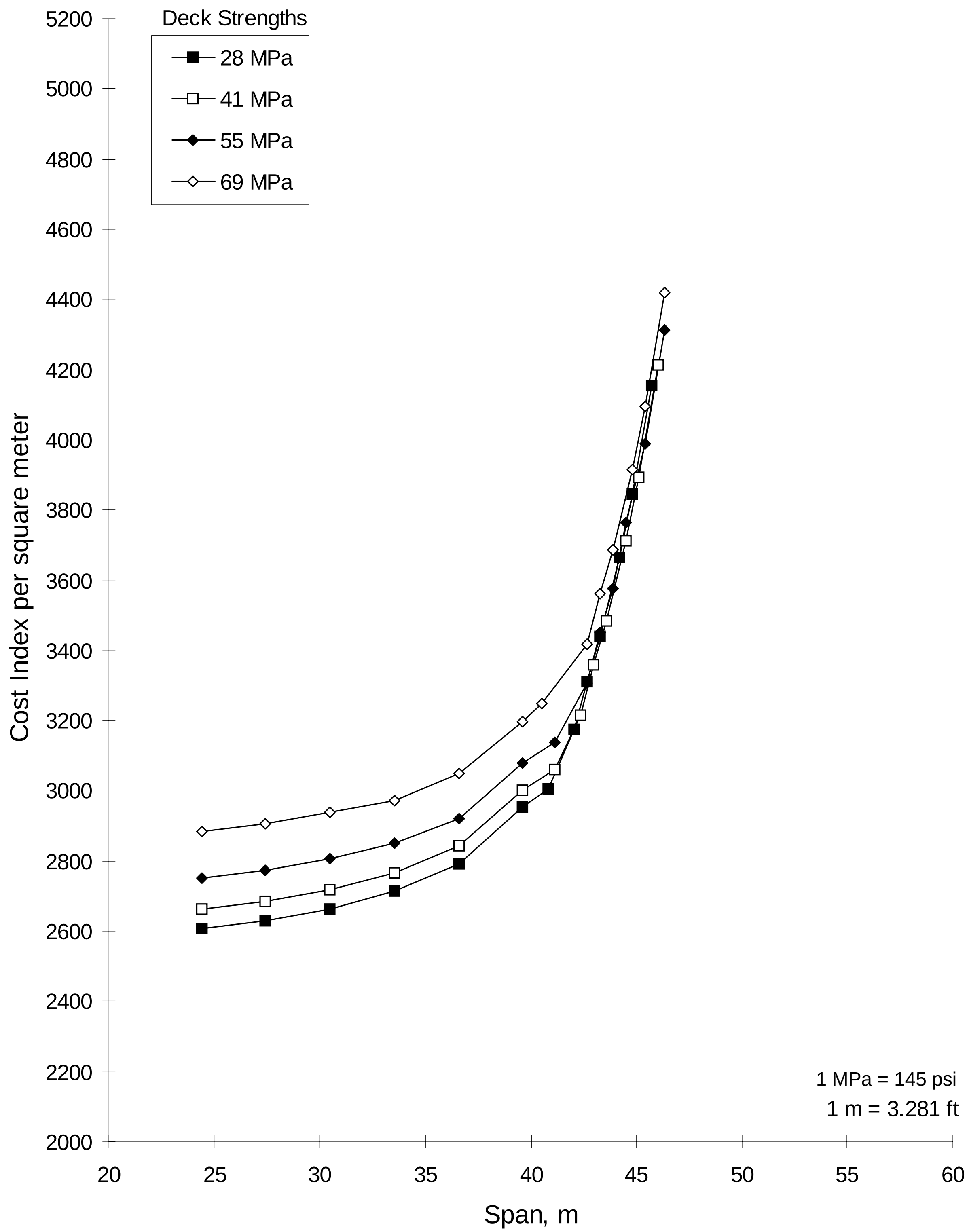

Figure 14. Optimum cost curves for a FL BT-72, 41 MPa with cost premium.

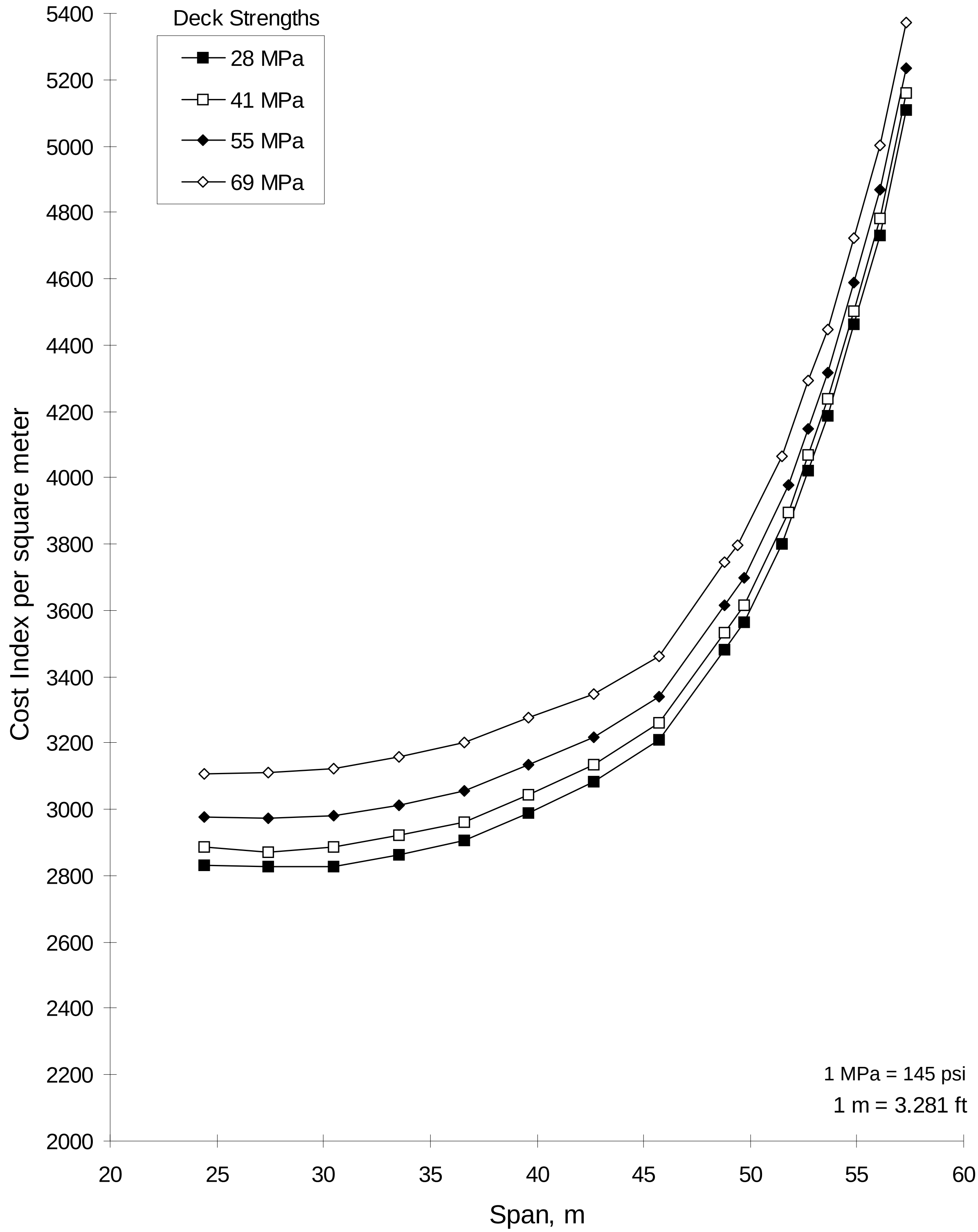

Figure 15. Optimum cost curves for a FL BT-72, 83 MPa with cost premium.

Consequently, it may be concluded that even if a 69-MPa (10,000-psi) concrete has an in-place cost that is 25 percent greater than a 28-MPa (4,000-psi) concrete, the overall unit cost of the superstructure will only increase in the range of 5–10 percent.

TASK 1 CONCLUSIONS

The analyses performed in task 1 considered only the initial costs. They did not take into account that high-strength concrete in the deck is also high-performance concrete and will have improved durability compared to a deck produced with a lower strength concrete. This should result in less maintenance costs and reduced life cycle costs. A life cycle cost study is beyond the scope of work of this report.

Based on analyses made in task 1, the following conclusions are made:

- The use of high-strength concrete in bridge decks will not result in a reduction of deck thickness or in the amount of transverse reinforcement in the deck. Therefore, no cost savings can be expected from a reduction in materials.
- The use of high-strength concrete in bridge decks allows for a slight increase in the maximum span length of the BT-72 and FL BT-72.
- With no premium costs for high-strength concrete in the deck and girders, there is a slight reduction in the cost per unit area for the longer span lengths.
- With a 25 percent increase in the in-place cost of the deck concrete, the overall unit cost of the superstructure will only increase in the range of 5–10 percent.
- The use of high-strength concrete in bridge decks will result in less live load deflection.

CHAPTER 3. TASK 2: ANALYSES OF FLEXURAL STRENGTH AND DUCTILITY

The general philosophy in prestressed concrete design is that concrete members shall be designed so that the steel is yielding as ultimate capacity is approached. This is generally achieved by specifying a maximum amount of reinforcement for a given cross section. When the reinforcement exceeds the specified amount, the design-moment strength is based on the compression portion of the moment couple. In an over-reinforced section, an increase in the compressive strength of the deck concrete results in an increase in the moment capacity. It should be noted that current design requirements have been developed based on lower strength concretes. It was, therefore, deemed appropriate to investigate the flexural strength and ductility that would result when higher-strength concretes are used in bridge decks.

RESEARCH APPROACH

Analyses for the effect of higher concrete strengths on the flexural strength and moment-curvature relationships were investigated using a computer program known as BEAM BUSTER. The program BEAM BUSTER performs a moment-curvature analysis of a reinforced or prestressed element using actual material properties. The program takes into account uncracked, cracked, and post-yield behavior of flexural members for a specified cross section and curvature. The program BEAM BUSTER calculates the strains and stresses that satisfy equilibrium of forces on the cross section and compatibility of strains. A cross section may be divided into many elements. A different stress–strain curve may be utilized for each element of the cross section. The stress–strain data are input as discrete data points along the complete stress–strain curve for each element specified in the cross section.

Analyses were performed for the following variables:

- Girder concrete compressive strength: 41 and 83 MPa (6,000 and 12,000 psi, respectively).
- Deck concrete compressive strength: 28, 41, 55, and 69 MPa (4,000, 6,000, 8,000 and 10,000 psi, respectively).
- Girder cross section: PCI Bulb-Tee with a depth of 1.83 m (72 inches).
- Span lengths: 24.3, 44.5, and 53.3 m (80, 146, and 175 ft, respectively).

The combination of variables selected for analysis is defined in tables 9 and 10. The combinations were selected to represent a range of girder strengths and spans.

Table 9. Task 2 variables (SI units).

Series	Girder Strength (MPa)	Deck Strength (MPa)	Span (m)	No. of Strands*	Flange Width (m)
A	41 41 41 41	28 41 55 69	24.4 24.4 24.4 24.4	19 19 19 19	2.44 2.44 2.44 2.44
B	83 83 83 83	28 41 55 69	24.4 24.4 24.4 24.4	19 19 19 19	2.44 2.44 2.44 2.44
C	41 41 41 41	28 41 55 69	44.5 44.5 44.5 44.5	41 41 41 41	1.37 1.37 1.37 1.37
D	83 83 83 83	28 41 55 69	53.3 53.3 53.3 53.3	76 76 76 76	1.37 1.37 1.37 1.37

* For consistency between tasks, the odd number of strands calculated by program BRIDGE in task 1 was retained in task 2.

Table 10. Task 2 variables (English units).

Series	Girder Strength (psi)	Deck Strength (psi)	Span (ft)	No. of Strands*	Flange Width (inches)
A	6,000	4,000	80	19	96
	6,000	6,000	80	19	96
	6,000	8,000	80	19	96
	6,000	10,000	80	19	96
B	12,000	4,000	80	19	96
	12,000	6,000	80	19	96
	12,000	8,000	80	19	96
	12,000	10,000	80	19	96
C	6,000	4,000	146	41	54
	6,000	6,000	146	41	54
	6,000	8,000	146	41	54
	6,000	10,000	146	41	54
D	12,000	4,000	175	76	54
	12,000	6,000	175	76	54
	12,000	8,000	175	76	54
	12,000	10,000	175	76	54

* For consistency between tasks, the odd number of strands calculated by program BRIDGE in task 1 were retained in task 2.

The cross sections used in the analyses are shown in figure 16. For purposes of simplicity, the non-prestressed reinforcement in the deck was not included in the analyses. The flange width of the composite section and number of strands were based on the calculations performed by the program BRIDGE in task 1.

Prior to performing the analyses, it was necessary to define the stress–strain curves for the individual constituent materials consisting of deck concrete, girder concrete, and prestressing strand.

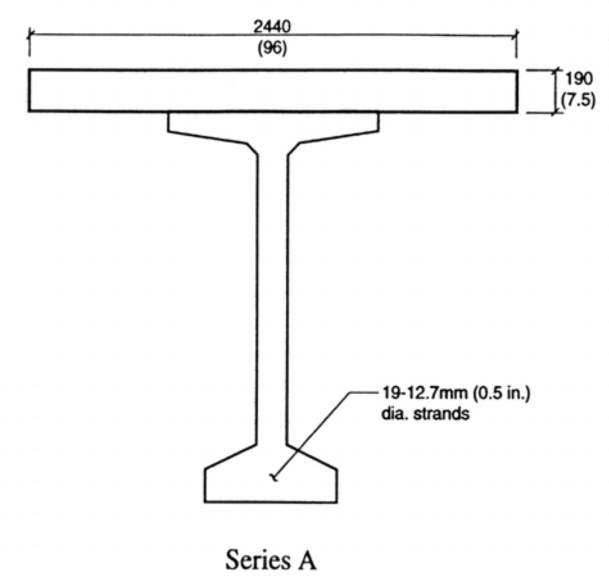

Figure 16 (part 1). Cross section of series A girder (BT-72) analyzed in task 2. All dimensions are in millimeters (inches).

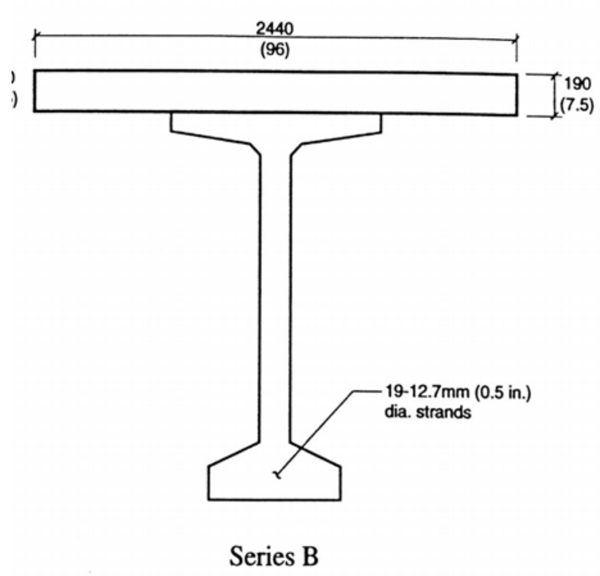

Figure 16 (part 2). Cross section of series B girder (BT-72) analyzed in task 2. All dimensions are in millimeters (inches).

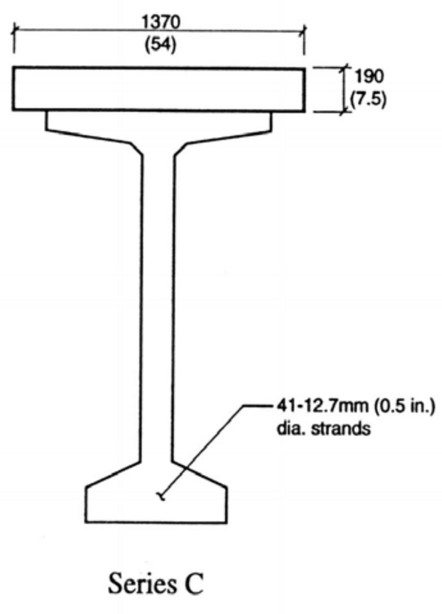

Figure 16 (part 3). Cross section of series C girder (BT-72) analyzed in task 2. All dimensions are in millimeters (inches).

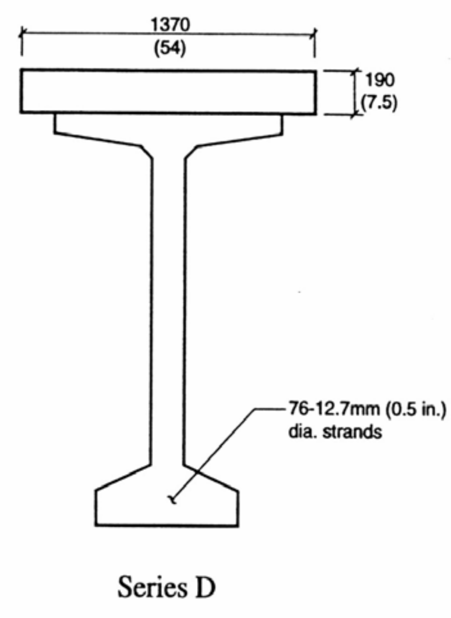

Figure 16 (part 4). Cross section of series D girder (BT-72) analyzed in task 2. All dimensions are in millimeters (inches).

MATERIAL PROPERTIES

Required input for the program BEAM BUSTER consists of stress–strain curves for each element analyzed in the cross section. Therefore, it was necessary to define a family of stress–strain curves for different strength concretes and to select an appropriate curve for the prestressing strand.

Stress–Strain Curves for Concrete

Since the calculations for ductility are very sensitive to the assumed shape of the stress–strain curves, it was necessary to establish a family of curves that realistically represent the properties of high-strength concrete. Several researchers have experimentally determined the complete stress-strain curves of concrete for strengths up to about 100 MPa (14,500 psi).[18,19] The measurements have indicated that the following occur with increasing strength:

- The initial slope of the stress–strain curve increases.
- The ascending portion of the stress–strain curve is more linear.
- The strain at peak stress increases.
- The slope of the descending portion of the curve increases.

The slope of the ascending portion of the stress–strain curve is the modulus of elasticity of the concrete. Several equations have been proposed that relate the modulus of elasticity, E_c, to the concrete compressive strength, f'_c, and the unit weight of the concrete, w_c, including:

ACI 318:[20]
$$E_c = 33 w_c^{1.5} \sqrt{f_c'} \qquad (1)$$

Martinez:[21]
$$E_c = \left(40{,}000\sqrt{f_c'} + 1{,}000{,}000\right)\left(\frac{w_c}{145}\right)^{1.5} \qquad (2)$$

Canadian Code:[22]
$$E_c = \left(39{,}740\sqrt{f_c'} + 1{,}000{,}000\right)\left(\frac{w_c}{143.5}\right)^{1.5} \qquad (3)$$

Ahmad:[19]
$$E_c = \left(w_c\right)^{2.5}\left(f_c'\right)^{0.325} \qquad (4)$$

Values of modulus of elasticity calculated according to the above equations are tabulated in table 11.

Table 11. Calculated values of modulus of elasticity.

Compressive Strength (MPa)	Unit Weight (kg/m^3)	Equation ACI[19] (GPa)	Martinez[20] (GPa)	Canadian Code[21] (GPa)	Ahmad[18] (GPa)
28	2320	25.1	24.3	24.5	25.8
41	2370	31.7	29.2	29.0	31.0
55	2420	37.8	33.5	33.9	35.9
69	2480	43.9	38.1	38.5	39.1
83	2500	48.5	41.4	41.8	44.3
(psi)	(lb/ft^3)	(10^6 psi)	(10^6 psi)	(10^6 psi)	(10^6 psi)
4,000	145	3.64	3.53	3.55	3.75
6,000	148	4.60	4.23	4.21	4.50
8,000	151	5.48	4.86	4.92	5.20
10,000	155	6.37	5.53	5.59	5.97
12,000	156	7.04	6.01	6.07	6.43

The ACI equation was based on an analysis for concrete strengths up to about 41 MPa (6,000 psi).[20,23] Several investigators have indicated that the ACI equation tends to overestimate the modulus of elasticity for the higher-strength concretes.[1,13,19] The Martinez equation was developed as an alternative for the ACI equation and gives lower values of modulus of elasticity at the higher strength levels.[21] Some publications have indicated that the Martinez equation may underestimate the modulus of elasticity at the very high strength levels.[19] The Canadian Code equation was based on the Martinez equation with some rounding off for use in SI units and then converted back into English units. The rounding off results in slightly different calculated values. The Ahmad equation was based on a statistical analysis of data. Modulus values from the Ahmad equation lie between those of the ACI and the Martinez equations. For the present investigation, it was decided to use the ACI values for concrete strengths of 28 and 41 MPa (4,000 and 6,000 psi, respectively) and to use the values by the Ahmad equation for concrete strengths of 55, 69, and 83 MPa (8,000, 10,000, and 12,000 psi, respectively).

Collins et al. have indicated that the strain at peak stress, ε_c', can be calculated from the following equation:[24]

$$\varepsilon_c' = \frac{f_c'}{E_c} \times \frac{n}{n-1} \qquad (5)$$

where

$$n = 0.8 + \frac{f_c^{'}}{17} \quad \text{in MPa units} \tag{6}$$

$$n = 0.8 + \frac{f_c^{'}}{2500} \quad \text{in psi units} \tag{7}$$

These equations were used in the present analyses to calculate the strain at peak stress from the modulus of elasticity and concrete compressive strength.

Several investigators have published algebraic expressions to accurately describe the shape of the rising and descending branch of the stress-strain curve. The following expression was proposed by Popovics:[25]

$$\frac{f_c}{f_c^{'}} = \frac{\varepsilon_c}{\varepsilon_c^{'}} \times \frac{n}{(n-1) + \left(\frac{\varepsilon_c}{\varepsilon_c^{'}}\right)^n} \tag{8}$$

where f_c = stress at a strain of ε_c.

Thorenfeldt proposed a modification to Popovic's equation to increase the slope of the descending portion of the stress-strain curve.[26] He introduced a factor, *k*, to modify the equation as follows:

$$\frac{f_c}{f_c^{'}} = \frac{\varepsilon_c}{\varepsilon_c^{'}} \times \frac{n}{(n-1) + \left(\frac{\varepsilon_c}{\varepsilon_c^{'}}\right)^{nk}} \tag{9}$$

where *k* = 1 on the ascending portion of the curve, and *k* is greater than 1 for the descending portion of the curve. Collins has suggested the following for the descending portion of the curve.[24]

$$k = 0.67 + \frac{f_c^{'}}{62} \quad \text{in MPa units} \tag{10}$$

$$k = 0.67 + \frac{f_c^{'}}{9000} \quad \text{in psi units} \tag{11}$$

Stress–strain curves generated using the above approach are shown in figure 17.

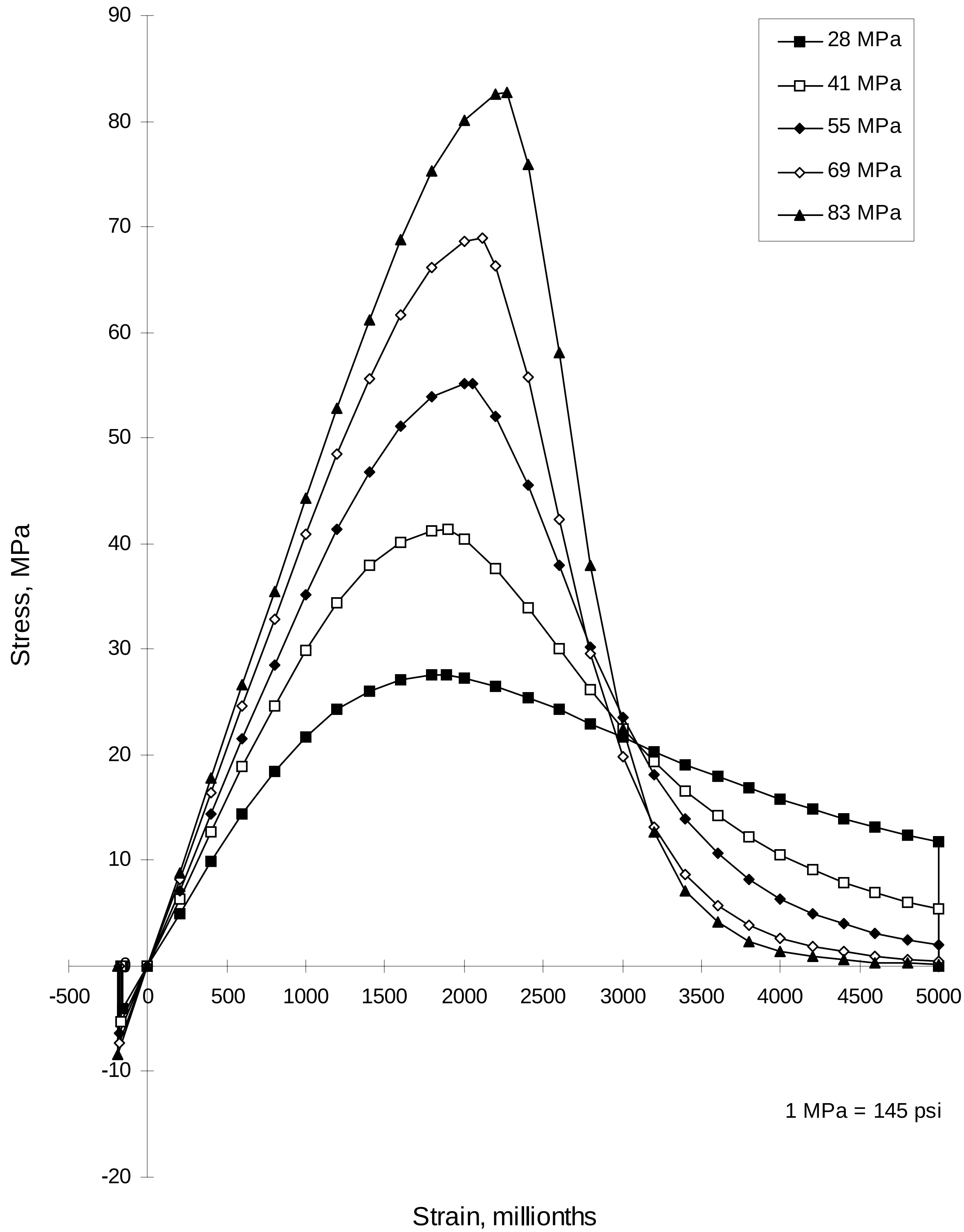

Figure 17. Stress–strain curves for concrete used in BEAM BUSTER analysis.

To provide some tensile capacity to the concrete prior to cracking, the stress–strain curves developed in compression were extrapolated backwards using the following assumptions:

- The slope of the curve in tension is the same as that in compression near the origin.
- A maximum tensile stress, ft, calculated by the following equation, was used:[19]

$$f_t = 2.30\left(f_c^{'}\right)^{2/3} \qquad (12)$$

The stress–strain curve in compression was calculated out to a strain of 0.005. The calculated stress–strain curves were then compared with the measured curves obtained by Kaar.[18] These comparisons were made to ensure that the calculated curves showed reasonable agreement with measured data. It should also be noted that in these curves, the peak stress corresponds to the designated concrete compressive strength. In concrete column testing, it has been observed that the maximum stress obtained in the column concrete can be less than the strength measured on a standard concrete cylinder. This difference is attributed to the difference in size and shape between the reinforced concrete column and the concrete cylinder; the differences in concrete casting, vibration, and curing; and to differences in the rate of loading. In his original column research, Hognestad chose a value of 0.85 for the ratio between maximum stress in the column and concrete cylinder strength.[27] He indicated that this factor may be systematically too high or too low and may not be a constant. In tests of plain concrete columns subjected to linearly varying strain, Hognestad and Kaar deduced (separately) that the factor was close to 1.0, varying from 0.96 to 1.12.[18,28] In design, a value of 0.85 is normally used. However, for purposes of the present analyses, a value of 1.0 was selected.

Stress–Strain Properties of Strand

An assumed stress–strain curve for strand was used in computer program BRIDGE in task 1. This curve is shown in figure 18. A slight modification to the curve was made for use in the program BEAM BUSTER. This modification is shown in figure 18.

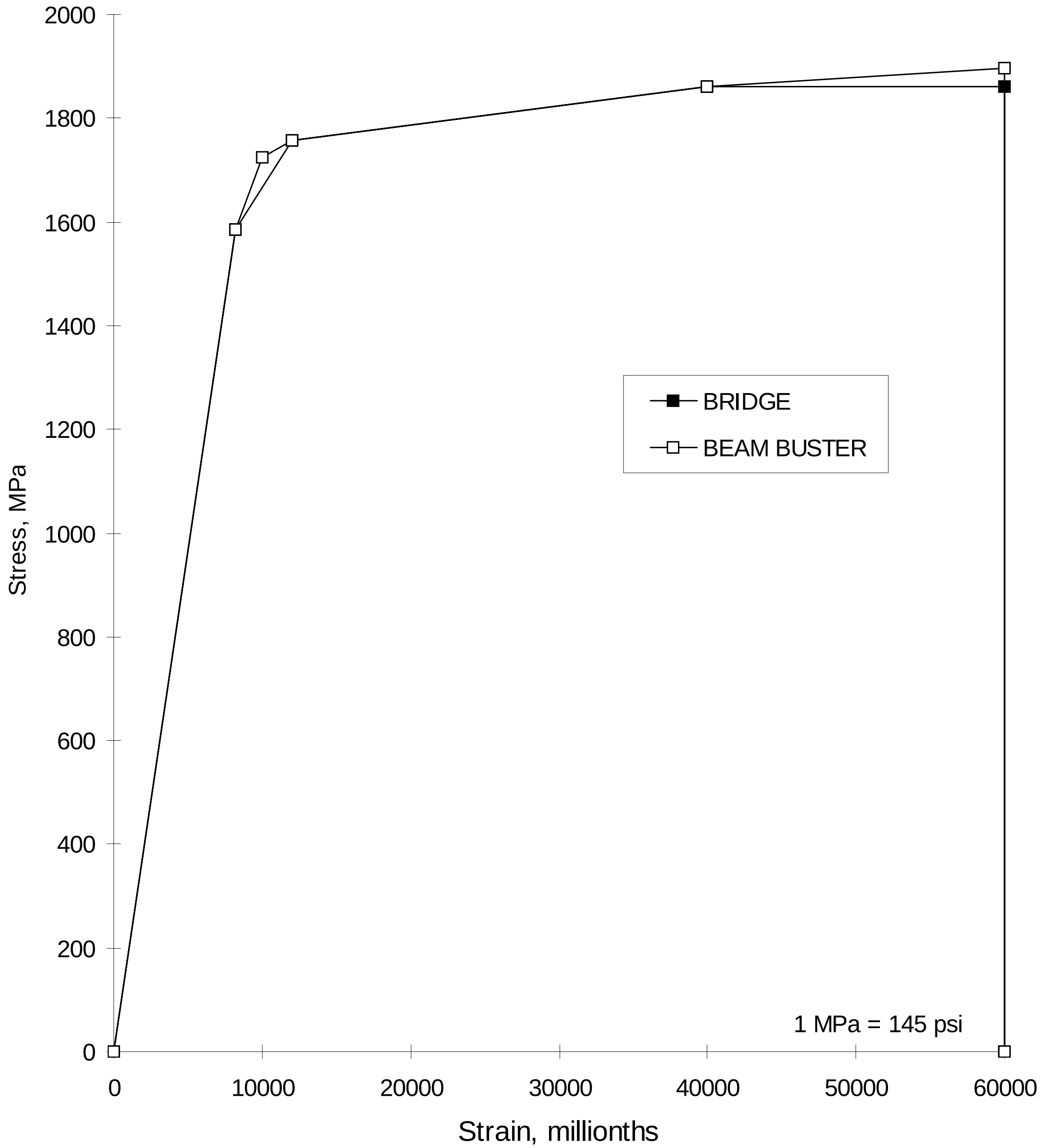

Figure 18. Stress–strain curve for prestressing strand used in BEAM BUSTER analysis.

MOMENT-CURVATURE RELATIONSHIPS

The moment-curvature relationships were calculated in two parts. In the first part, the moment was applied to the noncomposite section consisting of the prestressed concrete girder only. This part of the calculation represents the moments caused by girder dead load and deck dead load, and assumes that the dead load of the deck is carried entirely by the girder prior to development of composite action.

In the second part of the calculation, the moment-curvature relationship was calculated for the composite section. This calculation represents any moment applied after the deck and girder act as a composite member. To ensure compatibility between the two parts of the analyses, it was necessary to match the moment-curvature relationships at the transition point between the two parts of the calculation. This was accomplished by making an artificial adjustment in the stress-strain curves for the concrete deck. The deck was assumed to consist of three layers of concrete and a separate adjustment was made for each layer. The girder concrete was assumed to consist of one layer. For a specified input curvature, the program BEAM BUSTER calculates the corresponding moment that results in equilibrium of forces and compatibility of strains. The output consists of curvature, moment, and selected strains and stresses. If compatibility of strains or equilibrium of forces cannot be calculated for a given curvature, the calculation for that curvature is terminated after a specified number of iterations. This generally occurs after the stress-strain curve for a concrete element or prestressing strand has returned to zero strain.

Moment-curvature relationships for the four series of analyses described in tables 9 and 10 are shown in figures 19 through 22. Figures 19 and 20 show the relationships for 41-MPa (6,000 psi) and 83-MPa (12,000-psi) girder strength concrete, respectively, at a span of 24.4 m (80 ft)—the minimum span length considered in this investigation. Figure 21 shows the relationship for a girder compressive strength of 41 MPa (6,000 psi) at a span of 44.5 m (146 ft)—the longest span for which a 41 MPa (6,000 psi) compressive girder can be designed. Figure 22 shows the relationship for a girder with concrete compressive strength of 83 MPa (12,000 psi) at a span of 53.3 m (175 ft)—the longest span length for which an 83 MPa (12,000 psi) girder can be designed.

All of the moment-curvature relationships show a similar shape that can be divided into four parts. The first part consists of the moment-curvature relationship for the noncomposite section. This begins at a negative curvature because of the prestressing force and continues until a moment equivalent to the dead load of the deck and girder have been applied. The second part of the moment-curvature relationship consists of the moment applied to the composite section prior to cracking of the concrete. This part has a slightly steeper slope than that for the noncomposite section because of the higher stiffness of the composite section. The third part of the curve consists of the moment-curvature relationship following cracking and prior to yielding of the prestressing strand. The slope of this portion of the curve is considerably less than the curve for the uncracked section. The slope is greatest for the sections that contain the largest number of strands. The final part of the curve consists of the moment-curvature relationship following yielding of the strand. This portion of the curve is largely horizontal and generally continues until the end point of one of the stress-strain curves is reached. The moments corresponding to girder and deck dead load, girder and deck dead load and live load plus impact, and required strength are also shown on the figures.

As shown in figures 19 and 20, the moment-curvature relationships for the girders containing 19 strands at a span of 24.4 m (80 ft) are not influenced by the deck concrete strength. Minor differences do occur, but at the scale shown in these figures, the differences are not discernible. The maximum moment for these sections is achieved when the prestressing strand reaches its maximum stress (which was assumed to be at a strain of 6 percent). This indicates that these cross sections will have a flexural capacity determined by fracture of the prestressing strand. These sections had a final curvature that was about 10 times the curvature when the first strands yielded.

The impact of utilizing higher-strength concrete in the deck is more evident in figures 21 and 22. In the cross sections with a larger number of strands, the breaking strength of the strand is not reached prior to the flexural strength of the section being achieved.

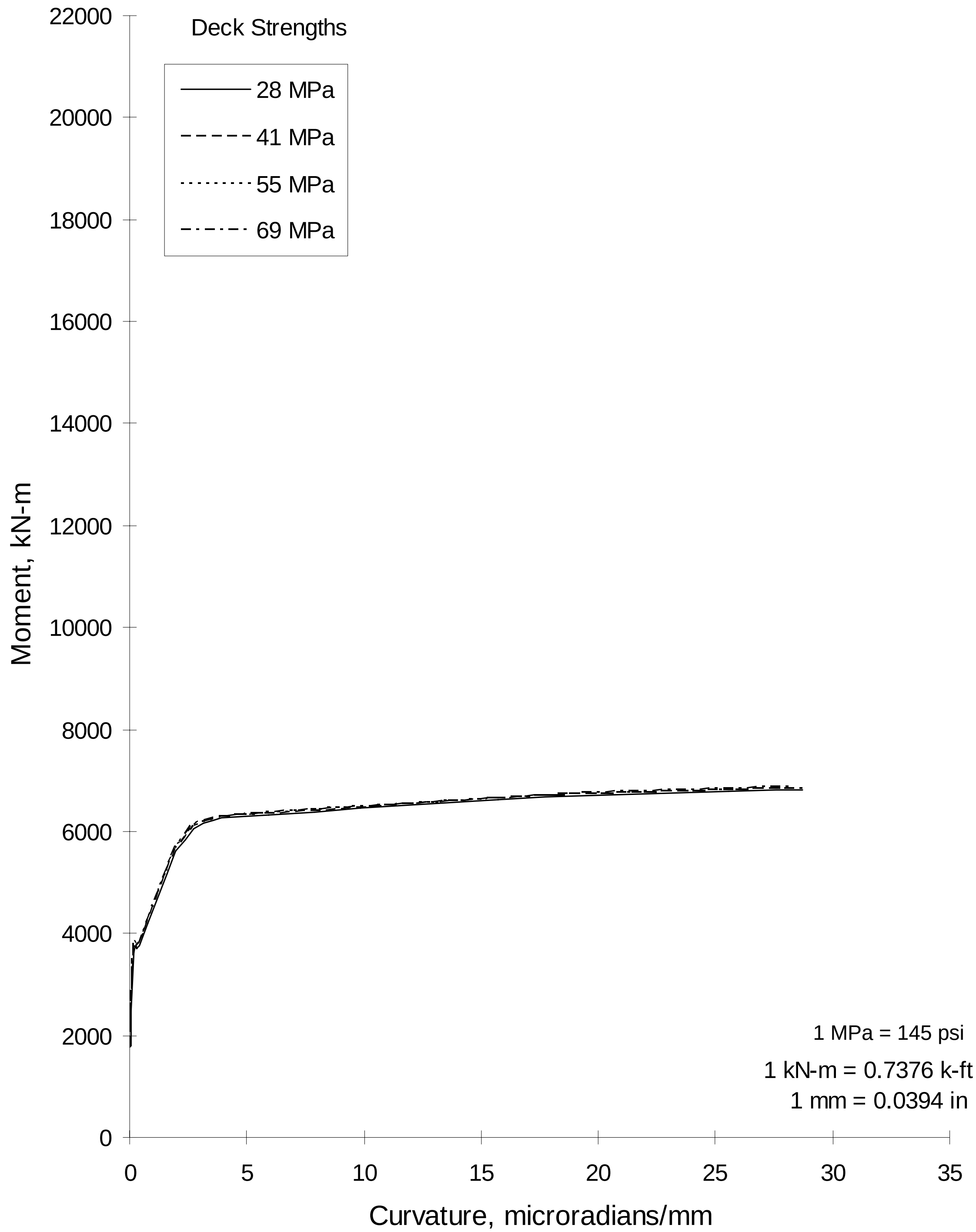

Figure 19. Moment-curvature relationships for BT-72, 41 MPa at a span of 24.4 m.

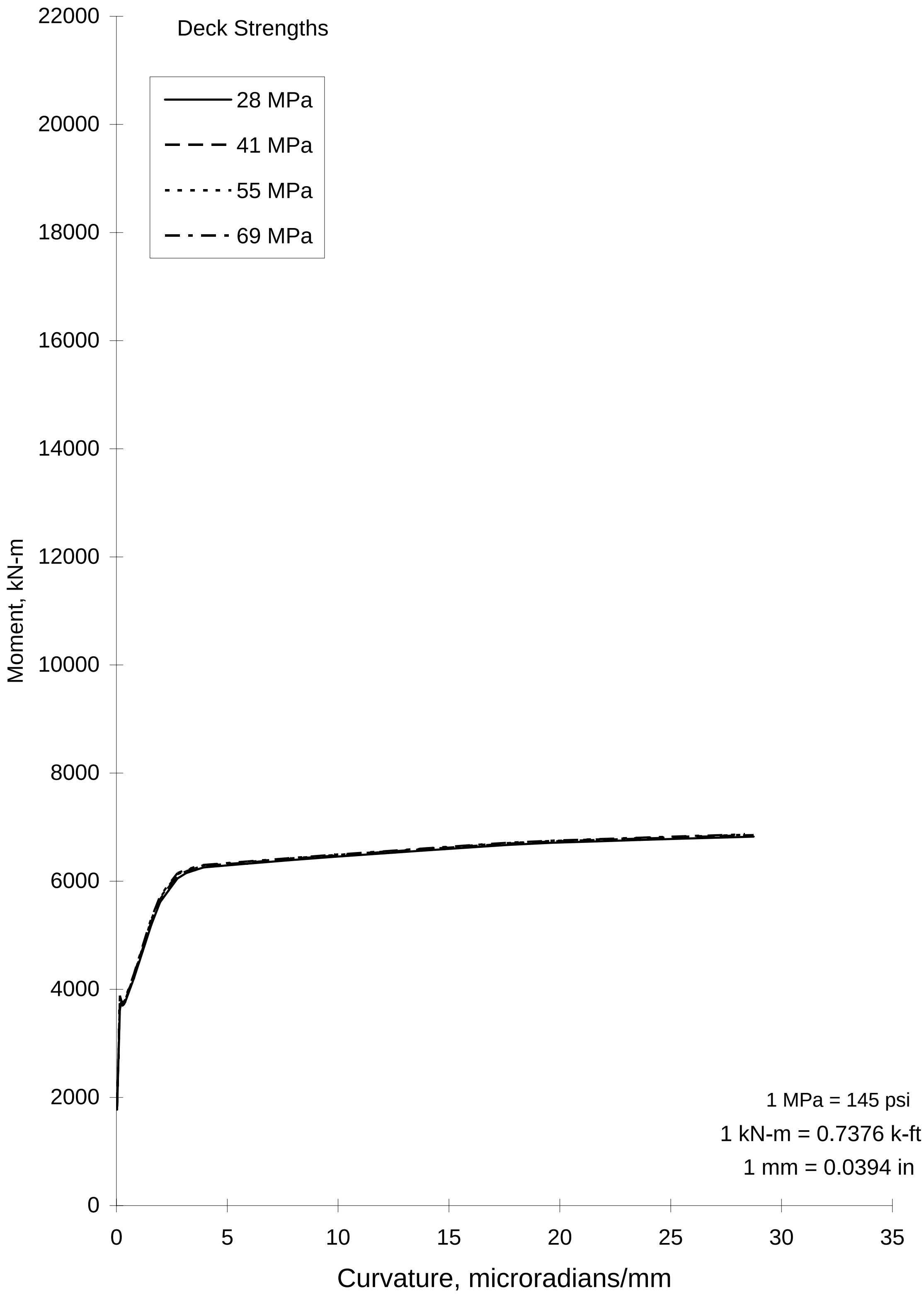

Figure 20. Moment-curvature relationships for BT-72, 83 MPa at a span of 24.4 m.

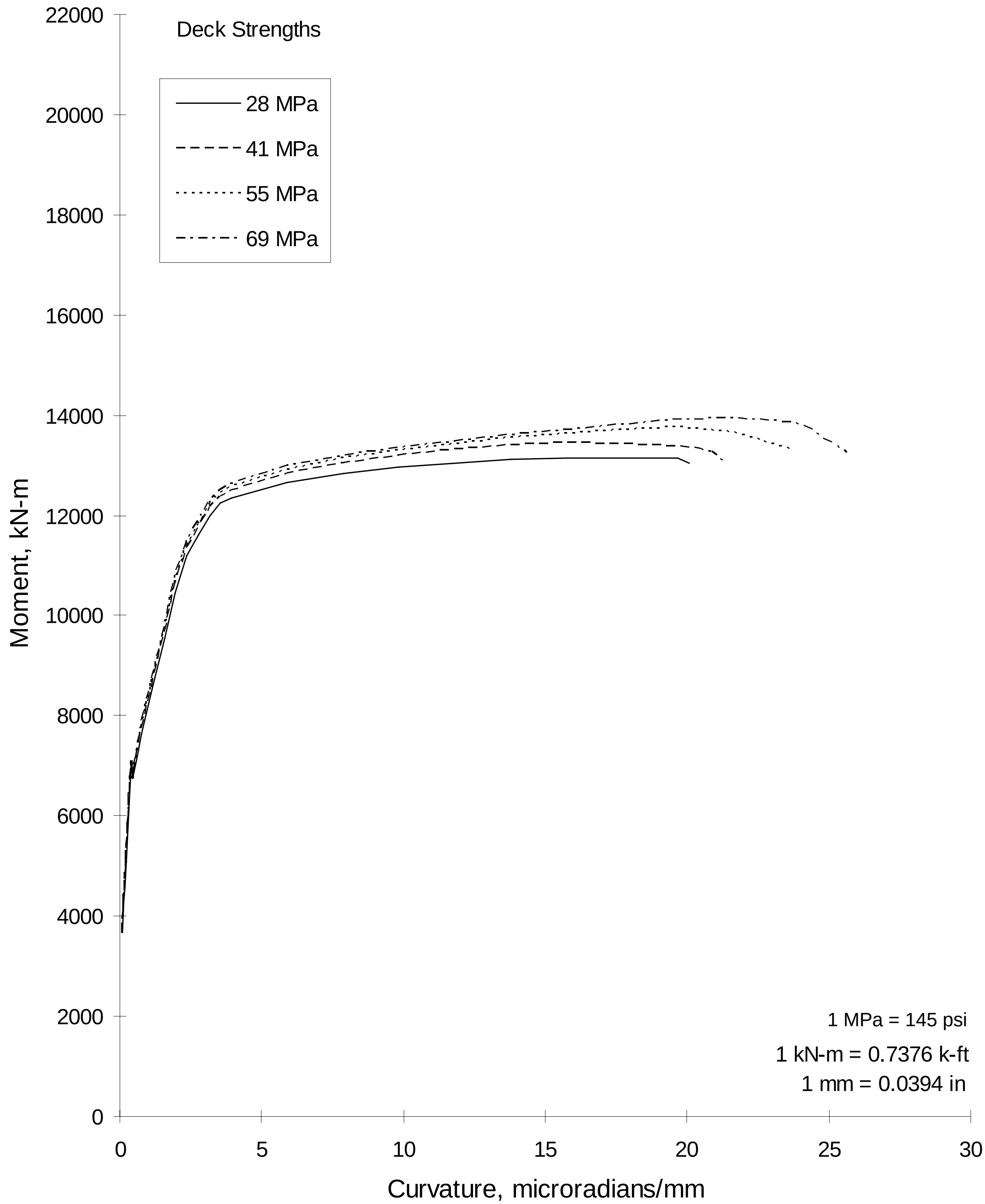

Figure 21. Moment-curvature relationships for BT-72, 41 MPa at a span of 44.5 m.

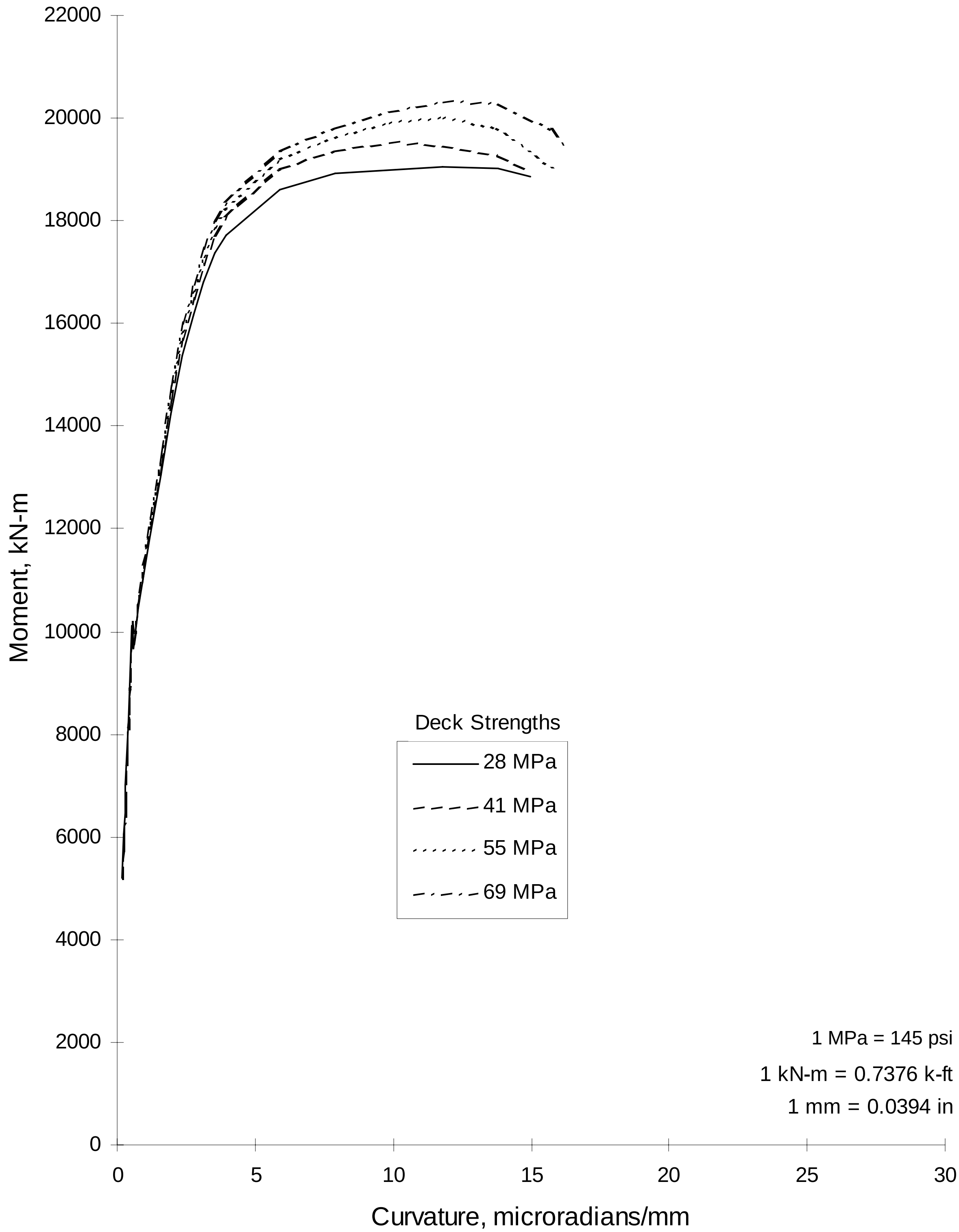

Figure 22. Moment-curvature relationships for BT-72, 83 MPa at a span of 53.3 m.

For the 44.5-m (146-ft) and 53.3-m (175-ft) spans, the use of higher-strength concrete in the deck resulted in a slightly higher flexural capacity and a small increase in the final curvature. For the 44.5-m (146-ft) span girders, the final curvature was six to eight times the curvature at yield of the lower layer of strands. This indicated that the sections achieve adequate ductility even though fracture of the strand was not achieved at maximum moment. For the 53.3-m (175-ft) spans, the final curvature was about four times the curvature at yield of the lower layer of strands. This ductility is less than was obtained with the shorter span lengths but one that still corresponds to a very large deflection which would give adequate visual warning of impending failure.

The strand stresses, deck strains, and girder strains at maximum moment are tabulated in tables 12 and 13. The strand stress is the stress in the lower layer of strands. The deck strain is the strain at the top surface of the deck. The girder strain is the strain at the top surface of the girder. This table shows that for series A and B, the strand stress reached the breaking strength of the strand while the maximum deck strain was 2,160 millionths, which is only slightly greater than the assumed strain at peak stress for the 28 MPa (4,000 psi) concrete. For the other strength concretes, the deck strains are less than the strains at peak stress. Also for series A and B sections, the depth of the neutral axis was less than the deck thickness so that the strain at the top of the girder was tensile and sufficient to cause cracking. This is denoted by the letter "T" in the last column of the table. Series A and B sections had a flange width of 2.44 m (96 inches), so there was sufficient concrete area to allow development of the compression force for the flexural resistance.

Table 12. Calculated stresses and strains at maximum moment (SI units).

Series	Girder Strength (MPa)	Deck Strength (MPa)	Span (m)	Strand Stress (GPa)	Deck Strain (millionths)	Girder Strain* (millionths)
A	41	28	24.4	1.90	2,160	T
	41	41	24.4	1.90	1,720	T
	41	55	24.4	1.90	1,540	T
	41	69	24.4	1.90	1,430	T
B	83	28	24.4	1.90	2,160	T
	83	41	24.4	1.90	1,720	T
	83	55	24.4	1.90	1,550	T
	83	69	24.4	1.90	1,430	T
C	41	28	44.5	1.84	4,250	1,340
	41	41	44.5	1.83	2,900	360
	41	55	44.5	1.86	3,020	T
	41	69	44.5	1.87	2,740	T
D	83	28	53.3	1.80	3,640	2,025
	83	41	53.3	1.79	2,560	1,320
	83	55	53.3	1.81	2,710	1,090
	83	69	53.3	1.80	2,410	800

* T denotes tensile strain sufficient to cause cracking.

Table 13. Calculated stresses and strains at maximum moment (English units).

Series	Girder Strength (psi)	Deck Strength (psi)	Span (ft)	Strand Stress (ksi)	Deck Strain (millionths)	Girder Strain* (millionths)
A	6,000	4,000	80	275	2,160	T
	6,000	6,000	80	275	1,720	T
	6,000	8,000	80	275	1,540	T
	6,000	10,000	80	275	1,430	T
B	12,000	4,000	80	275	2,160	T
	12,000	6,000	80	275	1,720	T
	12,000	8,000	80	275	1,550	T
	12,000	10,000	80	275	1,430	T
C	6,000	4,000	146	267	4,250	1,340
	6,000	6,000	146	266	2,900	360
	6,000	8,000	146	270	3,020	T
	6,000	10,000	146	271	2,740	T
D	12,000	4,000	175	261	3,640	2,025
	12,000	6,000	175	260	2,560	1,320
	12,000	8,000	175	262	2,710	1,090
	12,000	10,000	175	262	2,410	800

* T denotes tensile strain sufficient to cause cracking.

Series C and D cross sections contained 41 and 76 strands, respectively, and had a flange width of 1.38 m (54 inches). Consequently, the demands on the deck for the development of the compressive force were much higher than in series A and B. Flexural strength of the section was limited by the strain capacity of the deck concrete for all deck concrete strength levels. For all strengths of deck concrete, the deck concrete strains at maximum moment exceeded the strains at peak stress as shown in figure 17. For the sections with 28-MPa (4,000-psi) deck concrete, the strains exceeded the normally assumed limit of 3,000 millionths (0.003 strain). To maintain the deck strains below 3,000 millionths, a concrete deck strength of at least 41 MPa (6,000 psi) is needed. It should also be noted that with one exception, all of the cross sections in series C and D have reinforcement indices that permit the flexural design to be based on yielding of the steel. The exception is the 83/28-MPa (12,000/4,000-psi) combination of girder and deck strengths. For this section, the strain in the girder is close to the strain at peak stress for the 83-MPa (12,000-psi) girder concrete. This is a combination that should be avoided. Based on the analyses, it appears that there should be a limit on the difference between the girder concrete strengths and the deck concrete strength. However, the limited scope of this investigation does not permit the development of a rationale analysis. As an interim measure, it is proposed that the specified deck concrete strength should be at least 60 percent of the specified girder concrete strength at 28 days when the specified girder concrete strength exceeds 41 MPa (6,000 psi).

FLEXURAL STRENGTH

The flexural strengths for each of the sections analyzed in task 2 are tabulated in tables 14 and 15. The required strength and design strength were calculated using the computer program BRIDGE in task 1. The required strength was based on the following equation:

$$M = 1.3\ (\textit{Dead Load Moment} + 1.67\ (\textit{Live Load} + \textit{Impact Moment})) \qquad (13)$$

The design strength was calculated through an iteration process.(17) The number in the first column of nominal strengths was based on the design strength divided by a strength reduction factor of 0.9. The second column of nominal strengths were calculated using the AASHTO provisions.(16) The calculated strengths represent the maximum moments determined by the program BEAM BUSTER and are assumed to represent the real strengths of the sections. For all sections analyzed, the design strength exceeded the required strength and the calculated strength was greater than the nominal strength calculated by two different methods. It should also be noted that the nominal strengths provided an excellent prediction of the calculated strengths. Comparisons of required strengths with calculated strengths are shown in figures 19 through 22.

Table 14. Calculated flexural strengths (SI units).

Series	Girder Strength (MPa)	Deck Strength (MPa)	Span (m)	Moments (kN·m)				
				Required Strength	Design Strength*	Nominal Strength*	Nominal Strength**	Calculated Strength***
A	41	28	24.4	6,060	6,100	6,780	6,640	6,820
	41	41	24.4	6,060	6,100	6,780	6,700	6,860
	41	55	24.4	6,060	6,100	6,780	6,720	6,860
	41	69	24.4	6,060	6,100	6,780	6,740	6,960
B	83	28	24.4	6,060	6,100	6,780	6,640	6,820
	83	41	24.4	6,060	6,100	6,780	6,700	6,860
	83	55	24.4	6,060	6,100	6,780	6,720	6,860
	83	69	24.4	6,060	6,100	6,780	6,740	6,870
C	41	28	44.5	9,720	11,730	13,030	12,950	13,150
	41	41	44.5	9,720	12,120	13,463	13,330	13,440
	41	55	44.5	9,720	12,370	13,734	13,520	13,750
	41	69	44.5	9,720	12,550	13,951	13,670	13,920
D	83	28	53.3	13,600	16,970	18,660	18,240	19,040
	83	41	53.3	13,600	17,490	19,430	19,160	19,520
	83	55	53.3	13,600	17,960	19,950	19,770	19,980
	83	69	53.3	13,600	18,330	20,360	20,240	20,310

* Calculated by the program BRIDGE.

** Calculated per AASHTO.

*** Calculated by the program BEAM BUSTER.

Table 15. Calculated flexural strengths (English units).

Series	Girder Strength (psi)	Deck Strength (psi)	Span (ft)	Moments (ft-kip)				
				Required Strength	Design Strength*	Nominal Strength*	Nominal Strength**	Calculated Strength***
A	6,000	4,000	80	4,470	4,500	5,000	4,900	5,030
	6,000	6,000	80	4,470	4,500	5,000	4,940	5,060
	6,000	8,000	80	4,470	4,500	5,000	4,960	5,060
	6,000	10,000	80	4,470	4,500	5,000	4,970	5,060
B	12,000	4,000	80	4,470	4,500	5,000	4,900	5,030
	12,000	6,000	80	4,470	4,500	5,000	4,940	5,060
	12,000	8,000	80	4,470	4,500	5,000	4,960	5,060
	12,000	10,000	80	4,470	4,500	5,000	4,970	5,070
C	6,000	4,000	175	7,170	8,650	9,610	9,550	9,700
	6,000	6,000	175	7,170	8,940	9,930	9,830	9,910
	6,000	8,000	175	7,170	9,120	10,130	9,970	10,140
	6,000	10,000	175	7,170	9,260	10,290	10,080	10,270
D	12,000	4,000	175	10,030	12,520	13,910	13,450	14,040
	12,000	6,000	175	10,030	12,900	14,330	14,130	14,400
	12,000	8,000	175	10,030	13,250	14,720	14,580	14,740
	12,000	10,000	175	10,030	13,520	15,020	14,930	14,980

* Calculated by the program BRIDGE.

** Calculated per AASHTO.

*** Calculated by the program BEAM BUSTER.

TASK 2 CONCLUSIONS

Based on the task 2 analyses, the following conclusions are made:

- For span lengths of 24.4 m (80 ft) with lower amounts of prestressing strands, the use of high-strength concrete did not affect the flexural strengths.
- At the maximum span lengths for each girder concrete strength, the use of high-strength concrete in the decks had a slight effect in increasing the design and nominal strengths. The use of high-strength concrete also increased the flexural ductility of the cross sections.
- To ensure that the flexural strength is based on yielding of the reinforcement and to limit the strains in the deck, a minimum deck concrete strength of 41 MPa (6,000 psi) is recommended for span lengths in excess of 24.4 m (80 ft) when concrete girder strength exceeds 41 MPa (6,000 psi). Until further analyses can be performed, the specified deck concrete strength should be at least 60 percent of the specified girder concrete strength at 28 days when the specified girder concrete strength exceeds 41 MPa (6,000 psi).
- The applicability of the AASHTO specification for flexural design should be examined with respect to the use of higher-strength concretes and larger differential strengths between the deck and girder concretes.[16]

CHAPTER 4. TASK 3: ANALYSES OF PRESTRESS LOSSES AND LONG-TERM DEFLECTIONS

RESEARCH APPROACH

Analyses to determine the effect of high-performance concrete on prestress losses and long-term deflections were performed using a computer program known as PBEAM.[29] The program PBEAM is capable of analyzing composite prestressed concrete structures of any cross sectional shape having one axis of symmetry. The program accounts for the effects of nonlinearity of stress-strain response of materials and their variations of strength, stiffness, creep, and shrinkage of concrete, and relaxation of steel with time. A step-by-step method is used in the time-dependent analysis, and a tangent stiffness method is implemented for solving nonlinear response.

Precast, prestressed bridge girders with composite cast-in-place decks are modeled using a discrete element method. Element deformations and forces are estimated by analyzing stress-strain relationships of a series of rectangular fibers distributed over the depth of a cross section. Strain in each fiber is assumed to be constant at the centroidal axis of the fiber, and strain distribution varies linearly through the depth of a section. For each time step, the equilibrium at each element is maintained by determining the time-dependent stress corresponding to the level of strain in each fiber. The stress multiplied by area is summed over all fibers and force equilibrium is checked. If necessary, the strain distribution is adjusted and the process is repeated until all forces balance. A more detailed description of the program PBEAM and its verification against experimental data are given in references 29 and 30.

The following assumptions were utilized in the program PBEAM:

- Girders are simply supported.
- Calculations are based on a typical interior girder.
- Release of the prestressing strands occurs at an age of one day in several increments. Dead load is added at each increment.
- Concrete deck is cast in place and is cast when the girder is 83 days old. At age 90 days, the concrete deck acts compositely with the girder. Deck formwork is considered to be supported on the girder.
- Strands are low relaxation Grade 270 with a 12.7 m (0.5 inches) diameter spaced at 51-mm (2-inch) centers. Minimum cover to center of strands is 51 mm (2 inches).
- Girder cross section is a BT-72. Material properties are varied according to the discussion in section 4.2.

Analyses were performed for the following variables:

- Girder concrete compressive strength: 41, 55, 69, and 83 MPa (6,000, 8,000, 10,000, and 12,000 psi, respectively).
- Deck concrete compressive strength: 28, 41, 55, and 69 MPa (4,000, 6,000, 8,000 and 10,000 psi, respectively).
- Span lengths: 24.4, 44.5, and 53.3 m (80, 146, and 175 ft, respectively).

The combination of variables are defined in tables 16 and 17. Cross sections of the girders are shown in figure 23. Series A through D represent a complete parametric study of girder concrete strength and deck concrete strength for constant cross section and span length. Series E is an investigation of span lengths for a constant concrete strength. Design of the cross sections for series A and E were based on the analyses performed in task 1.

Table 16. Task 3 variables (SI units).

Series	Girder Strength (MPa)	Deck Strength (MPa)	Span (m)	No. of Strands*
A	41	28	44.5	41
	41	41	44.5	41
	41	55	44.5	41
	41	69	44.5	41
B	55	28	44.5	41
	55	41	44.5	41
	55	55	44.5	41
	55	69	44.5	41
C	69	28	44.5	41
	69	41	44.5	41
	69	55	44.5	41
	69	69	44.5	41
D	83	28	44.5	41
	83	41	44.5	41
	83	55	44.5	41
	83	69	44.5	41
E	83	55	24.4	20
	83	55	44.5	77
	83	55	53.3	77

* For consistency between tasks, the odd number of strands calculated by the program BRIDGE in task 1 were retained in task 3.

Table 17. Task 3 variables (English units).

Series	Girder Strength (psi)	Deck Strength (psi)	Span (ft)	No. of Strands*
A	6,000 6,000 6,000 6,000	4,000 6,000 8,000 10,000	146 146 146 146	41 41 41 41
B	8,000 8,000 8,000 8,000	4,000 6,000 8,000 10,000	146 146 146 146	41 41 41 41
C	10,000 10,000 10,000 10,000	4,000 6,000 8,000 10,000	146 146 146 146	41 41 41 41
D	12,000 12,000 12,000 12,000	4,000 6,000 8,000 10,000	146 146 146 146	41 41 41 41
E	12,000 12,000 12,000	8,000 8,000 8,000	80 146 175	20 77 77

* For consistency between tasks, the odd number of strands calculated by the program BRIDGE in task 1 were retained in task 3.

To satisfy design stress conditions at the ends of the girders, every strand within the width of the web was draped upwards at the ends. The drape started at a distance of 30 percent of the span from the end of the girders. The center 40 percent of the span length had the strands at maximum and constant eccentricity.

MATERIAL PROPERTIES

The computer program PBEAM allows for a variety of inputs for material properties and also contains default values. Because the properties of high-performance concrete may be different from those used as the basis for the default properties, a study was made to select the most appropriate material properties for analysis. This study involved selecting appropriate properties for modulus of elasticity, shrinkage, and creep, and their variation with time.

Modulus of Elasticity

In task 2, complete stress-strain curves for various strengths of concrete were established. The slope of the ascending portion of the stress-strain curve is the modulus of elasticity. The following equations were utilized for calculation of the modulus:

For f'_c of 28 and 41 MPa (4,000 and 6,000 psi, respectively):

$$E_c = 0.043(w_c)^{1.5}\sqrt{f_c^{'}} \text{ in SI units} \qquad (14)$$

$$E_c = 33(w_c)^{1.5}\sqrt{f_c^{'}} \text{ in English units}^{(20)} \qquad (1)$$

For f'_c of 55, 69, and 83 MPa (8,000, 10,000, and 12,000 psi):

$$E_c = 0.0000339(w_c)^{2.5}\left(f_c^{'}\right)^{0.325} \text{ in SI units} \qquad (15)$$

$$E_c = (w_c)^{2.5}\left(f_c^{'}\right)^{0.325} \text{ in English units}^{(19)} \qquad (4)$$

For the girder concrete, the variation of compressive strength with time was determined from the following equation:

$$\frac{\left(f_c^{'}\right)_t}{\left(f_c^{'}\right)_{28}} = \frac{t}{0.346 + 0.988t} \qquad (16)$$

where

$(f'_c)_t$ = compressive strength at a concrete age of *t* days
$(f'_c)_{28}$ = compressive strength at a concrete age of 28 days

The above relationship was based on the recommendations of ACI 209 and corresponds to a compressive strength at 1 day equal to 75 percent of the compressive strength at 28 days.[31]

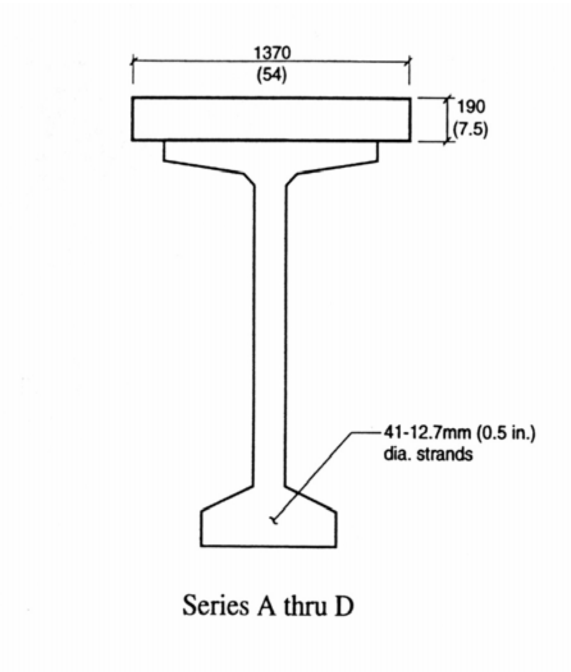

Figure 23 (part 1). Cross section of series A through D girders (BT-72) analyzed in task 3. All dimensions are in millimeters (inches).

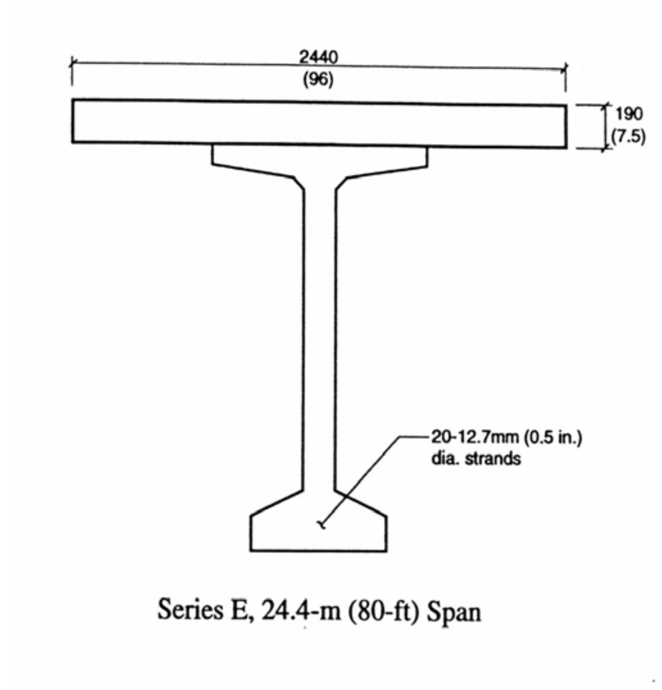

Figure 23 (part 2). Cross section of series E girder (BT-72), 24.4 m (80-ft) span, analyzed in task 3. All dimensions are in millimeters (inches).

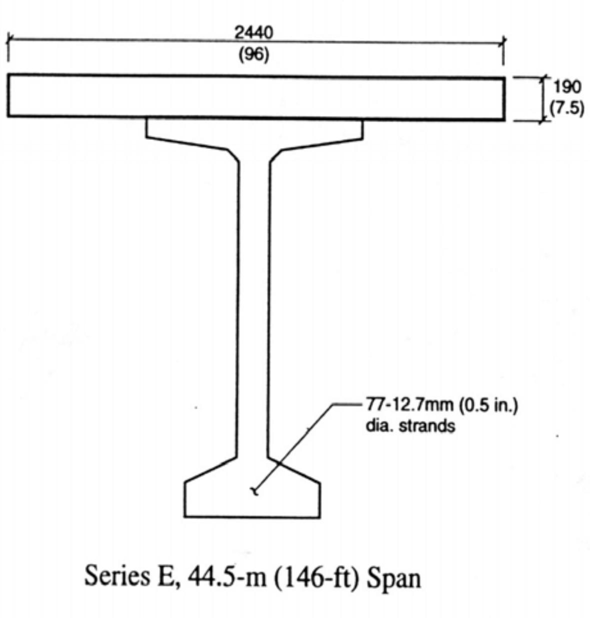

Figure 23 (part 3). Cross section of series E girder (BT-72), 44.5 m (146-ft) span, analyzed in task 3. All dimensions are in millimeters (inches).

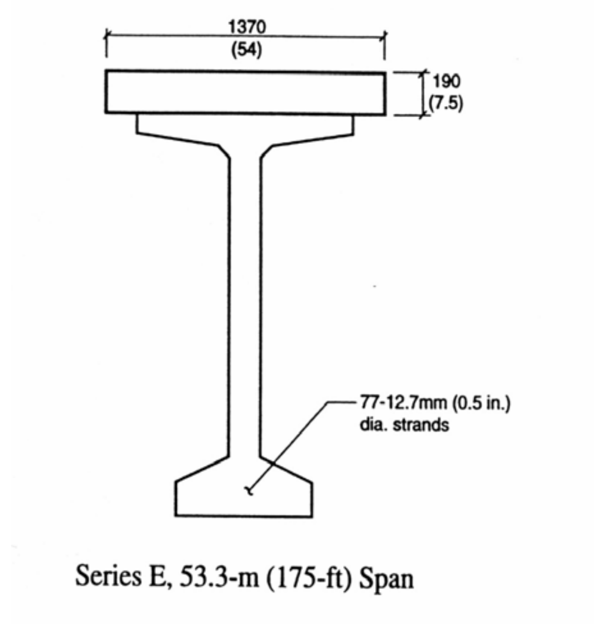

Figure 23 (part 4). Cross section of series E girder (BT-72), 53.3 m (175-ft) span, analyzed in task 3. All dimensions are in millimeters (inches).

For the deck concrete, the variation of compressive strength was assumed to be in accordance with ACI 209 as follows:[31]

$$\frac{\left(f_c^{'}\right)_t}{\left(f_c^{'}\right)_{28}} = \frac{t}{4.00 + 0.85t} \tag{17}$$

Equation 17 reflects a slower strength gain for moist, cured concrete compared with equation 16 which applies to rapid strength development.

Consequently, for a specified 28-day compressive strength, the compressive strength at any other age may be calculated. Using this value of compressive strength, the corresponding modulus of elasticity for the concrete can be determined. In this manner, the variation of modulus of elasticity with time can be calculated.

Shrinkage

Most research has indicated that the final shrinkage of high-strength concretes is of the same order of magnitude as that for lower strength concretes.[1] Consequently, the values proposed by ACI 209 were utilized in the program PBEAM analysis. ACI 209 recommends that, in the absence of specific creep and shrinkage data for local aggregates and conditions, an average value of 780 millionths be utilized for the shrinkage of a 153- by 305-mm (6- by 12-inch) cylinder exposed to drying at 40 percent relative humidity.[31] This value was then corrected for the effects of girder size and relative humidity in accordance with the procedures of ACI 209. An average mean annual relative humidity of 70 percent was taken as representing a large portion of the United States. Consequently, a relative humidity correction factor of 0.7 was applied. A size correction factor of 0.837 was also applied as representing a volume-to-surface-area ratio of 3.0 for a BT-72. These two correction factors resulted in a final shrinkage strain for the girder of 457 millionths. The shrinkage strain of the girder concrete was assumed to vary with time according to the following equation:[31]

$$\frac{\left(\varepsilon_{sh}\right)_t}{\left(\varepsilon_{sh}\right)_u} = \frac{t}{55 + t} \tag{18}$$

where

$(\varepsilon_{sh})_t$ = shrinkage at time t
$(\varepsilon_{sh})_u$ = final shrinkage strain

For the concrete in the deck, a relative humidity correction factor of 0.7 was applied along with a size correction for a 190-mm (7.5-inch) thick deck of 0.77, resulting in a final shrinkage of 420 millionths. The deck shrinkage was assumed to vary with time according to the following equation:[31]

$$\frac{\left(\varepsilon_{sh}\right)_t}{\left(\varepsilon_{sh}\right)_u} = \frac{t}{35 + t} \tag{19}$$

It is possible that, with the higher-strength concretes and the use of fly ash or silica fume to obtain the strengths, the concrete may take longer to dry out than the lower strength concretes. Consequently, the assumed variation of shrinkage with time may not truly reflect actual behavior. However, a lack of data for steam-cured, high-strength concretes precluded the determination of an alternative equation.

Creep of Girder Concrete

Creep of concrete can be expressed in terms of creep coefficients or specific creep. The creep coefficient is the ratio of creep strain to the initial strain at loading. For most concretes, the values vary between 1.30 and 4.15. Specific creep is defined as creep strain per unit stress and varies between 15 and 220 millionths/MPa (0.1 and 1.5 millionths per psi). The relationship between creep coefficient and specific creep is as follows:

Creep coefficient = specific creep × modulus of elasticity at age of loading

The computer program PBEAM allows the input of creep as a creep coefficient. However, for purposes of selecting appropriate creep values for use in the analyses, the following discussion is based on specific creep.

Specific creep data for 153- by 305-mm (6- by 12-inch) cylinders published by several authors are shown in figure 24. These data have been obtained for a variety of concrete constituent materials, cured under different conditions, loaded at different ages, and maintained under constant load for different lengths of time. To partially eliminate the variable associated with the length of time under load, the published data were corrected to final values based on variations of creep with time following the equations listed above. A plot of the same data including this correction factor is shown in figure 25. All of the data are for cylinders maintained at 50 percent relative humidity while under load. A comparison with the predicted values according to ACI 209 for 50 percent relative humidity is also included in figure 25. This curve is very close to the best fit for all data.

The solid symbols shown in figure 25 are for concrete specimens obtained by steam curing.[14,30,32] Since it is anticipated that high-strength concrete prestressed girders will either be produced by steam curing or will achieve relative high temperatures from heat of hydration, the effects of curing temperatures on the properties of concrete are important. Hanson indicated that the effect of atmospheric steam curing was to reduce the creep of concrete cylinders containing type I cement by 20–30 percent and that of concretes containing type III cements by 30–40 percent below that of the same concretes moist cured for 6 days[32] It is also apparent from figure 25 that the reduction in specific creep as compressive strength increases is more rapid with the steam-cured concretes than with the moist-cured concretes. Figure 25 shows a best-fit curve to the data for steam-cured concretes alone. This curve indicates a very rapid change in the specific creep as the concrete compressive strength increases. However, no data are available for concrete compressive strengths above 69 MPa (10,000 psi), so the validity of the extrapolation beyond 69 MPa (10,000 psi) is questionable. Consequently, in the PBEAM analyses, a variation of specific creep with concrete compressive strength was selected that lay between the ACI 209 values and that for steam cured concrete alone. This line is labeled in figure 25 as PBEAM.

Since most of the data in figure 25 represent concrete loaded at 28 days, this age was selected as the age for which the specific creep values would be selected. Values of specific creep and creep coefficient for 28-day age of loading at 50 percent relative humidity and a volume-to-surface ratio of 1.5 are listed in table 21. These data were then corrected using the procedures of ACI 209 for a relative humidity of 70 percent, a volume-to-surface ratio of 3.0 corresponding to a BT-72, and a loading age of 1 day. The corrected calculated creep coefficients for each concrete strength are tabulated in table 18. These values were used in the PBEAM analyses.

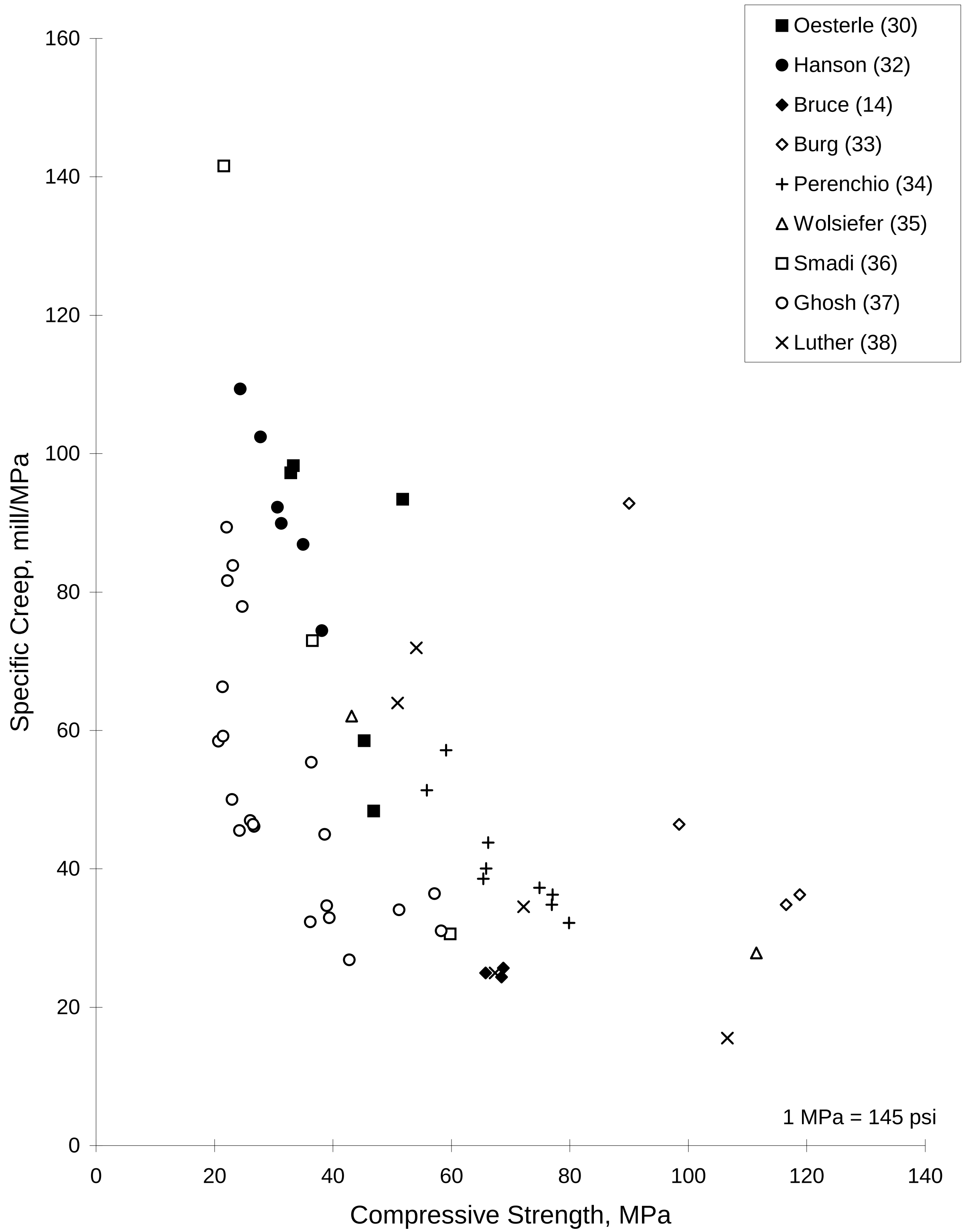

Figure 24. Variation of specific creep with compressive strength as published.

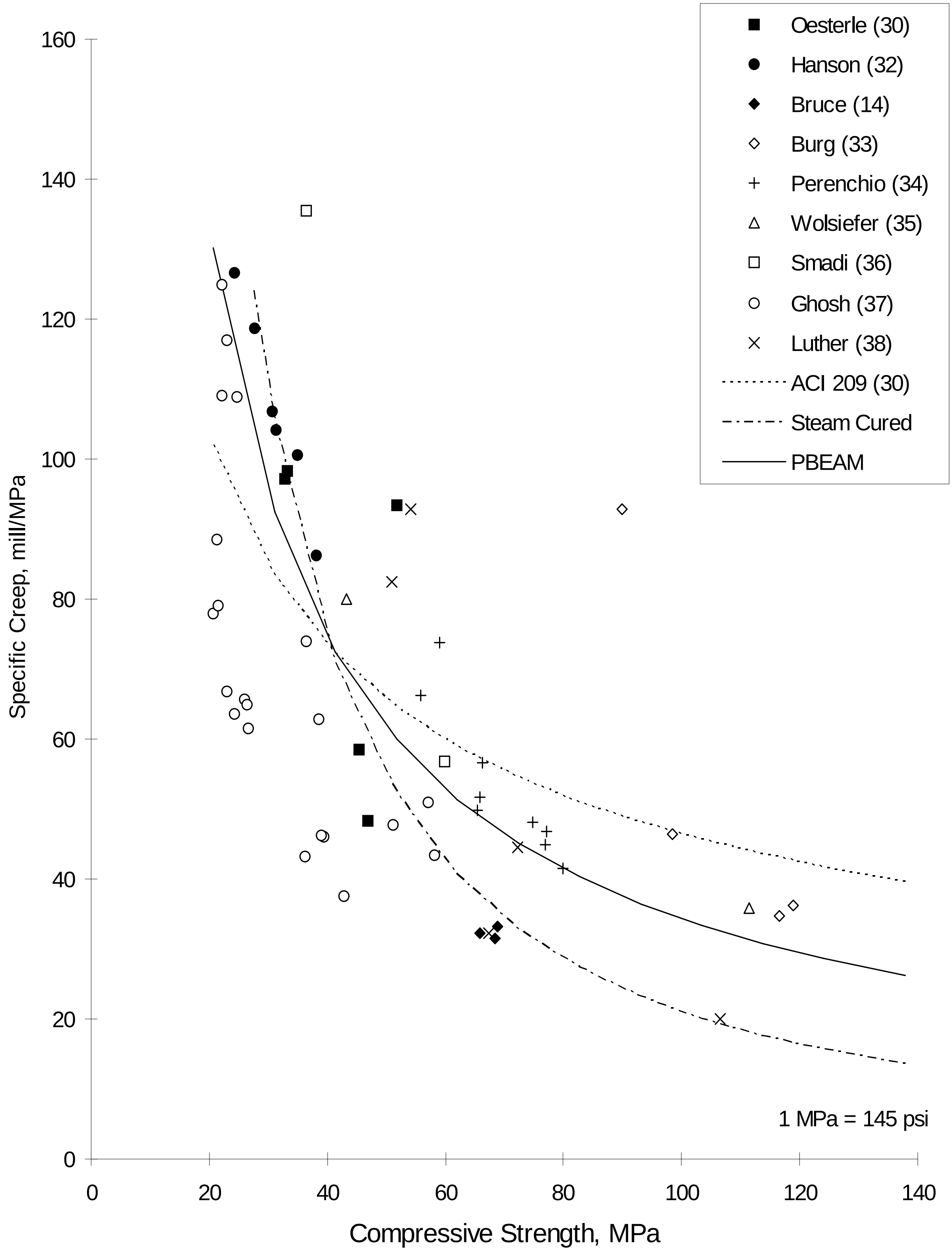

Figure 25. Variation of ultimate specific creep with compressive strength.

Table 18. Values of creep used in PBEAM.

			Specific Creep	Creep Coefficient	
28-Day Compressive Strength	**Unit Weight**	**28-Day Modulus of Elasticity**	**Loading Age = 28 Days**		
			RH = 50% V/S = 1.5	**RH = 50% V/S = 1.5**	**RH = 70% V/S = 3.0**
MPa	kg/m^3	GPa	millionths/MPa		
41	2,370	31.7	72.5	2.30	1.95
55	2,420	35.9	56.9	2.04	1.73
69	2,480	39.1	47.0	1.93	1.63
83	2,500	44.3	40.3	1.79	1.51
psi	lb/ft^3	10^6 psi	millionths/psi		
6,000	148	4.60	0.500	2.30	1.95
8,000	151	5.20	0.392	2.04	1.73
10,000	155	5.97	0.324	1.93	1.63
12,000	156	6.43	0.278	1.79	1.51

The variation of creep with time was assumed to be in accordance with the following equation by ACI 209:

$$\upsilon_t = \frac{t^{0.60}}{10 + t^{0.60}} \upsilon_u \tag{20}$$

where

υ_t = creep at time t

υ_u = final value of creep

t = number of days under load

The effect of age of loading was also assumed to be in accordance with ACI 209 as follows:

$$\gamma_{la} = 1.3 (t_{la})^{-0.094} \tag{21}$$

where

$\gamma_{\ell a}$ = correction factor for age of loading

$t_{\ell a}$ = age of concrete at loading

The resulting relationship between specific creep and age for different strength concretes and two ages of loading are shown in figure 26.

Creep of Deck Concrete

Since the equations utilized by ACI for creep coefficient represented a good fit for the data shown in figure 25, it was decided to use the ACI 209 values for the creep properties of the concrete used in the deck. The calculated creep coefficient for a deck with a thickness of 190 m (7.5 inches) was 1.44.

Steel Relaxation

Since steel relaxation was not a primary parameter in the evaluation, the default values contained within PBEAM were utilized. These are based on the PCI recommendations.[39]

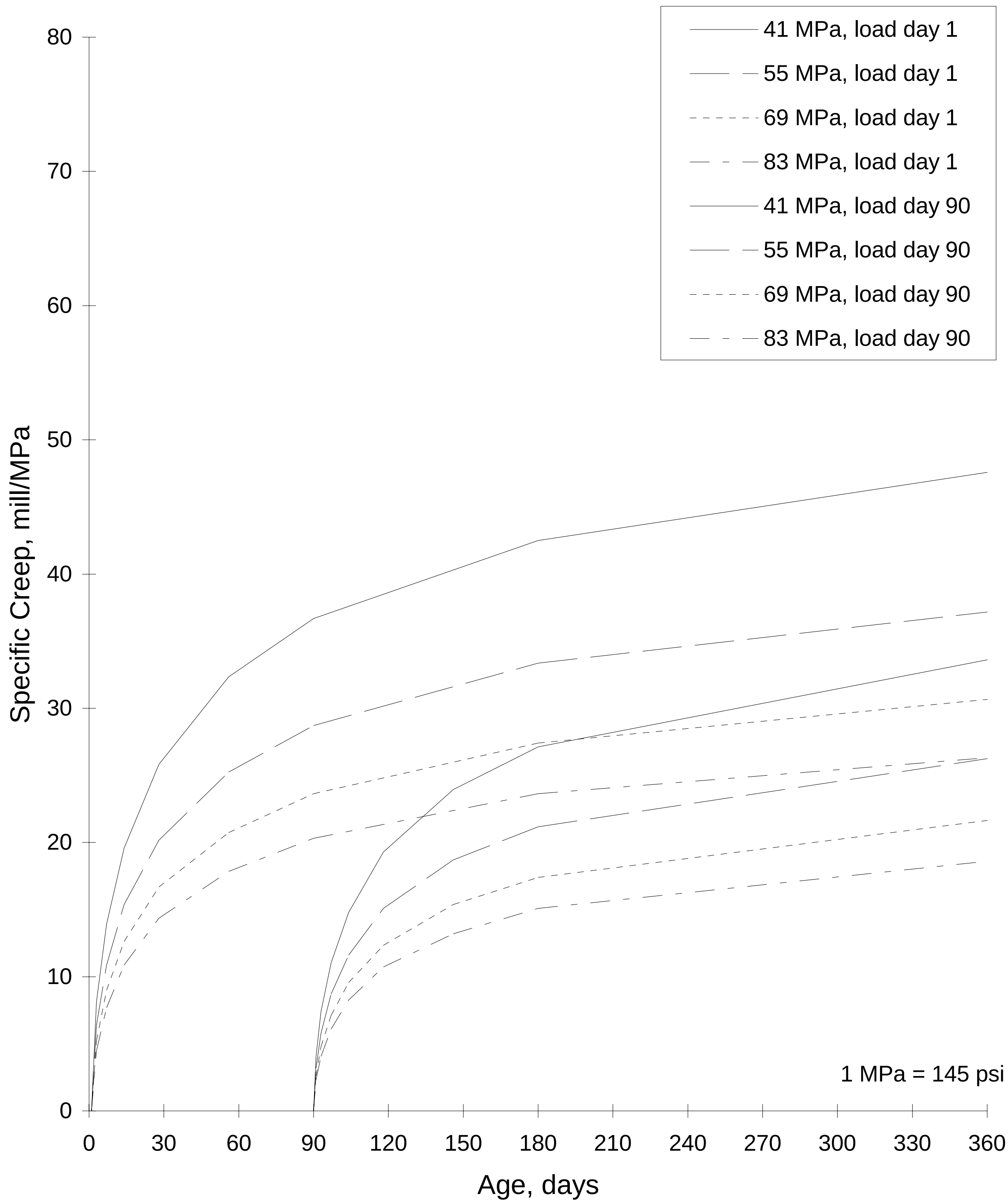

Figure 26. Variation of specific creep with age.

PRESTRESS LOSSES

The variation of prestressing strand stress with time for the bottom layer of strands for two BT-72 girders with concrete compressive strengths of 41 and 83 MPa (6,000 and 12,000 psi, respectively) and a span of 44.5 m (146 ft) is shown in figure 27. Each curve consists of four stages. The first stage comprises the initial elastic shortening caused by release of the prestressing. In PBEAM, this is accomplished by applying the prestressing force in a series of stages corresponding to the addition of the girder dead load. The force is applied in increments to prevent cracking of the concrete. The second stage of the curves consists of prestress losses between the time of release and the time when the deck is cast on the girder. The third stage is the elastic change in stress caused by application of the dead load of the deck concrete to the girder cross section. The fourth and final stage of the curve consists of losses in strand stresses as the composite girder is loaded by the dead load of the deck and girder. For this analysis, no additional dead load was assumed after the deck was added. The general shape of the curve was the same for all span lengths and concrete strengths analyzed.

At each level of girder concrete compressive strength, the variation of deck concrete compressive strength did not have any effect on prestress losses. This occurs because the deck does not become an effective part of the composite section until the fourth stage of each curve. At the beginning of the fourth stage, the compressive stress in the deck is zero. The only increase in compressive stress occurs as the concrete girder shortens and tries to shorten the deck with a corresponding force transferred into the deck. However, at the same time, the deck is also shrinking and this shrinkage is of the same order of magnitude as the shortening of the top flange of the girder. Consequently, there is very little transfer of force between the girder and the deck, and the deck does not have a significant impact on the prestress losses. It should be noted that all the analyses in this investigation were based on a composite section becoming effective at 90 days. It is possible that the effect of the deck concrete compressive strength may be greater for earlier ages of loading.

The variation of strand stress with time for the three girders containing 83-MPa (12,000-psi) concrete compressive strength and varying span lengths is shown in figure 28. It can be seen that the prestress losses varied with span length, the 44.5-m (146-ft) length having the largest total loss. This is consistent with the magnitude of stress at the level of the bottom layer of strands following release. For the girder with the span length of 44.5 m (146 ft), the concrete compressive stress at release was the highest of the three girders. Consequently, the elastic shortening and the creep shortening were also higher.

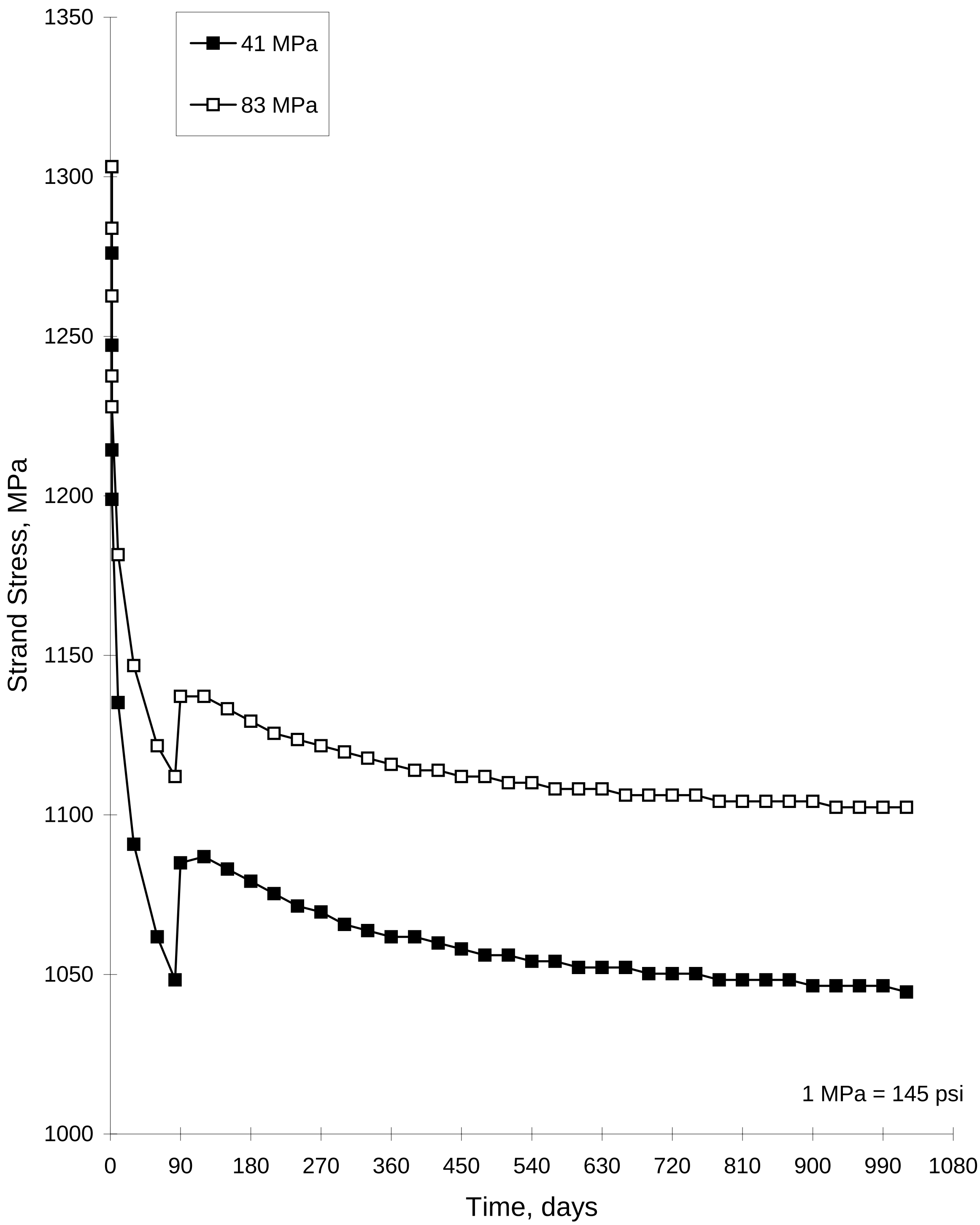

Figure 27. Prestressing strand stress versus time for varying girder concrete strength, 28-MPa deck strength, and 44.5-m span.

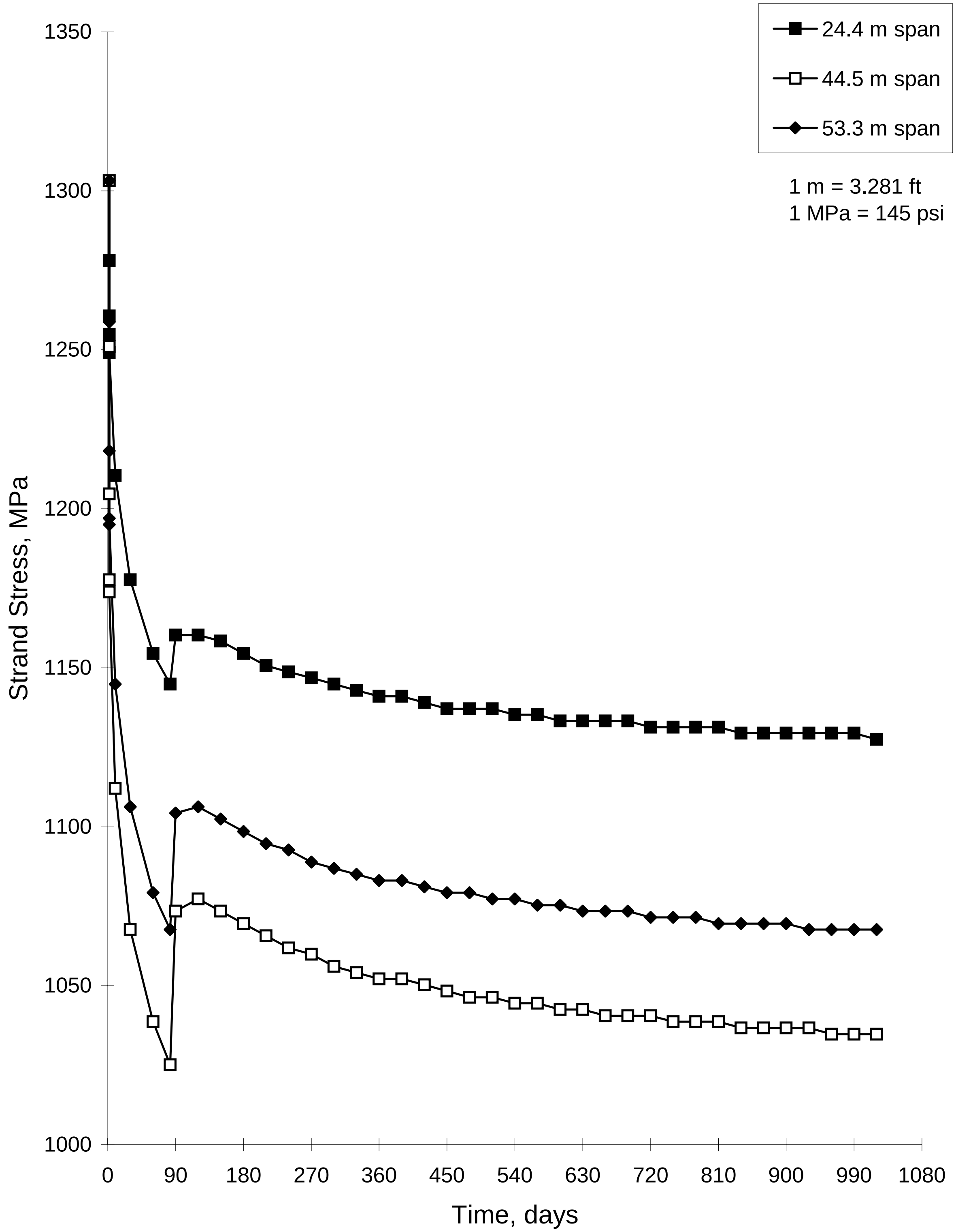

Figure 28. Prestressing strand stress versus time for 83-MPa girder concrete strength, 55-MPa deck strength, and varying spans.

The prestress losses determined for each level of girder concrete compressive strength are tabulated in tables 19 and 20. The tabulated losses are the calculated losses for the lower layer of prestressing strand in the girder cross section. The total losses are those determined from the program PBEAM at an age of 25 years starting with an initial stress of 1.30 GPa (189,000 psi) in the prestressing strand. The shrinkage stresses were calculated from the assumed shrinkage strain based on a modulus of elasticity of the prestressing strand of 193 GPa (28,000,000 psi). The elastic shortening at release corresponds to the prestress loss determined from the program PBEAM during application of the prestressing force. Because of the manner in which the analyses are performed, a small amount of relaxation is included in these stresses. The creep and relaxation losses represent the net difference between the total losses and the shrinkage and elastic losses. Because of the manner in which the program PBEAM calculates the interactive stresses from creep and relaxation, it is not possible to separate the two effects in the analysis. Consequently, they are listed together in tables 19 and 20. From the analyses, it can be seen that the direct substitution of a higher-strength concrete for one of lower strength reduces the prestress losses, part of the reduction being caused by the lower elastic losses and part by the lower creep losses. It may also be concluded from tables 19 and 20 that the magnitude of the total prestress losses will not be greater through the use of high-strength concrete in the girders and are likely to be less.

Prestress losses calculated according to AASHTO standard specifications are also shown in tables 19 and 20 for comparison with the losses calculated according to PBEAM.[16] In the current specifications, the creep losses are calculated based on the concrete stresses at the center of gravity of the prestressing steel. For purposes of comparison, the AASHTO losses shown in tables 19 and 20 were calculated using the procedure detailed in AASHTO specifications, but the stresses were calculated at the level of the bottom layer of prestressing steel. For the shorter span lengths, the AASHTO calculations show reasonable agreement with the PBEAM calculations. However, for the higher-strength concretes at the longer span lengths considerable deviation exists. For all calculations, the elastic losses compare favorably. However, AASHTO underestimates the losses caused by shrinkage and overestimates considerably the losses caused by creep. These data indicate that a revision of the AASHTO specification to take into account the different creep properties of the high-strength concretes is needed.

Table 19. Comparison of prestress losses (SI units).

Girder Strength (MPa)	Span (m)	Losses (MPa)				
		Elastic	Shrinkage	Creep	Relaxation	Total
		PBEAM				
41	44.5	104	88	119		311
55	44.5	92	88	99		279
69	44.5	81	88	85		254
83	44.5	75	88	76		239
83	24.4	54	88	60		202
83	44.5	130	88	116		334
83	53.3	106	88	93		287
		AASHTO				
41	44.5	98	45	126	16	285
55	44.5	83	45	128	17	173
69	44.5	72	45	130	19	266
83	44.5	66	45	131	19	261
83	24.4	45	45	91	23	204
83	44.5	112	45	212	10	379
83	33.3	92	45	181	14	332

Table 20. Comparison of prestress losses (English units).

Girder Strength (psi)	Span (ft)	Losses (ksi)				
		Elastic	Shrinkage	Creep	Relaxation	Total
		PBEAM				
6,000	146	15.1	12.8	17.2		45.1
8,000	146	13.4	12.8	14.4		40.6
10,000	146	11.8	12.8	12.4		37.0
12,000	146	10.9	12.8	11.0		34.7
12,000	80	7.8	12.8	8.7		29.3
12,000	146	18.8	12.8	16.8		48.4
12,000	175	15.4	12.8	13.5		41.7
		AASHTO				
6,000	146	14.2	6.5	18.2	2.3	41.2
8,000	146	12.1	6.5	18.6	2.5	39.7
10,000	146	10.5	6.5	18.8	2.7	38.5
12,000	146	9.6	6.5	19.0	2.8	37.9
12,000	80	6.5	6.5	13.2	3.4	29.6
12,000	146	16.2	6.5	30.7	1.5	54.9
12,000	175	13.4	6.5	26.2	2.0	48.1

LONG-TERM DEFLECTIONS

The variation of midspan deflection with time for four girders of varying girder concrete compressive strength and at a constant deck concrete strength of 28 MPa (4,000 psi) is shown in figure 29. These curves consist of four stages. The initial stage corresponds to an upward deflection at release of prestressing and includes the effects of prestressing and dead load of the girder. The second stage consists of continued upward deflection as a result of creep in the concrete. The third stage consists of a downward deflection caused by the dead load of the deck at the time the concrete deck is placed at 83 days. The fourth stage consists of further downward deflection as a result of creep and shrinkage followed by a period in which the deflections essentially level off. By age 1,000 days, the maximum net deflection was +6 mm (0.25 inch). These analyses indicate that very little change occurs after 180 days, consistent with results obtained by Bruce.[14]

The effect of deck concrete compressive strength on midspan deflection is shown in figure 30. This figure shows the concrete compressive strength of the deck had very little effect on the midspan deflections (which was true for all girder strength levels).

The variation of midspan deflection with time for the 83-MPa (12,000 psi) concrete girders with varying span lengths is shown in figure 31. The deflections of the 24.4-m (80-ft) girder are relatively small compared with the deflections of the girder for other span lengths. This results partly from the shorter span length but also from the relatively low number of strands (only 20).

The initial camber of the 44.5-m (146-ft) girder is similar to the camber of the girders shown in figure 29 for the same span length. A slight difference in camber occurs because of the different number of strands: 77 for the girder in figure 31 compared with 41 for the girders in figure 29. The downward deflection caused by casting the deck and the subsequent creep and shrinkage are larger for the girder shown in figure 31 compared with that in figure 29 because of the larger girder spacing. The net result for the girder shown in figure 31 is a downward deflection of approximately 20 mm (0.8 inch).

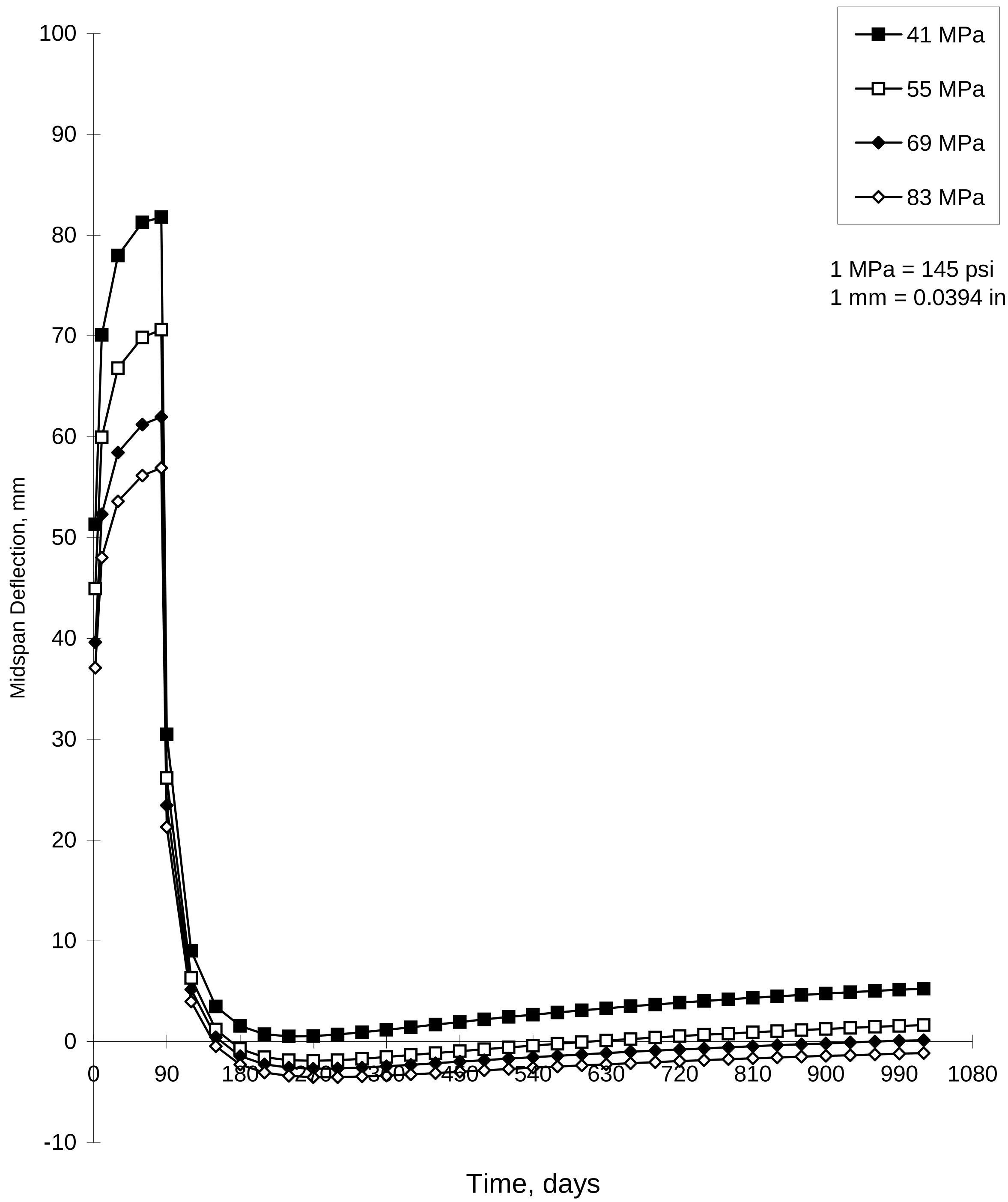

Figure 29. Midspan deflection versus time for varying girder concrete strengths, 28-MPa deck strength, and 44.5-m span.

The 53.3-m (175-ft) span girder shows a deflection pattern after release that is different from the other girders. For a short time, the girder creeps upwards but it then reverses direction. Following release of the prestress, the stress distribution across the depth of the girder is nearly constant. This is different from the other girders where the compressive stress in the bottom flange is always greater than the stress in the top flange. Following a small amount of prestress loss, the compressive stress in the top flange exceeds the stress in the bottom flange and the girder begins to creep downwards. A large deflection occurs when the deck is cast because of the long span. The final result is a downward deflection of approximately 90 mm (3.5 inches). This deflection is small compared with the span length (1 in 600) and could be compensated for by cambering the deck formwork. However, it indicates that there may be deflection considerations that could limit the span length for which high-strength concrete girders can be used.

TASK 3 CONCLUSIONS

Based on the task 3 analyses, the following conclusions are made:

- The use of high-strength concrete in the decks did not change the magnitude of the prestress losses.
- Prestress losses in high-strength concrete girders will generally be less than the losses in lower strength concrete girders.
- The current AASHTO procedure for calculations of prestress losses needs to be modified to account for the properties of high-strength concrete.
- The use of high-strength concrete in the decks did not affect the magnitude of the long-term deflections.
- The use of high-strength concrete in girders in place of lower strength concrete will result in less initial camber and similar long-term deflections for the same span lengths.
- There may be deflection requirements that limit the span lengths for which high-strength concrete girders can be used.

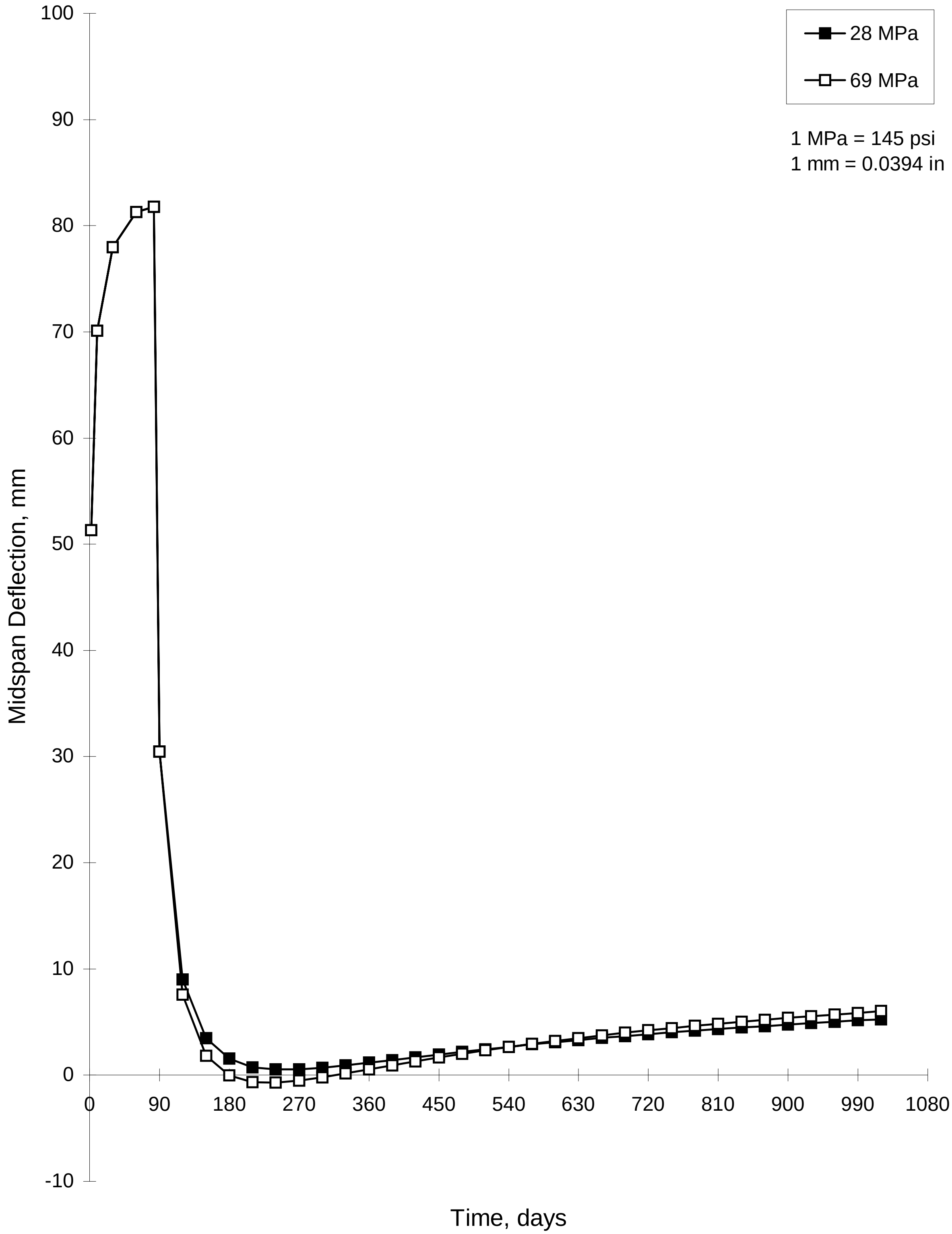

Figure 30. Midspan deflection versus time for 41-MPa girder concrete strength, varying deck concrete strengths, and 44.5-m span.

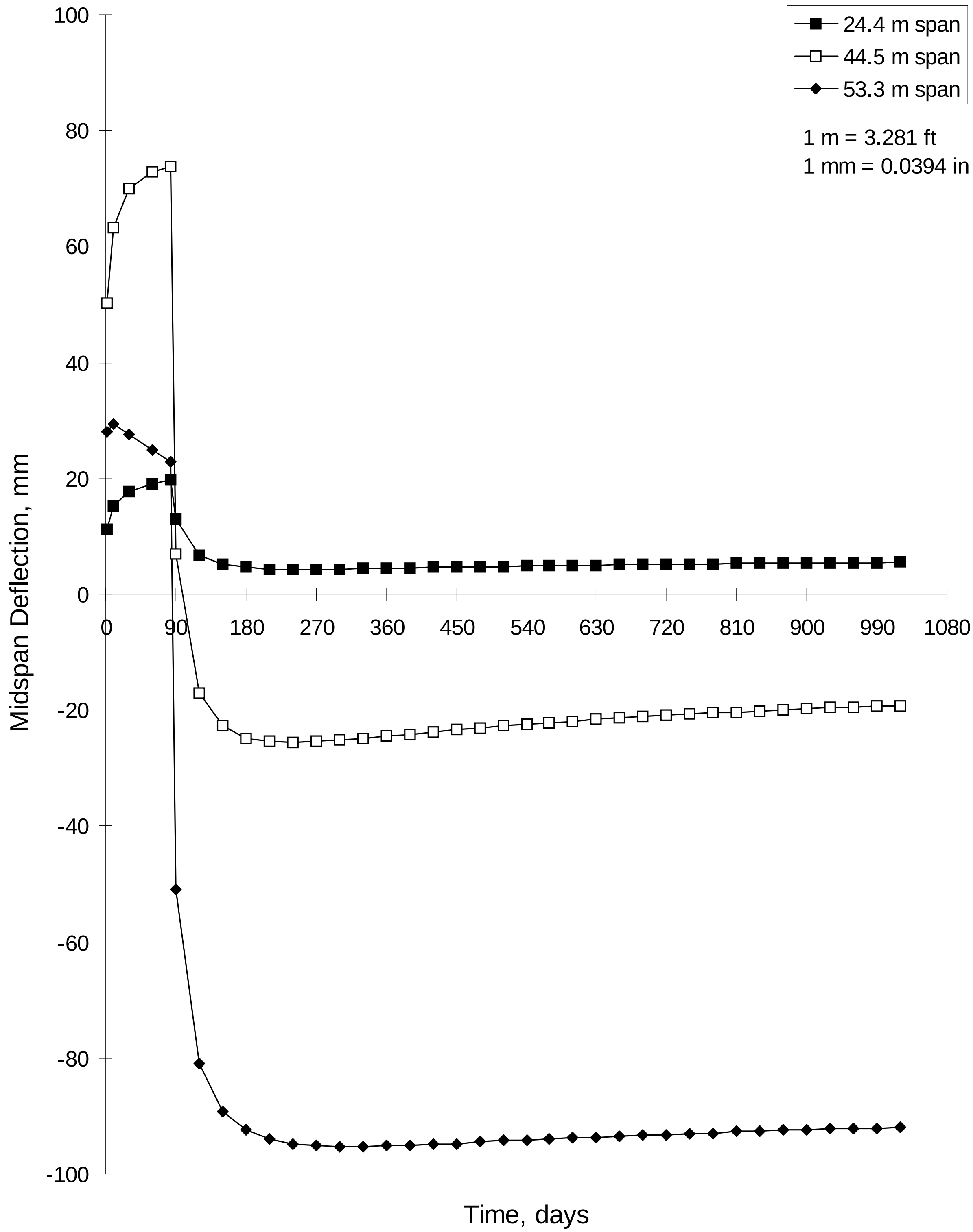

Figure 31. Midspan deflection versus time for 83-MPa girder concrete strength, 55-MPa deck strength, and varying spans.

CHAPTER 5. CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

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

Cost Analyses

- The use of high-strength concrete in bridge decks will not result in a reduction of deck thickness or in the amount of transverse reinforcement. Therefore, no corresponding savings will occur.
- The use of high-strength concrete in bridge decks allows for a slight increase in maximum span lengths of bulb-tee girders.
- An increase of 25 percent in the in-place cost of high-strength deck concrete will only increase the overall superstructure cost by 5 to 10 percent.
- The use of high-strength concrete in bridge decks will result in less live-load deflection.

Flexural Strength and Ductility

- The use of high-strength concrete in bridge decks did not affect flexural strengths of the shorter span girders. At the maximum span lengths for each girder concrete strength, the high-strength concrete in the decks had a slight effect in increasing the flexural strength and ductility of the section.
- A minimum specified deck concrete strength of 41 MPa (6,000 psi) should be used for span lengths in excess of 24.4 m (80 ft) when girder concrete compressive strength exceeds 41 MPa (6,000 psi). Until further analyses can be performed, the specified deck concrete strength should be at least 60 percent of the specified girder concrete strength at 28 days when the specified girder concrete strength exceeds 41 MPa (6,000 psi).
- The applicability of the AASHTO specifications for flexural strength design with high-strength concrete needs to be evaluated.

Prestress Losses and Long-Term Deflections

- The use of high-strength concrete in the decks did not affect the magnitude of the prestress losses or long-term deflections.
- Prestress losses in high-strength concrete girders will generally be less than the losses in lower strength concrete girders.
- The current AASHTO procedure for calculation of prestress losses needs to be modified to account for the properties of high-strength concrete.
- The use of high-strength concrete in girders in place of lower strength concrete will result in less initial camber and similar long-term deflections for the same span lengths.
- Deflection requirements may limit the span lengths for which high-strength concrete girders with high-strength concrete decks can be used.

RECOMMENDATIONS

The Federal Highway Administration should continue to pursue the use of high-performance concrete in bridge decks. The impact of the increased initial costs is likely to be small compared to the long-term benefits. In addition to specifying durability requirements for the deck concrete, a minimum compressive strength of 41 MPa (6,000 psi) should be specified when the girder concrete compressive strength at 28 days is specified to be in excess of 41 MPa (6,000 psi) and span length exceeds 24.4 m (80 ft).

The industry should continue to pursue the usage of concrete with compressive strengths up to 69 MPa (10,000 psi) for prestressed concrete girders. The present research has not identified any limitations that would prevent existing design procedures from being utilized for concrete compressive strengths up to 69 MPa (10,000 psi). Special attention should be given to the deflections of long-span girders.

Additional work should be undertaken to evaluate the applicability of current design procedures for bridges constructed with high-performance concrete. This is particularly important for the longer span lengths where the amount of prestressing will be large and the girders will be spaced close together so that the effective width of the top flange is limited. A rationale should be developed that addresses the effects of the difference in compressive strength between the deck and girder concretes. Additional work is needed to address long-term deflections of long-span girders.

In a previous report, it was concluded that the application of high-strength concrete in bridge girders is limited by the amount of prestressing force that can be applied to the cross section.[6] A reduction in the assumed prestress losses will allow a higher force to be used in design for the same amount of prestressing steel. There is, however, a lack of data about the creep and shrinkage of high-strength concrete as used in prestressed girders. As part of the ongoing showcase projects, FHWA should encourage the monitoring of prestress losses and measurement of creep and shrinkage properties of the concretes.

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