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Applications of High Strength Concrete for Highway Bridges

Executive Summary

U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
Turner-Fairbank Highway Research Center
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FOREWORD

This report, "Applications of High Strength Concrete for Highway Bridges," presents the results of research conducted by the Concrete Technology Corporation for the Federal Highway Administration (FHWA), Office of Engineering and Highway Operations Research and Development under Contract Number DOT-FH-11-9510.

This research was conducted to evaluate the potential structural and economic benefits of using high strength (10,000 psi compressive strength) concrete for highway bridge construction. The technology for making higher strength concretes has improved dramatically over the past 10 years. High strength concrete has been used for high-rise building construction in several United States cities and for bridge girders in the State of Washington for many years, and more recently in the State of Texas. The results of this study indicate that high strength concrete is a viable construction material.

Sufficient copies of the report are being distributed by FHWA Memorandum to provide one copy for each FHWA regional and division office, and five copies for each State highway agency. Additional copies of the report may be obtained from the National Technical Information Service (NTIS), U.S. Department of Commerce, 5285 Port Royal Road, Springfield, Virginia 22161, at a small charge.

Richard E. Hay

Richard E. Hay, Director
Office of Engineering and Highway
Operations Research and Development

NOTICE

It should be noted that the results of this study have served as the basis for initiation of several State highway agency-funded research studies dealing with high strength concrete. The results of these subsequent studies will provide additional information for structural designers.

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16. Abstract

This document presents the results of a study and experimental work to determine the potential benefits of using high strength (10,000 psi compressive strength) concrete for highway bridge construction. Current knowledge and applications of high strength concrete are reviewed and related to use with the AASHTO bridge specifications. Conceptual designs are developed that show the potential of increased span lengths and/or wider girder spacings for commonly used prestressed concrete girder shapes. The designs show that cross sectional areas of compression members (such as struts) can be reduced and thinner sections can be used for segmental bridges, arch bridges, and piers. The capacity of thin concrete plates in buckling is studied. A computer program is presented that is capable of developing comprehensive interaction diagrams to determine whether local thin plate buckling or overall stability is the controlling factor for square hollow piers made with thin plates of high strength concrete. Piers 15-feet (4.6m) long, 5-feet (1.5m) square with 1-1/2-inch (40 mm) thick walls were load tested under varying eccentricities to confirm the interaction diagrams. Code proposals and concepts for more rapidly adapting the use of high strength concrete to highway bridges are suggested.

NOTE: The full final report for this study (FHWA/RD-87/080) is not being published, but it is available through the National Technical Information Service. A limited number of "hot copies" have been made of the full report and they will be available from the Structures Division while the supply lasts.

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METRIC (SI*) CONVERSION FACTORS

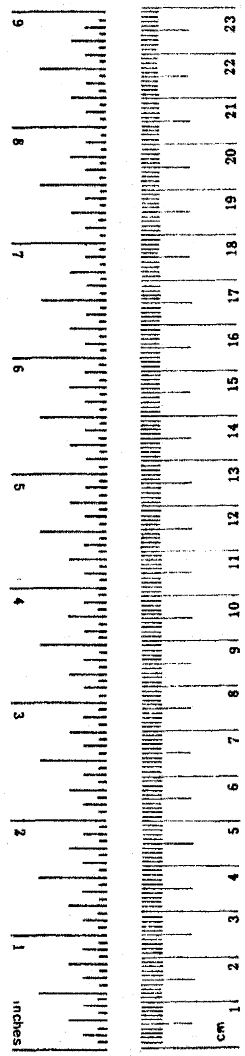
APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
ac	acres	0.395	hectares	ha
	MASS (weight)			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
	VOLUME			
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

TEMPERATURE (exact)

°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
	AREA			
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectores (10 000 m^2)	2.53	acres	ac
	MASS (weight)			
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T
	VOLUME			
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3

TEMPERATURE (exact)

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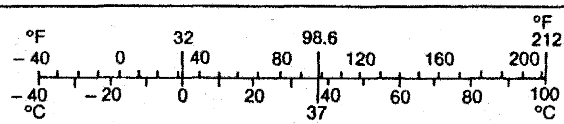

These factors conform to the requirement of FHWA Order 5190.1A.

* SI is the symbol for the International System of Measurements

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EXECUTIVE SUMMARY

INTRODUCTION

The U. S. Department of Transportation authorized Concrete Technology Corporation to identify, through analytical development and laboratory investigation, applications of high strength concrete for highway bridges. The applications were to optimize the usage of 10,000 psi (70 MPa) concrete for advancement in the development of bridge members.

Over the years, the practical design compressive strength of portland cement concrete has steadily increased from 3 ksi (21 MPa) to 6 ksi (42 MPa). Now it is feasible to produce and design concrete with compressive strengths of 10 ksi (70 MPa) and higher. Concrete of this strength requires careful selection of high quality mixing materials and more rigid quality control. High strength concrete has been used successfully in the columns of a 76-story office building and in a 461 ft (140 m) liquid petroleum gas storage vessel.

High strength concrete has been made and used by prestressed concrete producers for years. However, the producers use the high *early* strength of the concrete so that the formwork may be removed, the concrete members prestressed, and the forms reused every day without taking advantage of the eventual 8 to 10 ksi (56 to 70 MPa) compressive strength in the structural design.

OBJECTIVE

The objective of the project was to demonstrate the advantages of high strength concrete in highway bridges by identifying analytically those members or systems most likely to benefit. Certain analytical predictions would be confirmed by test and the results interpreted in the form of design recommendations and proposed code changes. The investigation was limited to plant precast members since those products showed the most potential for effective utilization of high strength concrete. Practical techniques for manufacturing and constructing with high strength concrete were to be included in the report with the goal of illustrating the advantages in a way which would lead to immediate use for practical application.

MIXING AND PLACING HIGH STRENGTH CONCRETE

Guidelines for mixing and placing high strength concrete are based on the following:[1-3]

Constituents: Cement, aggregates, and admixtures that will produce concrete of the required strength and insure consistency of properties over a period of time must be selected. The maximum size of coarse aggregate must be small, usually not greater than 0.5 in (13 mm). Superplasticizers may be used.

Water-cement ratio: The water-cement ratio must be low, usually between 0.30 and 0.35, by weight.

Cement content: The cement content must be high, usually between 800 and 950 pcy (475 and 565 kg/m^3).

Consolidation: Good consolidation in sturdy forms is necessary to produce adequately dense concrete.

Quality control: A well-managed quality control program, including planning and cooperation of all parties involved, is essential to success.

PHYSICAL PROPERTIES OF HIGH STRENGTH CONCRETE

The properties of high strength concrete, similar to the properties of normal strength concrete, can be predicted by anyone familiar with concrete engineering. Ratios of tensile strength to square root compressive strength for high strength concrete are similar to those obtained for normal strength concrete.[4] Stress-strain relationships for high strength concrete are shown in Figure 1.[4-6] The initial slope of the curves becomes steeper with increasing compressive strength (modulus of elasticity increases), and the strain at maximum stress tends to increase slightly. Also, the slope of the descending portion of the curves is steeper for the high strength concretes than for normal strength concretes, so the high strength concrete in effect is more brittle.

Data indicate that modulus of elasticity for 10 ksi (70 MPa) concrete is represented better by the formula $E_c = 26w^{1.5}\sqrt{f'_c}$ $(0.0337w^{1.5}\sqrt{f'_c})$[4,6,7] than by the formula $E_c = 33w^{1.5}\sqrt{f'_c}$ $(0.0428w^{1.5}\sqrt{f'_c})$ presently suggested by the American Association of State

Highway and Transportation Officials (AASHTO) code and the American Concrete Institute (ACI) code and used for normal strength concrete.(8,9) High strength concretes creep less than normal strength concretes when loaded to a given percentage of compressive strength.(4) Shrinkage of high strength concrete is about the same as that of normal strength concrete.(4,7)

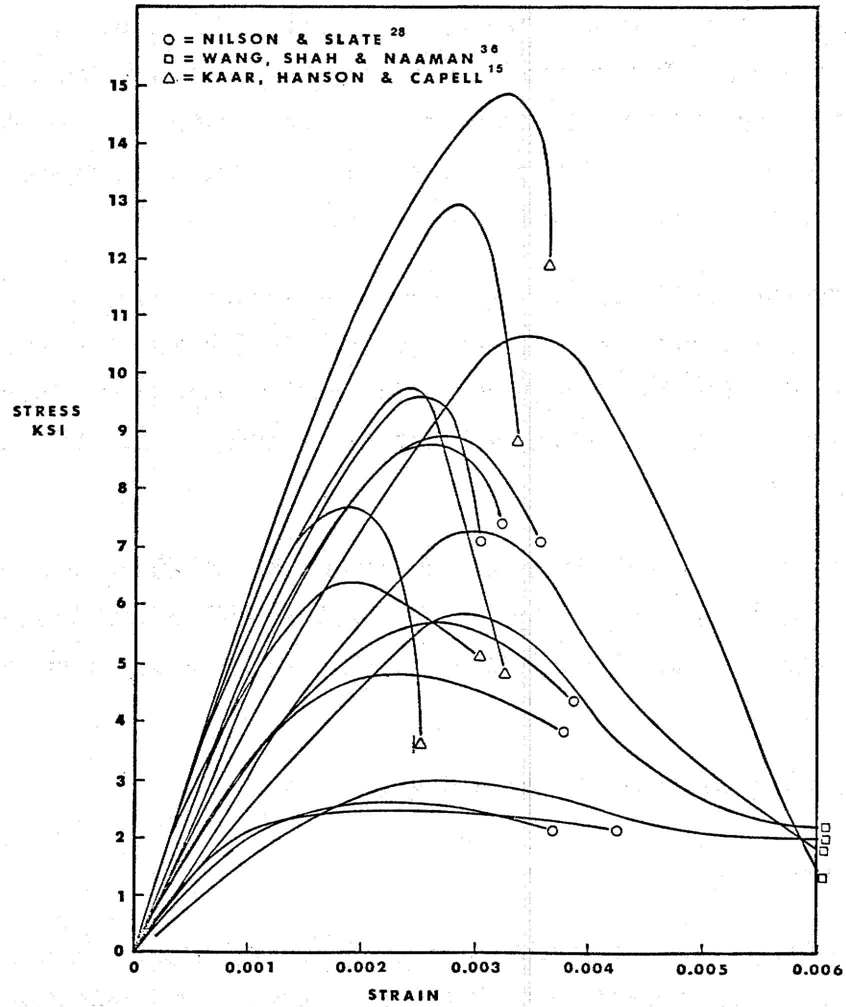


Figure 1: Typical stress-strain curves.

ADVANTAGES AND DISADVANTAGES OF HIGH STRENGTH CONCRETE

One advantage of high strength concrete is its greater compressive strength, which can be evaluated in relation to unit cost, unit weight, and unit volume. High strength concrete, with its greater compressive strength per unit cost, is the least expensive means of carrying compressive force. In addition, its greater compressive strength per unit weight and unit volume allows lighter, more slender bridge members.

Other advantages of high strength concrete include increased modulus of elasticity and increased tensile strength. Increased stiffness is advantageous when deflections or stability govern the bridge design, and increased tensile strength is advantageous in service load design in prestressed concrete.

A disadvantage of high strength concrete is that the mix has much less water than normal strength concrete. This results in mixes that have reduced workability and handling time, making them more difficult to place and properly compact. In addition, high quality aggregates in necessary sizes and cement that will consistently produce concrete of the required strength may not be available in some localities.

Structural design considerations may preclude effective use of the increased concrete strength. Cross section dimensions often are governed by factors other than stress, such as minimum cover, so that the full strength capability of the concrete is not used. Further, the total prestress force that can be generated may not be sufficient to take advantage of the high strength concrete.

Other disadvantages of high strength concrete are its additional cost and the additional expenses of increased quality control. Finally, AASHTO specifications[8] tend to discourage the use of high strength concrete because the specifications are based on the properties of normal strength concrete.

APPLICATIONS OF HIGH STRENGTH CONCRETE

Solid Section Girders

The effect of using high strength concrete in four different solid section girders was investigated. Cross section dimensions of the four girders — the bulb tee, the Washington State Department of Transportation (WSDOT) Standard, the AASHTO-Prestressed Concrete Institute (PCI) Standard, and the Colorado Standard — are shown in Figure 2. Two methods of attaching the decks were considered: integrally cast and cast-in-place. Decks were cast-in-place on the completed girder without shoring, so that the entire dead load of both girder and deck was carried by the girder section.

To determine span capabilities of the girders, AASHTO HS20-44 loading was used, with a lateral distribution factor S/5.5. Allowable stresses conformed to the AASHTO code, with allowable tension in the precompressed tension zone equal to $3\sqrt{f'_c}$ $(0.249\sqrt{f'_c})$ and allowable compression equal to $0.4\ f'_c$.

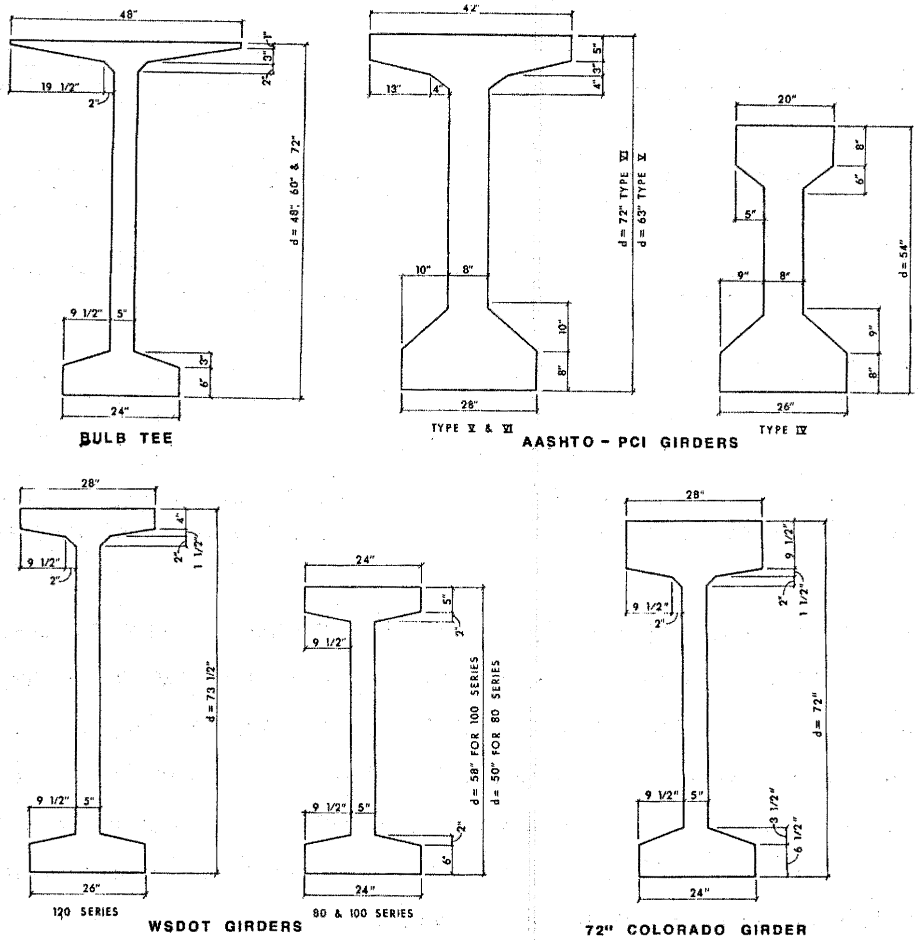


Figure 2: Solid section girders — cross sections investigated.

The effectiveness of high strength concrete in increasing the span capabilities of a given cross section is shown in Figure 3. Here, for a beam spacing of 8 ft (2.4 m), the span capabilities of all the basic 72 in (1,830 mm) deep integral deck sections are shown for the different concrete strengths. The maximum available prestress force sets span length capabilities at the same value for all concrete strengths for the AASHTO-PCI, and Colorado sections. However, for the Bulb Tee and WSDOT sections, the benefits of using high strength concrete are apparent.

The same information for the basic 72 in (1,830 mm) deep sections with cast-in-place decks is shown in Figure 4. The benefits of high strength concrete in increasing span capabilities are easily identified, since the available prestress factor does not control.

A comparison of Figures 3 and 4 shows the advantage of precasting the deck as an integral part of the beam section before prestressing. In addition to providing a high strength concrete in the deck, the prestress force is extended into the compression flange resulting in greater span capabilities.

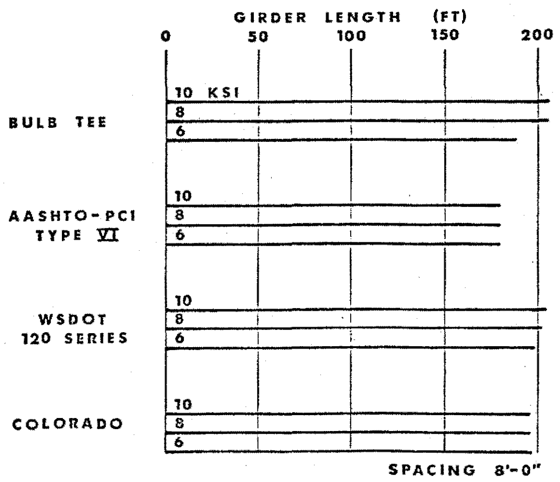


Figure 3: Span capabilities - basic 72-in sections with integral decks.

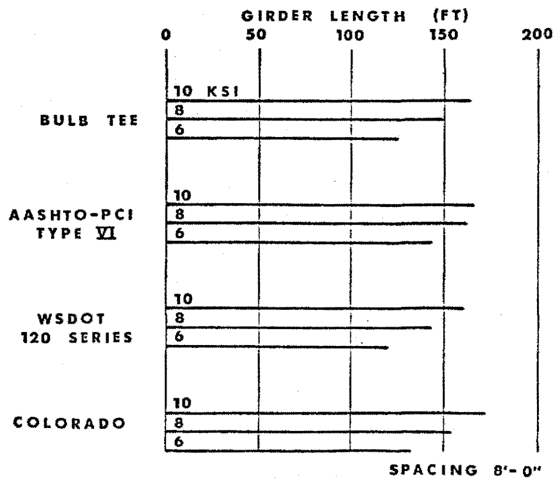


Figure 4: Span capabilities - basic 72-in sections with cast-in-place decks.

The potential of using shallower members with increase in concrete strength is shown in Figures 5 and 6. For the integral deck, the potential of a reduction in depth from 72 in (1,830 mm) to 48 in (1,220 mm) is available for girder spacings up to 6 ft (1.8 m). The influence of maximum available prestress force limits the potential reduction for wider girder spacings. For the cast-in-place deck, the potential of reducing depth of section from 72 in (1,830 mm) to 48 in (1,220 mm) with increase in concrete strength from 6 ksi to 10 ksi (42 MPa to 70 MPa) is available for all girder spacings.

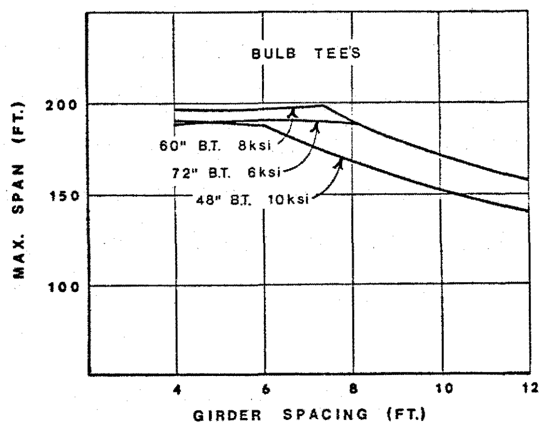


Figure 5: Depth variations - bulb tees with integral deck.

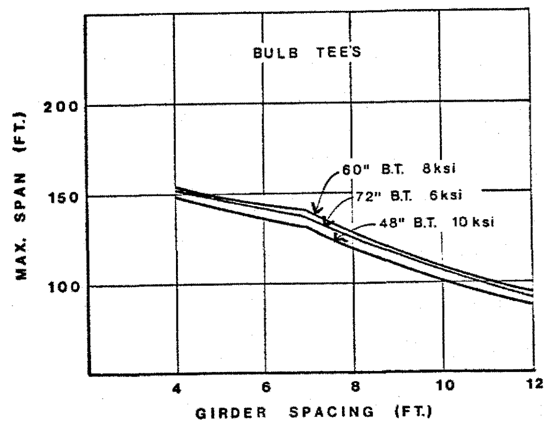


Figure 6: Depth variations - bulb tees with cast-in-place deck.

Making maximum use of the greater load carrying capabilities of high strength concrete girders requires different designs for bridges. Two designs of a 150-ft (46 m), simple span bridge are shown in Figure 7. On the left side, nine 6 ksi (42 MPa) concrete girders were used. On the right side, four 10 ksi (70 MPa) concrete girders were used. The advantages of the high strength concrete are evident: only four girders are needed for the high strength concrete design, while nine girders are needed for the normal strength concrete design. In spite of the thicker cast-in-place deck needed for the greater transverse span between the four girders, the overall dead load is reduced, and therefore total prestressing requirements are reduced.

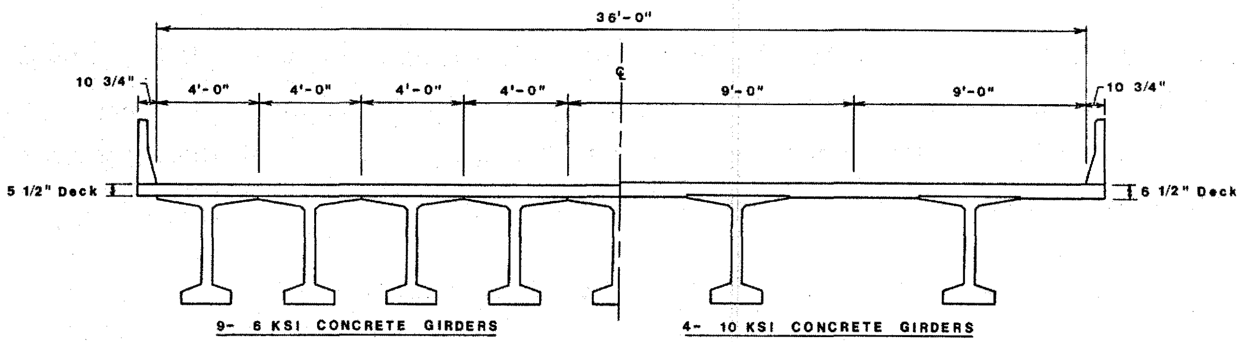


Figure 7: 150-ft,simple span bridge - alternate concrete strengths.

Post-tensioned Box Girders

Multiple span, cast-in-place, continuous post-tensioned box girder bridges of constant depth were represented by the two-span continuous structure shown in Figure 8. Concrete strengths were 6, 8, and 10 ksi (42, 56, and 70 MPa). Overall beam depths were 4.5, 5.5, and 6.5 ft (1.4, 1.7, and 2 m). Allowable stresses were the same as those used in the solid section girder analysis, except that allowable tension in the precompressed tension zone was assumed to be $6\sqrt{f'_c}$ ($0.498\sqrt{f'_c}$). Loading was three lanes of AASHTO HS20-44, without lane reduction.

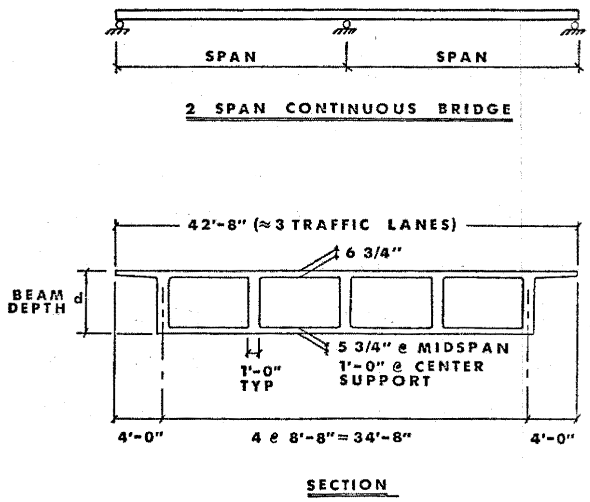


Figure 8: Two-span continuous post-tensioned box girder bridge.

Span capabilities for the different girder depths are shown in Figure 9. High strength concrete for continuous box girders of 150-to 250-ft (46 to 76 m) spans increased span capabilities. As with the integral deck solid section girders, the maximum available prestress force limited capabilities of the high strength concrete.

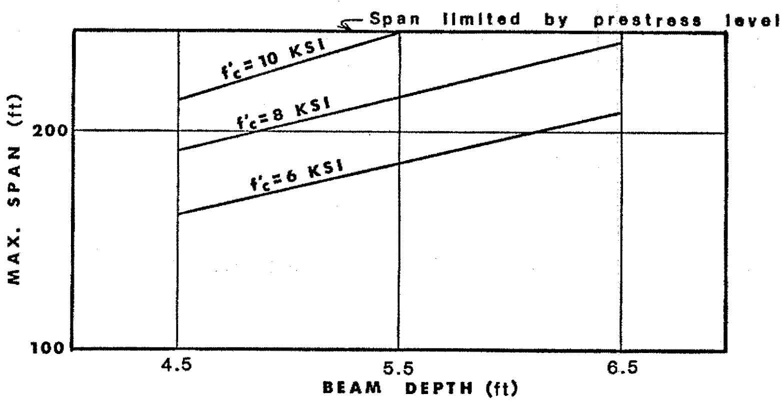


Figure 9: Span capabilities - two-span continuous box girder bridge.

Segmentally Post-tensioned Box Girders

Segmentally post-tensioned box girder bridges of medium to long span were represented by the free cantilever Shubenacadie Bridge (South Mainland, Nova Scotia) with a 700-ft (213 m) main span and 372-ft (113 m) side spans.[10] Overall dimensions of the bridge are shown in Figure 10. The bridge was constructed with 5 ksi (35 MPa) concrete and used 1.25 in. (31.75 mm) diameter thread bars for post-tensioning. No tension was allowed in the precompressed tension zone. AASHTO HS20-44 loading was used.

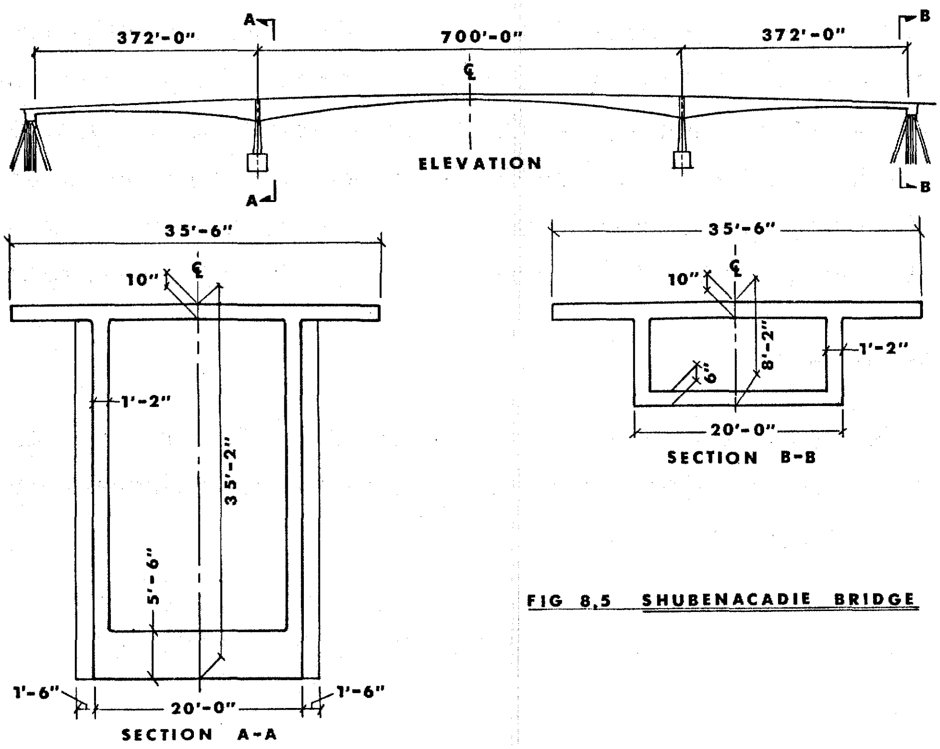


Figure 10: Shubenacadie free cantilever bridge.

The bridge was reanalyzed using 10,000 psi (70 MPa) concrete to determine how much the thickness of the lower flange could be reduced and what effect this reduction would have on the overall moments. As shown in Figure 11, using high strength concrete reduced the total flexural prestress force by more than 10% as a result of the reduced dead load. The optimum lower flange thickness is 1.6 ft (0.5 m), obtained at about 8 ksi (56 MPa) strength.

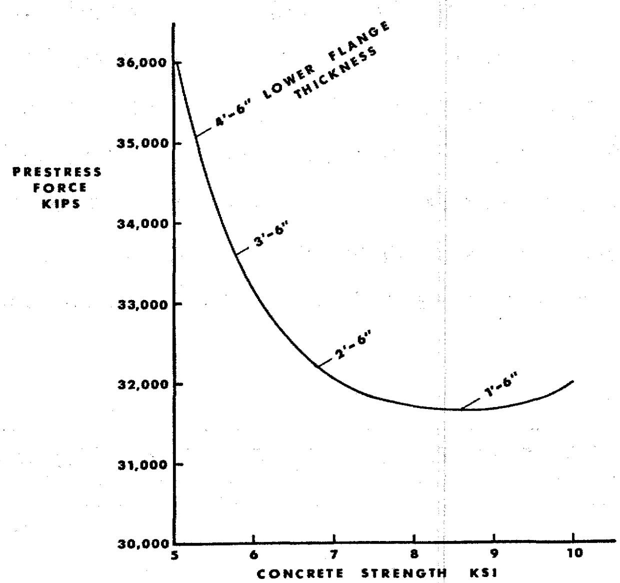


Figure 11: Variation of prestress force and flange thickness with concrete strength - Shubenacadie Bridge.

For segmentally post-tensioned box girder bridges, high strength concrete is feasible in regions, such as the lower flange, where the design is controlled by stress. In regions, such as in the deck, where the design is controlled by other factors, normal strength concrete can be used. The webs may be constructed of either high strength concrete or normal strength concrete, depending on minimum thickness requirements, minimum shear reinforcement requirements, and the contribution of concrete in the webs to the shear carrying capacity.

Compression Members

Solid pier shafts and elements of Y-piers (Figure 12) are examples of compression members in bridges. Compression members were expected to benefit greatly from applications of high strength concrete.

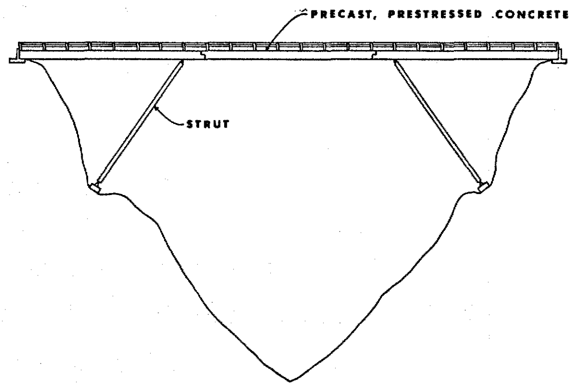


Figure 12: Application of compression strut.

To study the effect of increasing the compressive strength of concrete from 6 ksi (42 MPa) to 10 ksi (70 MPa), interaction diagrams were developed for the compression strut shown in Figure 13. The diagrams were developed for a pin end condition, assuming that the cross section possessed the bilinear moment-curvature relationship defined by strain compatibility and the elastic properties of the concrete and prestressing strand. The deflected shapes were determined by integrating the curvatures along the length of the member. The ultimate strength design method was used with factored loads.

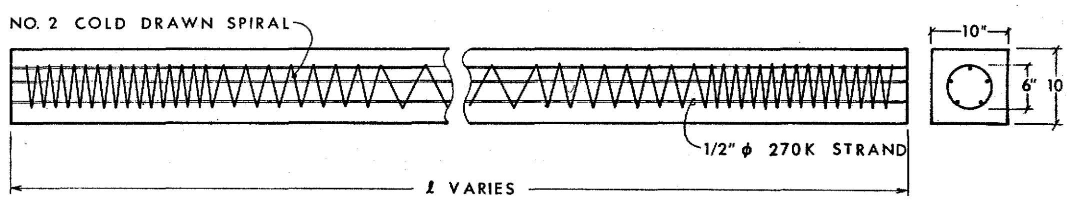


Figure 13: Strut details.

Figure 14 shows that capacities of the compression strut increase with the increase in concrete strength.

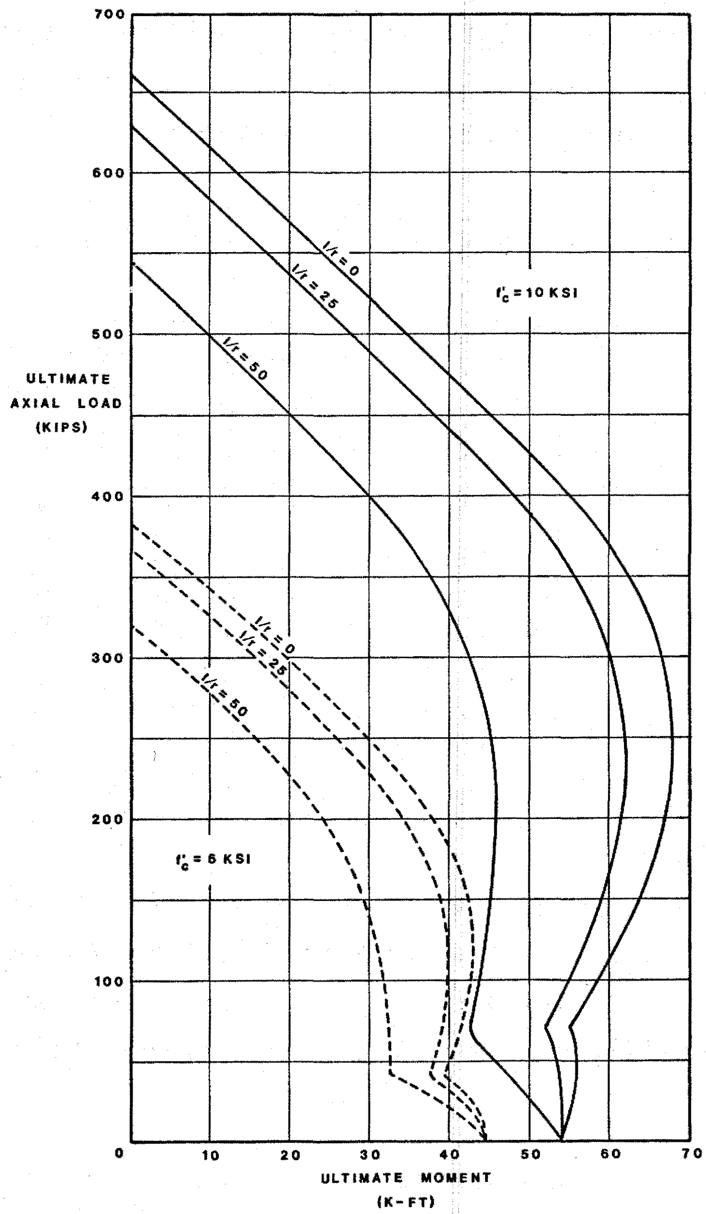


Figure 14: Interaction diagram - 10 in square prestressed strut.

--- f'_c = 6 ksi
—— f'_c = 10 ksi

Results of the compression member study are shown in Table 1. Three slenderness ratios, three concrete strengths, and three eccentricities are shown. For short, concentrically loaded struts, capacity is determined by multiplying the cylinder strength by the cross sectional area and then subtracting the prestress force. Because the prestress force is constant, the benefits of increasing concrete strength are more than the strength increase itself. This is illustrated by the strength ratios in the table that exceed 10/6 = 1.67 for those struts.

For combined axial load and bending, using high strength concrete, even for relatively slender columns, is beneficial. It can be concluded that compression members are an excellent application for high strength concrete. Smaller sections can be used for a given number of members, or fewer members can be used in a given location. In either case, weight as well as material and construction cost are reduced.

Table 1.
Axial load capacities for 10-in square strut.

ℓ/r	f'_c (ksi)	ECCENTRICITY					
		e = 0		e = 0.1D		e = 0.2D	
		P (kips)	P/P_6	P (kips)	P/P_6	P (kips)	P/P_6
0	6	375	1.00	284	1.00	217	1.00
	8	518	1.38	389	1.37	294	1.35
	10	662	1.77	478	1.69	362	1.67
50	6	320	1.00	233	1.00	168	1.00
	8	437	1.37	317	1.36	220	1.31
	10	545	1.70	390	1.67	267	1.59
100	6	205	1.00	133	1.00	80	1.00
	8	270	1.32	165	1.24	90	1.13
	10	325	1.59	190	1.43	100	1.25

Note: ϕ factors P in accordance with ACI 318-77 Section 9.3.2c and AASHTO code Section 1.5.33.4[38] are included in this table.

Thin Walled Sections

A circular and a square pier with 6-in (152 mm) wall thicknesses were chosen for investigation. A 15-ft (4.6 m) outside diameter hollow circular pier had a prestress steel area of 20.2 sq in (130 sq cm). A 10-ft (3 m) sq hollow pier had a prestress steel area of 18.4 sq in (118 sq cm) concentrated in the corners. Concrete strengths studied were 6 ksi (42 MPa) and 10 ksi (70 MPa).

The piers were considered to have pinned end connections which were free to rotate but not free to translate. AASHTO 1977 code section 1.5-31[8] was used to design the piers. Interaction diagrams were constructed using the assumption that the concrete stress equal to 0.85 f'_c was distributed as a rectangular stress block. A bilinear moment-curvature relationship was used to approximate the actual parabolic moment-curvature of the section. The final deflected shape of the column from which the information for the interaction diagram was obtained was constructed using the same principles as outlined for compression struts. Concrete stresses were obtained from the parabolic stress-strain curve for the concrete, assuming a maximum strain of 0.003. From the column-deflection curves, a combination of eccentricity of axial load and height of column is found for a particular axial load. A range of axial loads were considered from zero to the case of pure compression.

The interaction diagrams in Figure 15 and in Figure 16 show the increased capacity of the piers when concrete strength is increased from 6 ksi (42 MPa) to 10 ksi (70 MPa). The benefit of increasing concrete strength is more than directly proportional to the strength increase.

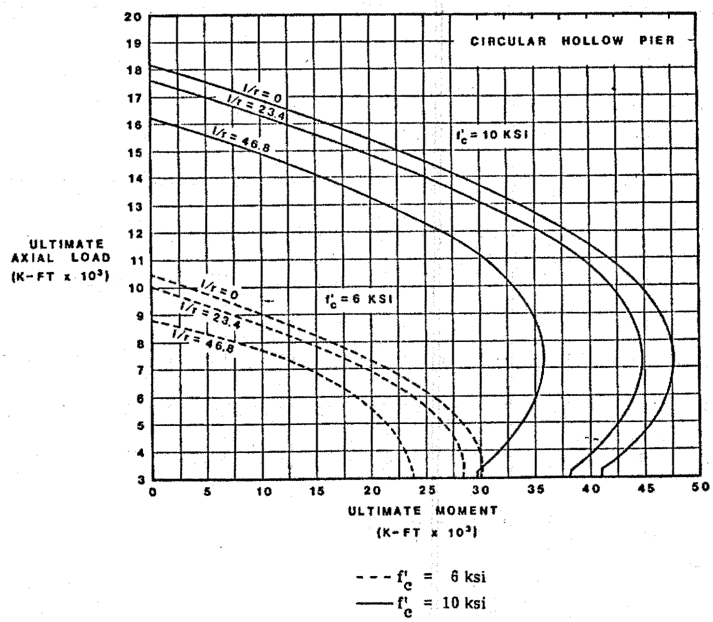


Figure 15: Interaction diagram - 15 ft diameter circular hollow pier.

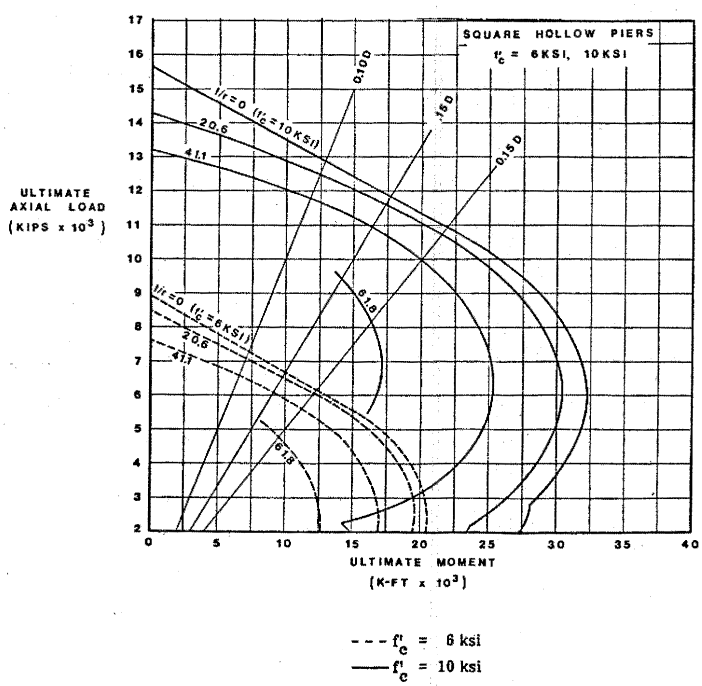


Figure 16: Interaction diagrams - 10 ft square hollow pier.

High strength concrete permits the adaptation of thin wall members to pier construction for major bridge spans. The strength of these sections permits their use when tall piers are required. The increased load carrying capability permits longer spans or fewer piers. The lighter section permits construction procedures with minimal disruption to surrounding terrain.

Two construction procedures are shown in Figures 17 and 18. In Figure 17, thin plates are joined at the corners with cast-in-place concrete. By building the piers out of sections of plates and installing the precast diaphragms shown at 5 to 10 ft (1.5 to 3 m) levels, these plates could be erected with the utilization of a gin pole and minimum winch capacity.

In Figure 18, the square prismatic box sections are match cast, transported to the site, and post-tensioned to form the pier using standard segmental construction methods. The size of the sections could be controlled by site requirements and crane capacities.

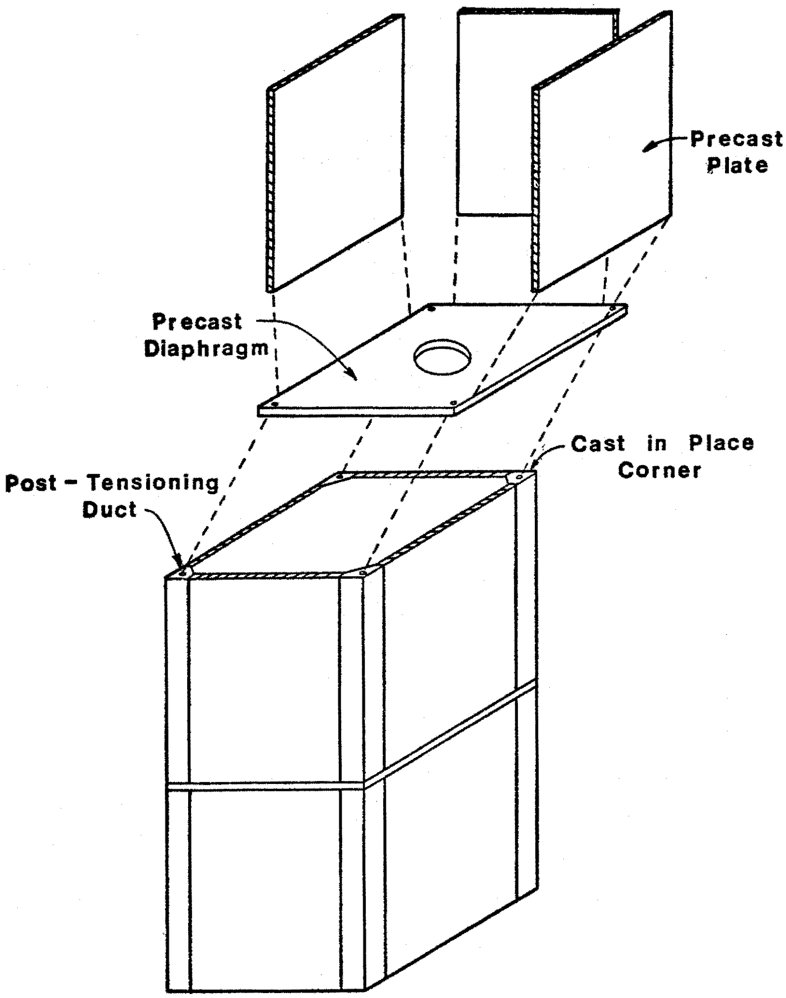


Figure 17: Square hollow pier — thin plate precast segments.

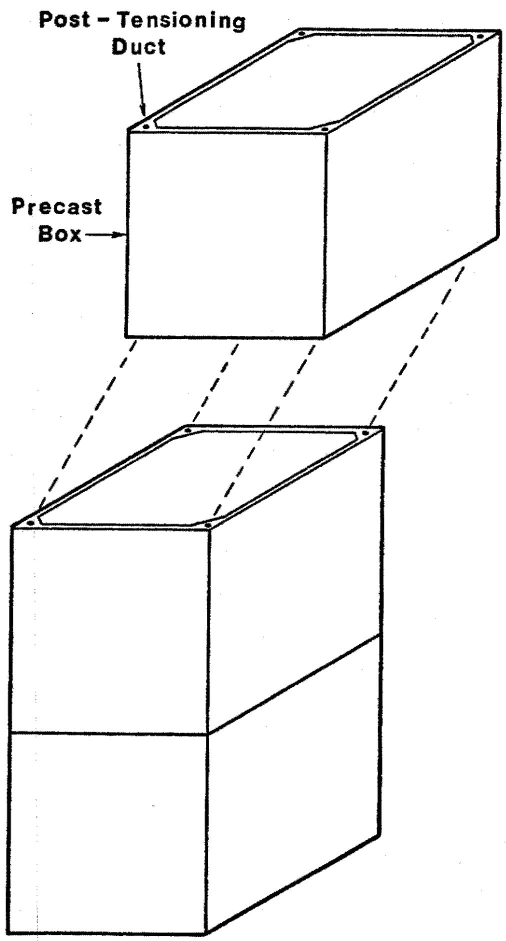


Figure 18: Square hollow pier — match cast precast segments.

Evaluation of Benefits

The results of the study of applications of high strength concrete to bridge members are evaluated qualitatively in terms of a number of criteria listed in Table 2. Struts and hollow piers offer the best possibility for materials savings percentage-wise over the other members since either the sizes of the individual members or the total number of members can be reduced. The potential for reduced shipping weight for hollow pier prismatic members or plates offers a relatively new construction procedure for this bridge member. These precast sections can be adapted to changing requirements by form modification if the change is not major. These factors led to the selection of a test program for square hollow piers.

Table 2.
Evaluation of benefits of use of high strength concrete.

Member or System	Materials Savings	Reduced Shipping Weight	Reduced Structure Depth	Difficulty of Production	Adaptable Standard Section
Solid Section Girder, Integral Deck	M	M	H	M	M
Solid Section Girder, Cast-in-place Deck	M	M	H	M	M
Commonly Used Box Girders	M	—	H	H	L
Segmentally Post-tensioned Box Girders					
Plant Precast	M	H	M	M	—
Cast-in-Place	M	—	M	H	—
Struts, Solid Square	H	H	—	L	H
Piers, Hollow Thin Walled	H	H	—	M	H

Key: H - High
M - Medium
L - Low
— - Does not apply

DESIGN OF TEST SPECIMEN

Objective

A square hollow pier specimen was selected to determine that design could be controlled by overall buckling of the pier rather than by buckling of the thin plate wall of the pier. Fabrication procedures for thin plate segments with cast-in-place corners would be compared with plant fabricated, match-cast prismatic boxes. Post-tensioned construction techniques would be confirmed.

Analytical Development

Although studies of thin plate buckling of steel plates have been made, the application of these studies to concrete thin plates was questionable. Applying formulas developed by Priest[11] for high strength steels and applying these formulas with k factors developed by Bleich[12] for buckling strength of metal structures develops Equation 1

$$\sigma_{cr} = \frac{\pi^2 E_t}{3(1-\mu^2)} \left(\frac{t}{b}\right)^2 \qquad \text{(EQ 1)}$$

in which

σ_{cr} = critical unit compressive stress, psi

E_t = tangent modulus of elasticity, psi

μ = Poisson's ratio

b = width of plate, inches

t = thickness of plate, inches

As can be seen from this equation, the critical stress is directly proportional to the modulus of elasticity and directly proportional to the square of the thickness-to-width ratio of the plate.

In applying this equation to hollow rectangular piers, the following comments can be made:

1. Under concentric axial load where the entire section is subject to uniform compression, all four sides of the cross section are equally susceptible to buckling.

2. When the section is subject to eccentric load, the compression flange will be the most likely to buckle first.

3. Considering that only eccentric loads are encountered in practical applications, local buckling of the thin compression flange should be considered in the capacity investigation of the section.

4. An interaction diagram that will take compression flange stability as well as overall stability into consideration in addition to material failure seems to represent the only logical solution to the problem.

Figure 19 shows a schematic of such an interaction diagram. Note that the ACI code[9] concrete stress block cannot be used to develop this diagram. Instead, representative stress-strain diagrams, as shown by Figure 20, must be used in the computation. In determining critical stress, average rather than maximum stress in the compression flange should be used.

The comprehensive interaction diagram shown in Figure 19 will handle any buckling situation for square or rectangular piers. A computer program written in Fortran IV language was prepared for this project enabling the analysis of hollow rectangular concrete sections containing prestressed and nonprestressed reinforcement.

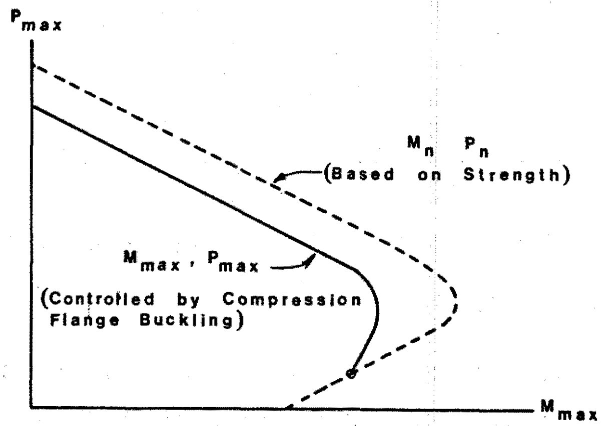


Figure 19: Interaction diagram - thin plate buckling.

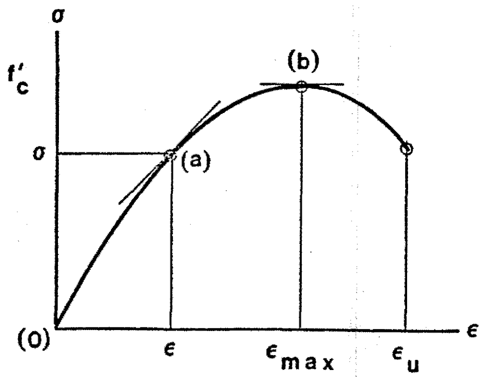


Figure 20: Typical concrete stress-strain curve.

Interaction diagrams developed from this program for 10 ft (3 m) square hollow piers with 3, 4, 5, and 6 in. (76, 102, 127, and 152 mm) wall thicknesses are shown in Figure 21. In order to show the effect of local buckling of the compression flange on the load resistance of the cross section, curves representing load capacities based on both material failure and local buckling are portrayed. An examination of the figure indicates that the gap between capacity based on material failure and that controlled by local buckling increased with the decrease of the thickness to the width ratio of the compression flange.

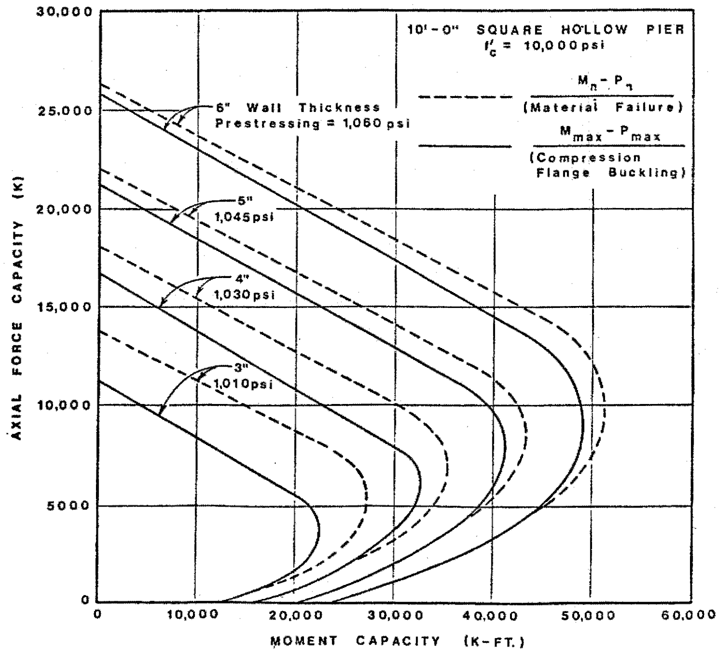


Figure 21: Interaction diagram -- 10 ft square hollow pier.

SQUARE HOLLOW PIER TEST SPECIMEN

A 5 ft (1.5 m) square cross section member with a 1-1/2-in (40 mm) thick wall was selected as representative of a half-scale model of a 10 ft (3 m) square pier with 3-in (76 mm) wall thicknesses shown in Figure 22. A 15-ft (4 m) length was selected to provide short column action removing the parameter of overall pier stability.

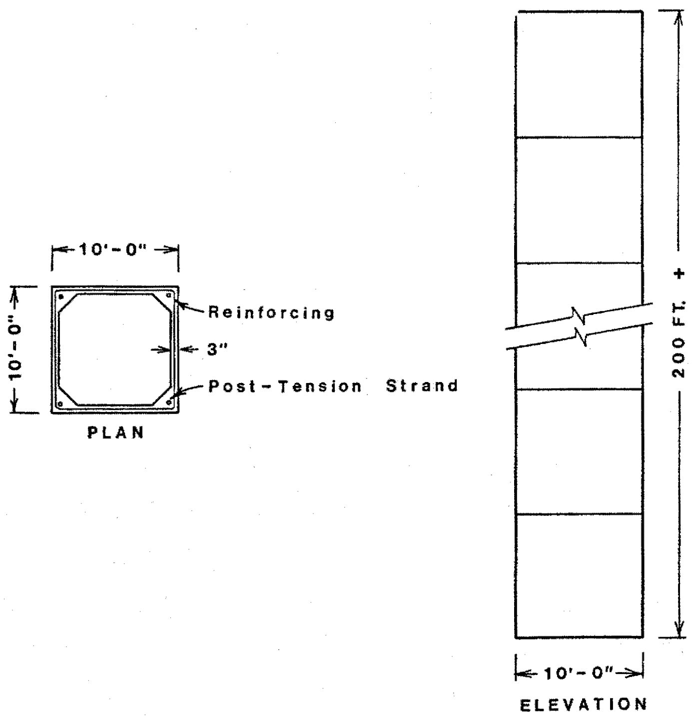


Figure 22: Square hollow pier.

The interaction diagram for the specimen is shown in Figure 23. The predicted test results based on the specimen performing in accordance with the theory developed are tabulated in Table 3.

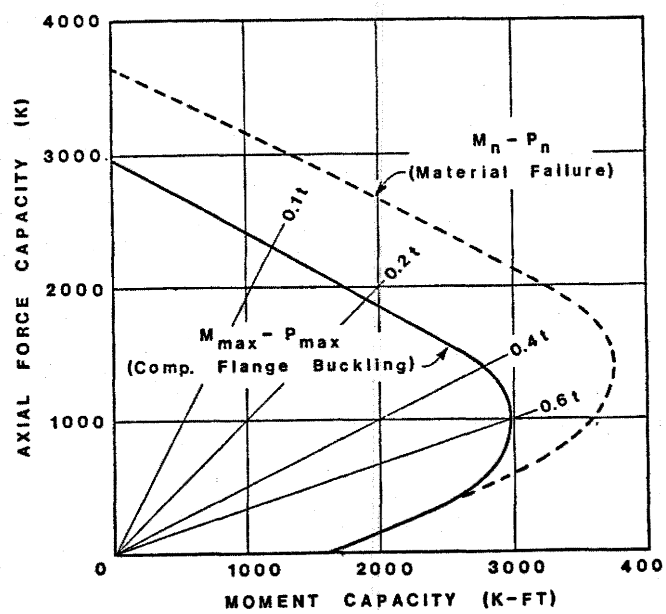


Figure 23: Interaction diagram - 5 ft square hollow pier specimen.

f'_c = 10 ksi

Table 3.
Predicted test results.

e (in.)	P (kips)	M (kip•ft)	Curvature (Rad/in.)	Deflection (in.)	End Rotation (degrees)
6	2300	1150	1.041×10^{-5}	0.04	0° 3' 14"
12	1890	1890	1.46×10^{-5}	0.06	0° 4' 32"
24	1390	2780	2.01×10^{-5}	0.08	0° 6' 14"
36	990	2970	3.43×10^{-5}	0.14	0° 10' 36"

Fabrication of Test Specimens

Match cast precast segments 5 ft (1.5 m) square with two 5 ft (1.5 m) long 1-1/2 in (40 mm) thick walls were selected for testing. As shown in Figure 24, three segments would be match cast and connected together by post-tensioning, making a 15 ft (4.6 m) long pier to be tested for local buckling. The match cast forming is shown in Figure 25. Specimens were cast in the Architectural Plant at Concrete Technology Corporation, epoxied and tensioned together and moved into the laboratory as shown in Figure 26.

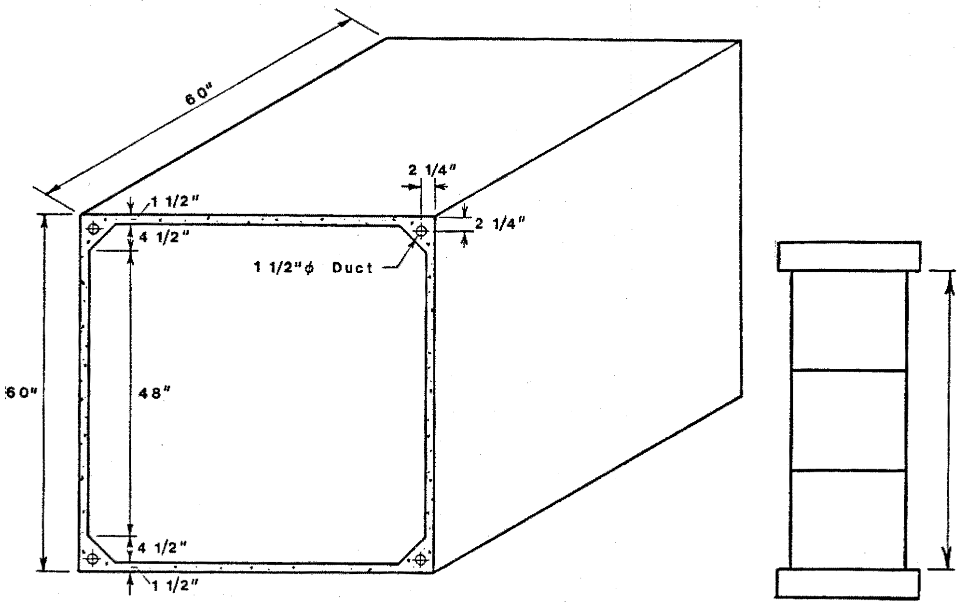


Figure 24: Test segment and specimen.

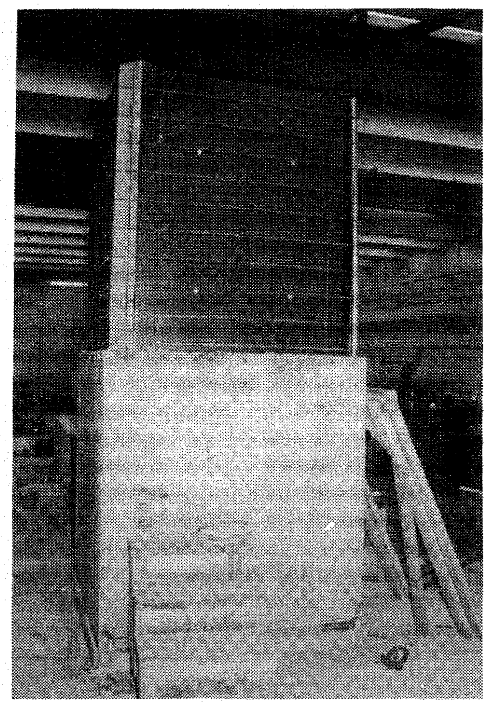


Figure 25: Match cast forming.

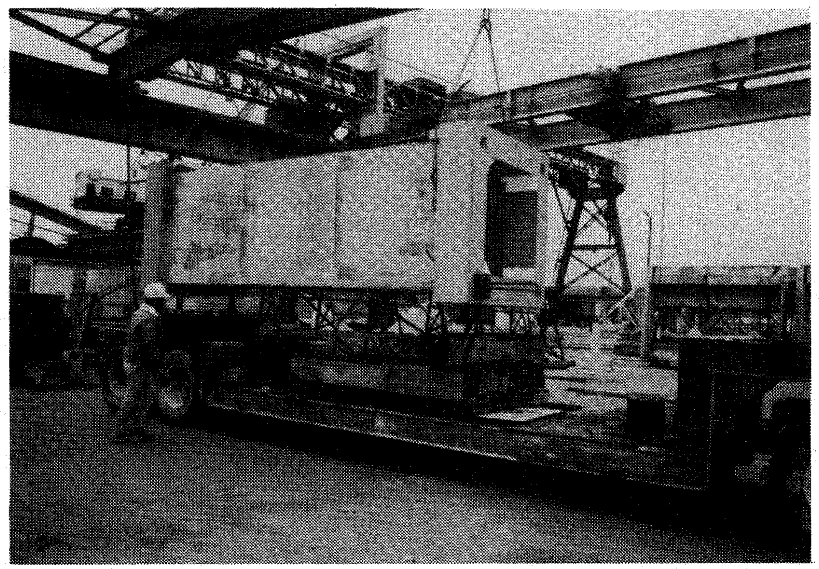


Figure 26: Moving specimen.

The fabrication procedures confirmed the difficulties that could be experienced in match casting square boxes with thin walls. It was found to be difficult to maintain the alignment of the walls. Furthermore, the thin sections created a problem in placing high strength concrete with the necessary slump requirements into the thin section. A preferable construction would be to cast thin plates on a continuous bed as is customary in precasting plants, truck them to the site on flatbed trucks, erect them in position and cast corners in place with longitudinal post-tensioning.

Testing Procedures

The test arrangement is shown in Figure 27. Loading rams were located to provide the necessary eccentricities for the specimens as required in Table 3. Loading was applied in two phases. The first load phase was planned to study the performance of the specimen under service load conditions by cycling the load in small increments from zero to service load level. The second load phase was planned to verify the ultimate load capacity of the specimen as well as the mode of failure. The load was applied in small increments from service load to failure load.

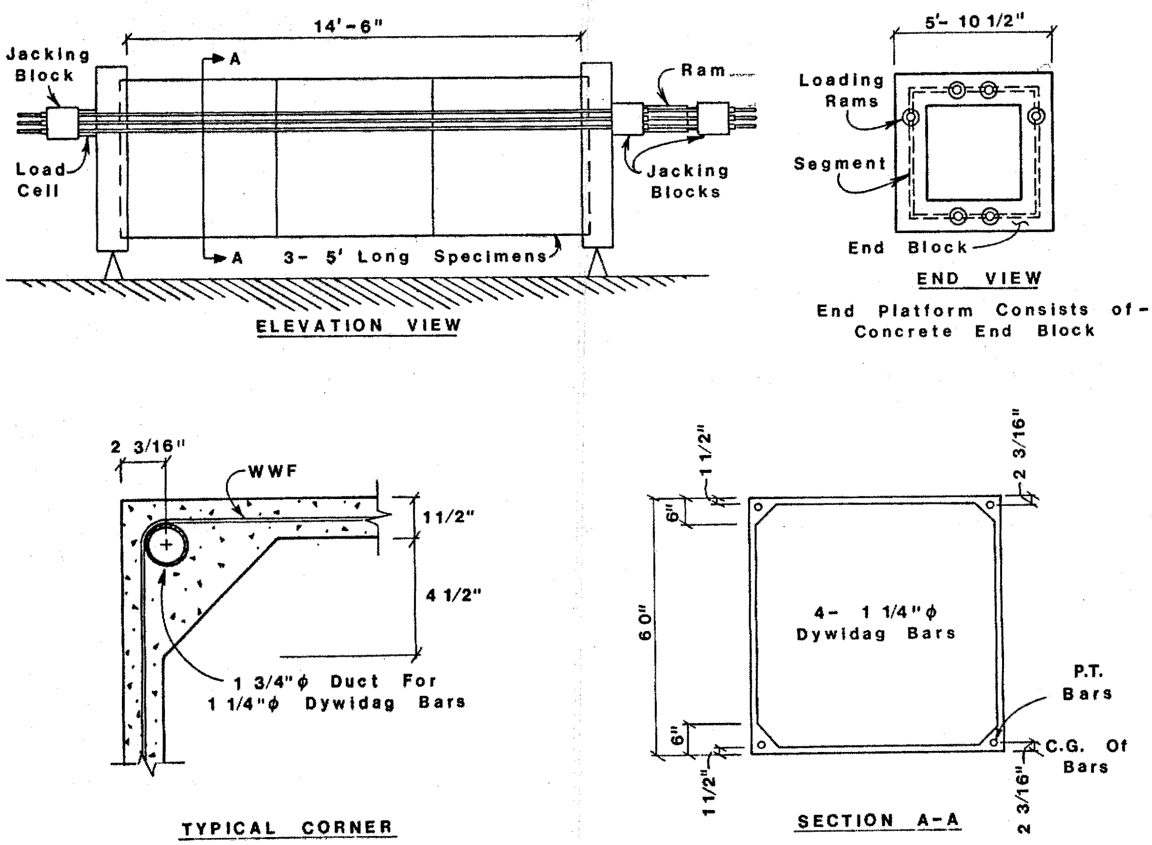


Figure 27: Square hollow pier test arrangement.

Instrumentation included load cells for measuring forces, potentiometers for measuring displacements and curvatures, and strain gages for measuring strain in concrete. Experimental information was recorded using a high speed automatic data acquisition system. Load cell arrangement for specimen 1 is shown in Figure 28.

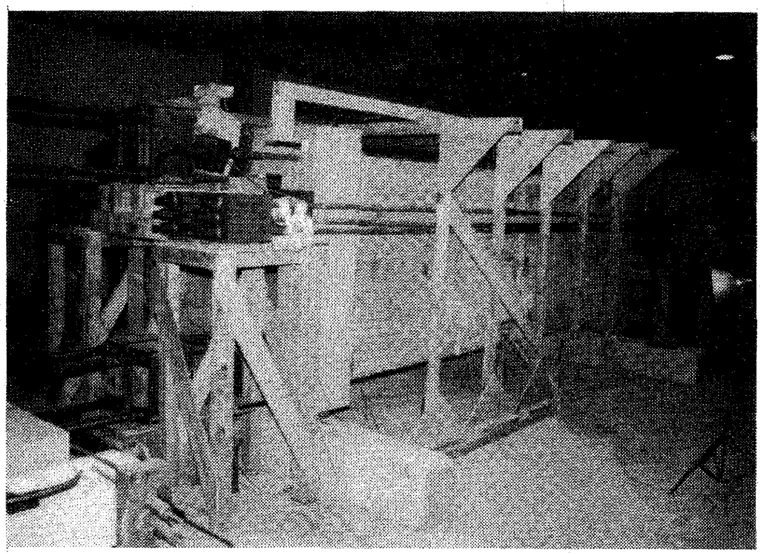


Figure 28: Load cell arrangement for specimen 1.

Analysis of Test Data

Loading was applied to specimen 1 to place the total reaction of all loads at a 6 in. (152 mm) eccentricity. The specimen failed explosively in the center segment at a load of 1,613,500 lbs (7,172 kN) at an eccentricity of 4.9 in. (124 mm). Figure 29 shows the buckling of the top plate. The failure extended totally through the specimen at approximately the same location. The concrete strength in the failed segment at the time of testing was 8,680 psi (60 MPa). An interaction diagram based on this strength is shown in Figure 30. The failure point indicated on the diagram shows that the actual failure load is reasonably close to the predicted envelope.

Specimen 2A was loaded at an eccentricity of 20 in. It failed explosively like specimen 1 at a load of 1,346,000 lbs (5,983 kN) at an eccentricity of 20.2 in. (513 mm). The failed specimen is shown in Figure 31. The compressive strength for segment 6A at time of failure was 9,880 psi (68 MPa). The interaction diagram shown in Figure 32 for 10,000 psi (70 MPa) concrete shows the failure point. Once again, the failure load is very close to the predicted failure envelope. These two specimens confirmed two points on the interaction diagram at widely separated eccentricities. The two points are on the flat portion of the diagram which are points of critical concern in most pier failures. Therefore, this data can be considered adequate as a verification of that portion of the diagram. Testing of additional specimens at eccentricities between the two tested would serve as additional confirmation of the straight portion of the diagram. When eccentricities are great enough to cause tension in the bottom plate, the interaction diagram closes rapidly to the point where overall stability and local plate buckling are approximately equal. Therefore, this condition is not crucial; however, an additional test at the apex of the curve is recommended.

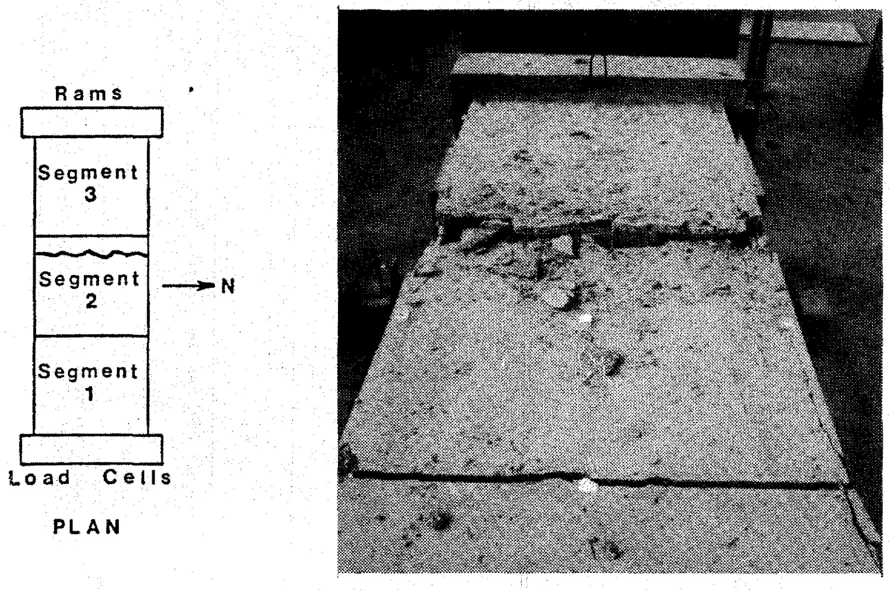


Figure 29: Specimen 1 - Top Plate.
Orientation of specimen and view of top plate looking west showing buckling of west plate over east plate.

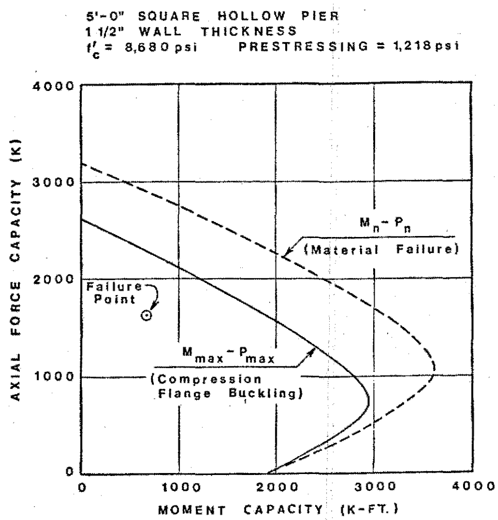
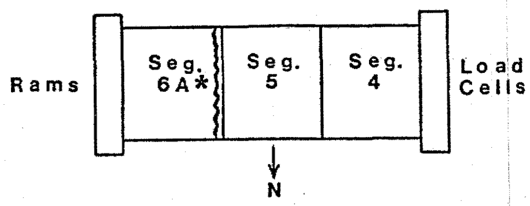


Figure 30: Specimen 1 — Interaction diagram.



* Replaces segment 6; which failed prematurely during test of specimen 2

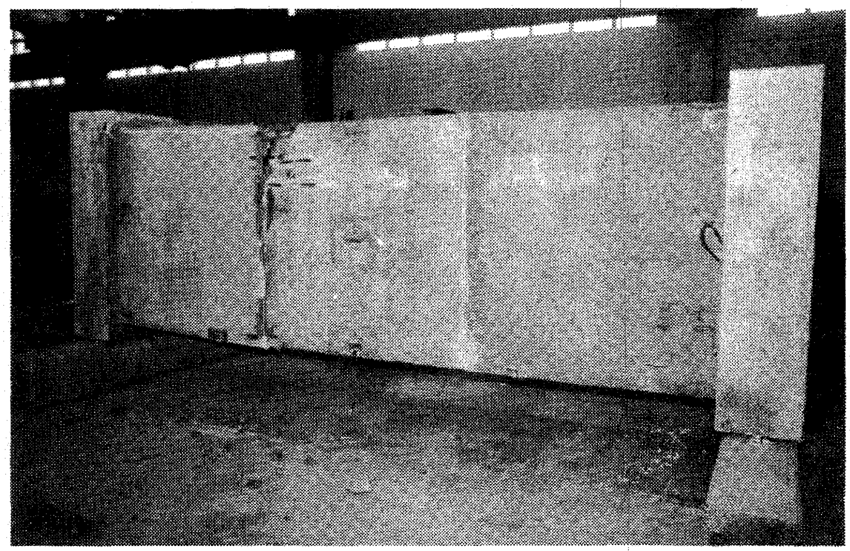


Figure 31: Specimen 2A
Orientation of specimen and overall view of north plate showing buckling in segment 6A.

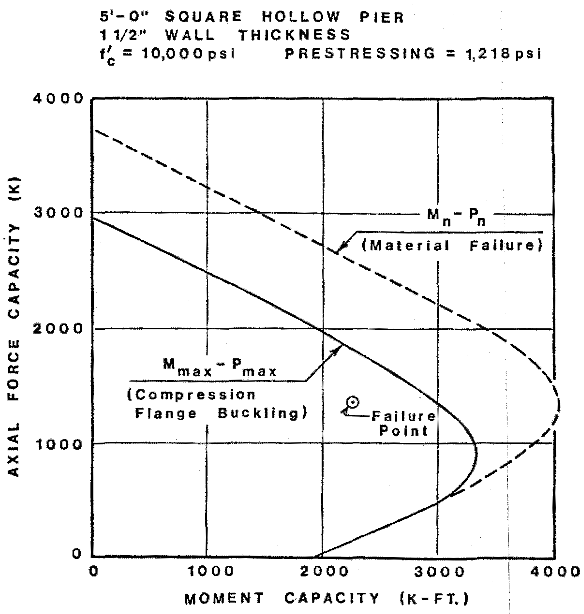


Figure 32: Specimen 2A — Interaction diagram.

CODE PROVISIONS

Requirements, such as minimum steel and concrete cover, limit the thinness of plates used in bridge construction to a condition where thin plate buckling generally is not a problem. Thin sections are not considered due to durability and impact concerns. Therefore, criteria for concrete thin plate buckling are not included in the current codes. With the advent of high quality dense concrete, these limitations have potential of being removed, which would then require the incorporation of buckling criteria into the code.

Current AASHTO specifications control thin plate buckling of steel members by restricting the width of plates through utilization of stiffeners. These restrictions are based on the studies of thin plate buckling characteristics of steel members. Similar design criteria has been developed for thin plates of concrete as a result of this study. The buckling failure mode for thin plates of concrete is brittle compared to the ductile behavior of steel. This brittle type failure indicates a conservative code provision should be adopted pending further studies of thin plate buckling. Therefore it is recommended that plate thinness be limited to

$$\frac{\text{width} - b}{\text{thickness} - t} \leq 10 \qquad \text{(EQ 2)}$$

without the rational analysis provided by this study. This limit applies only to rectangular hollow piers with k = 4.0. Other limits must be established for differing bridge member configurations. This limit would assure design of the pier would be controlled by overall stability rather than local plate buckling.

It was previously stated that the tensile strength and modulus of rupture of high strength concrete follow the trends established for normal strength concrete. This implies that high strength concrete flexural members designed to the normal allowable tension stress criteria will exhibit a superior margin of safety against cracking under service load conditions, because the nominal margin of $1.5\sqrt{f'_c}$ is approximately 40% greater for 10,000 psi (70 MPa) concrete than for 5,000 psi (35 MPa) concrete. Full advantage could be taken of this fact by revising allowable stress criteria in the following manner:

1.6.6—ALLOWABLE STRESSES

(B) Concrete

(2) Stress at Service Load After Losses have Occurred

Tension in the precompressed tensile zone

(a) For members with bonded reinforcement . . .

For normal weight concrete . . . . $7.5\sqrt{f'_c}-106$ psi

For sand-lightweight concrete . . . . $6.3\sqrt{f'_c}-106$ psi

For all other lightweight concrete . . . . $5.5\sqrt{f'_c}-106$ psi

(b) For severe corrosive exposure conditions, such as coastal areas . . .

For normal weight concrete . . . . $7.5\sqrt{f'_c}$—318 psi

For sand-lightweight concrete . . . . $6.3\sqrt{f'_c}$—318 psi

For all other lightweight concrete . . . . $5.5\sqrt{f'_c}$—318 psi

This proposed code revision would enable the designer to provide a constant margin of safety against flexural cracking in prestressed girders. That is, the same increase of applied moment over design service load moment will cause flexural cracking, irrespective of concrete strength. The allowable stresses given here are calibrated against the AASHTO standard of 5,000 psi (35 MPa) (see Article 1.6.6) and the present criteria of $6\sqrt{f'_c}$ and $3\sqrt{f'_c}$ allowable tension allowed for normal and severe exposure, respectively.

SUMMARY

Production and utilization of high strength concrete is rapidly becoming a viable concept in construction. Its application to precast prestressed concrete is readily apparent. The analytical design studies discussed demonstrate the benefits of using high strength concrete in flexural members as well as in compression members. These benefits include increased span lengths, reduced dead loads, and greater load capacities.

The potential of using thin plates of concrete fabricated on flat beds in the customary manner of the precast industry is shown to be advantageous. The analytical studies resulting in the development of a computer program make a rational analysis of these thin plates feasible. This would assure control of the section by other design criteria than local buckling of that thin plate.

The current AASHTO(8) specification is not conducive to the use of high strength concrete. Although use is not restricted, Article 1.6.6 does not encourage its use. Further, the restrictions on thickness of member and cover over steel do not permit full utilization of the high strength dense concrete that could be provided for thin sections. These criteria need to be constantly reviewed by code authorities and changes made similar to the provisions presented in this document. Continuing efforts should be made by professional bodies, governmental agencies, and code authorities to more rapidly implement into practice application of the knowledge being developed by numerous researchers on the application of high strength concrete to structural members.

EXECUTIVE SUMMARY

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH, DEVELOPMENT, AND TECHNOLOGY

The Offices of Research, Development, and Technology (RD&T) of the Federal Highway Administration (FHWA) are responsible for a broad research, development, and technology transfer program. This program is accomplished using numerous methods of funding and management. The efforts include work done in-house by RD&T staff, contracts using administrative funds, and a Federal-aid program conducted by or through State highway or transportation agencies, which include the Highway Planning and Research (HP&R) program, the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board, and the one-half of one percent training program conducted by the National Highway Institute.

The FCP is a carefully selected group of projects, separated into broad categories, formulated to use research, development, and technology transfer resources to obtain solutions to urgent national highway problems.

The diagonal double stripe on the cover of this report represents a highway. It is color-coded to identify the FCP category to which the report's subject pertains. A red stripe indicates category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, and green for category 9.

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1. Highway Design and Operation for Safety
Safety RD&T addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act. It includes investigation of appropriate design standards, roadside hardware, traffic control devices, and collection or analysis of physical and scientific data for the formulation of improved safety regulations to better protect all motorists, bicycles, and pedestrians.

2. Traffic Control and Management
Traffic RD&T is concerned with increasing the operational efficiency of existing highways by advancing technology and balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, coordinated signal timing, motorist information, and rerouting of traffic.

3. Highway Operations
This category addresses preserving the Nation's highways, natural resources, and community attributes. It includes activities in physical maintenance, traffic services for maintenance zoning, management of human resources and equipment, and identification of highway elements that affect the quality of the human environment. The goals of projects within this category are to maximize operational efficiency and safety to the traveling public while conserving resources and reducing adverse highway and traffic impacts through protections and enhancement of environmental features.

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Pavement RD&T is concerned with pavement design and rehabilititation methods and procedures, construction technology, recycled highway materials, improved pavement binders, and improved pavement management. The goals will emphasize improvements to highway performance over the network's life cycle, thus extending maintenance-free operation and maximizing benefits. Specific areas of effort will include material characterizations, pavement damage predictions, methods to minimize local pavement defects, quality control specifications, long-term pavement monitoring, and life cycle cost analyses.

5. Structural Design and Hydraulics
Structural RD&T is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highway structures at reasonable costs. This category deals with bridge superstructures, earth structures, foundations, culverts, river mechanics, and hydraulics. In addition, it includes material aspects of structures (metal and concrete) along with their protection from corrosive or degrading environments.

9. RD&T Management and Coordination
Activities in this category include fundamental work for new concepts and system characterization before the investigation reaches a point where it is incorporated within other categories of the FCP. Concepts on the feasibility of new technology for highway safety are included in this category. RD&T reports not within other FCP projects will be published as Category 9 projects.

TNR 10/11-87/2 1M/QE