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PRELIMINARY DESIGN

f'_c = compressive strength of concrete at service
f'_{ci} = compressive strength of concrete at time of initial prestress

Preliminary Design

6.0 SCOPE

Preliminary design is usually the first step in designing an economical precast, prestressed concrete bridge. This chapter discusses the preliminary plan, superstructure and substructure considerations, foundations, and member selection criteria with design aids and examples. Additional information is given in Chapter 4, "Strategies for Economy."

6.1 PRELIMINARY PLAN

6.1.1 General

The preliminary planning process consists of collecting and analyzing site information, applying established policies and practices, and considering alternatives including cost evaluations, for the purpose of providing the bridge that is the most cost-effective and the most functionally, structurally and aesthetically appropriate. The preliminary plan lays the groundwork for the final bridge design. It specifies the structure type and is the basis for the design schedule estimate and construction cost estimate.

6.1.2 Development

The preliminary planning process begins with bridge site data. Preliminary studies (such as type, size and location (TS&L) studies), geometric data, foundation data and hydraulic data are reviewed. Preliminary geometric approval is received. Struc-ture alternatives are evaluated considering such details as length, type, geometric constraints such as vertical and horizontal clearances, span arrangement, staging, falsework, substructure requirements, environmental and community issues, and costs. Plan, elevation and section views are developed and approved. Cost estimates are prepared. The preliminary plan and cost estimate are approved prior to beginning final design.

6.1.3 Factors for Consideration

A number of factors should be addressed at the preliminary design stage.

6.1.3.1 General

Funding classification (for example, state funds, federal and state funds, local funds) and available funding level should be determined. Environmental concerns include site conditions (for example, wetlands or environmentally sensitive areas) and mitigating measures.

6.1.3.2 Site

Site requirements that should be determined include topography, horizontal alignment (curves and skews), required clearances, vertical alignment and limits, superelevation, and existing and proposed utilities. Safety considerations include sight distances, horizontal clearance to piers, and hazards to pedestrians.

End slopes are controlled by soil conditions and stability, right-of-way availability, fill height or depth of cut, roadway alignment and functional classification, and existing site conditions.

6.1.3.3 Structure

Structural considerations include foundation and groundwater conditions, requirements for future widening, and anticipated settlement. Aesthetics, including general appearance, level of visibility and compatibility with surroundings and adjacent structures should be evaluated. Railroad separations may require negotiations with the railroad company concerning clearances, geometry, utilities, drainage and provision for maintenance roads.

The total length of the bridge is based on horizontal and vertical clearances to roadway(s) or rail(s) below or above, or hydraulic studies if over water, and/or environmental concerns or other restrictions as set by the owner agency. The bridge width is typically controlled by the geometry of the approaching roadway. The span arrangement is controlled by such factors as:

- Allowable girder depth due to clearance requirements
- Placement of piers in waterways
- Horizontal clearance between supports and rights-of-way below
- Economic ratio of end span to interior span

Considering the ratios of spans, the following have been found to produce a balanced design, where the reinforcement requirements for end spans are comparable to those for interior spans:

End span/interior span	Condition
0.95	Simple span for girder and deck weight, continuous span for all other loads
0.80	Simple span for girder weight, continuous span for all other loads

As previously discussed, bridge details are largely dictated by obstructions above and below ground, maximum span limitations, and required abutment locations. However, to the extent possible, large skews, steep profile grades, sharp horizontal curves and differing span lengths should be avoided. Slightly lengthening the bridge may be preferable to using an extreme skew angle that tightly fits the bridge site.

6.1.3.4 Hydraulics

Hydraulic considerations include bridge deck drainage, stream flow conditions and channel drift, passage of flood debris, scour and the effect of the pier as an obstruction (for example, the pier's shape, width, skew, number of columns), banks and pier protection, permit requirements for navigation, and stream work limitations. After piers have been located, specific information on scour and backwater is obtained.

Vertical clearances for water crossings should satisfy floodway clearance requirements. In accordance with the flood history, nature of the site, character of drift, and other factors, the minimum vertical clearance (for the 100-year flood, for example) is determined. The roadway profile and the bridge superstructure depth should accommodate this clearance requirement. Bridges over navigable waters should also comply with any clearance requirements of the U.S. Coast Guard.

6.1.3.5 Construction

Construction considerations include falsework and other construction clearances, working space requirements, hauling and erection details, access to the site, construction season, and construction scheduling limitations. Safety considerations such as traffic flow, staging, detours and falsework requirements should be addressed.

Access routes should be checked and sites reviewed to ensure that the precast concrete beams can be transported to the site. Possible routes to the site should be adequate to handle the truck and trailer which are hauling the beams. Generally, the designer is not responsible for construction of the bridge. However, prudent designers always consider constructability issues. Therefore, it is recommended that both size and weight of the beams be checked and hauling permit requirements determined. The details related to erecting the beams once they reach the site also need to be assessed. The site should be reviewed for adequate space for the contractor to set the cranes and equipment necessary to lift and place the beams.

6.1.3.6 Utilities

Often, electric, water, telephone and other utility conduits are required to be supported by the bridge. Most loads imposed by these utilities, except perhaps those of large water pipes, do not have significant impact on structural design. However, aesthetics and accessibility to utility lines, as well as relocation of existing utilities, may affect the selection of the superstructure system.

6.1.4 Required Details

The preliminary plan should include, as a minimum, the following details (see **Figure 6.1.4-1**):

- Location, including highway identification, name of city or county, and major features crossed
- Total length
- Total width
- Span arrangement with expansion joint locations
- Abutment and pier type with dimensions
- Foundation type with dimensions
- End slopes, with type and rate
- Profile grade and superelevation diagram
- Horizontal alignment
- Hydraulic data
- Cross-section, including barrier type and wearing surface type
- Beam type, number and spacing
- Deck thickness and build-up dimensions, if applicable
- Minimum vertical and horizontal clearances, with dimensions
- Utilities
- Borings
- Superstructure bearing types (expansion, fixed, guided, ... etc.)
- Design method (or specification)
- Design loads

6.2 SUPERSTRUCTURE

6.2.1 Beam Layout

Redundant supporting elements minimize the risk of catastrophic collapse. A typical guideline would recommend a minimum of four beams (webs). This number allows the bridge to be repaired in phases under traffic. For roadways less than 30 ft wide, a minimum of three beams (webs) may sometimes be justified.

When establishing beam layout, deck overhangs should be limited to 0.50 times the beam spacing. In some cases, this ratio has been increased to 0.625. However, large overhangs may require more costly form erection brackets and provisions to prevent overturning of the exterior beams.

Design aids are provided at the end of this chapter to assist with superstructure system selection for preliminary design.

6.2.2 Jointless Bridges

By using integral abutments at bridge ends, long continuous jointless bridge construction is possible with prestressed concrete beams. Some proponents believe that lengths on the order of 1,000 ft are realistic with this construction method. The elimination of joints minimizes beam end deterioration from inadequate protection from leaking joints and deleterious materials, such as deicing chemicals applied to the deck. Chapter 13 has more information on integral bridges.

6.3 SUBSTRUCTURES

6.3.1 Piers

In selecting the pier type, preliminary designs should be made for various configurations to evaluate costs. The most economical pier may not be the one with the least material, but instead, the one that is easiest to form and that maximizes repetitive use of forms. This is especially true on large bridge projects.

The most commonly used pier types are illustrated in **Figure 6.3.1-1** and discussed below.

6.3.1.1 Open Pile Bents

Open pile bents are used on low-volume roads and stream crossings where the possibility of debris entrapment between piles is not likely. Open pile bents are extremely economical. This type can be readily combined with precast concrete pile caps to permit very rapid construction.

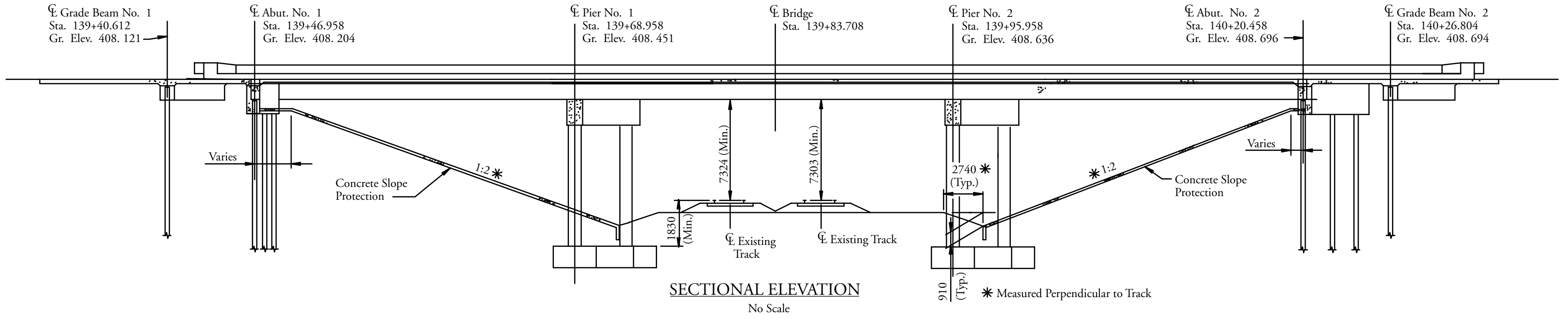
6.3.1.2 Encased Pile Bents

Encased pile bents are used in water crossings where the channel carries debris or where protection against ice is desired. This pier type is usually preferred when scour is a concern and spans are of medium length.

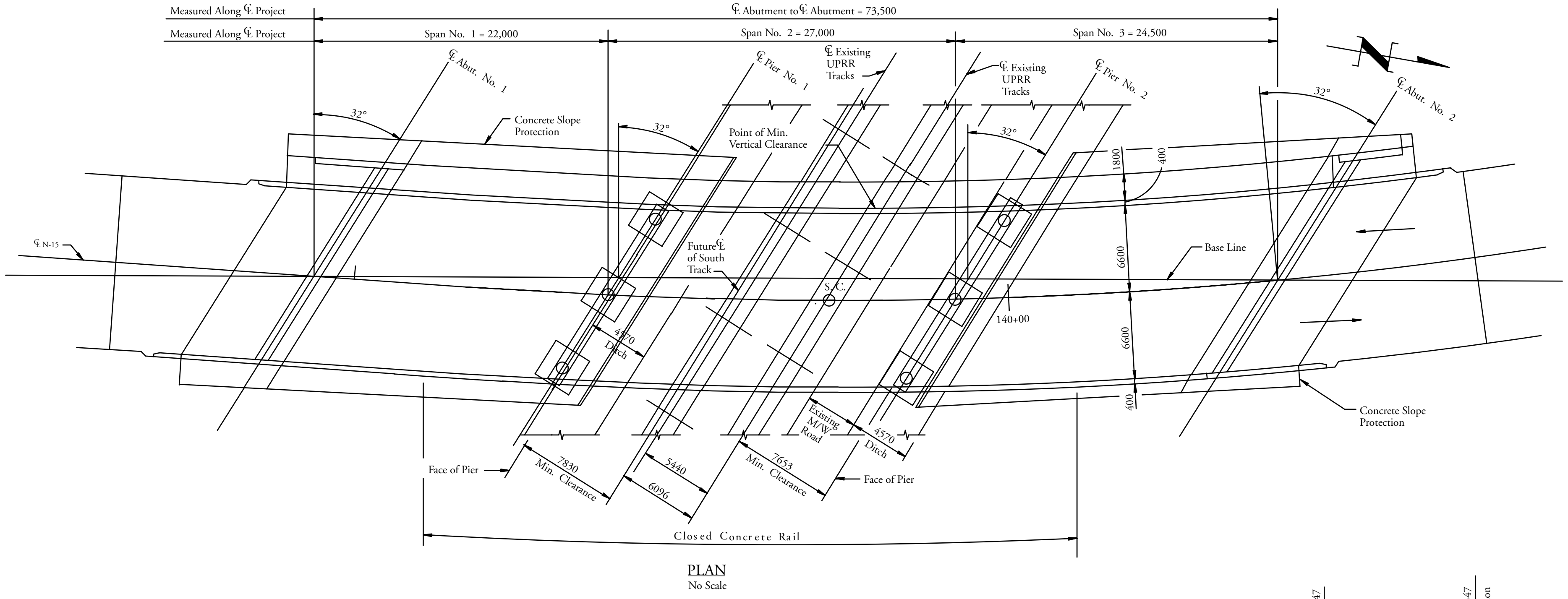
6.3.1.3 Hammerhead Piers

With increasing pier height, the hammerhead pier becomes more economical, since this type offers a reduction in material and forming. Hammerhead piers are sometimes used as crash walls when constructed adjacent to railroad tracks. Other types of piers may also be used next to railroads as long as sufficient crash wall requirements are provided.

Figure 6.1.4-1 Example Preliminary Plan



SECTIONAL ELEVATION
No Scale



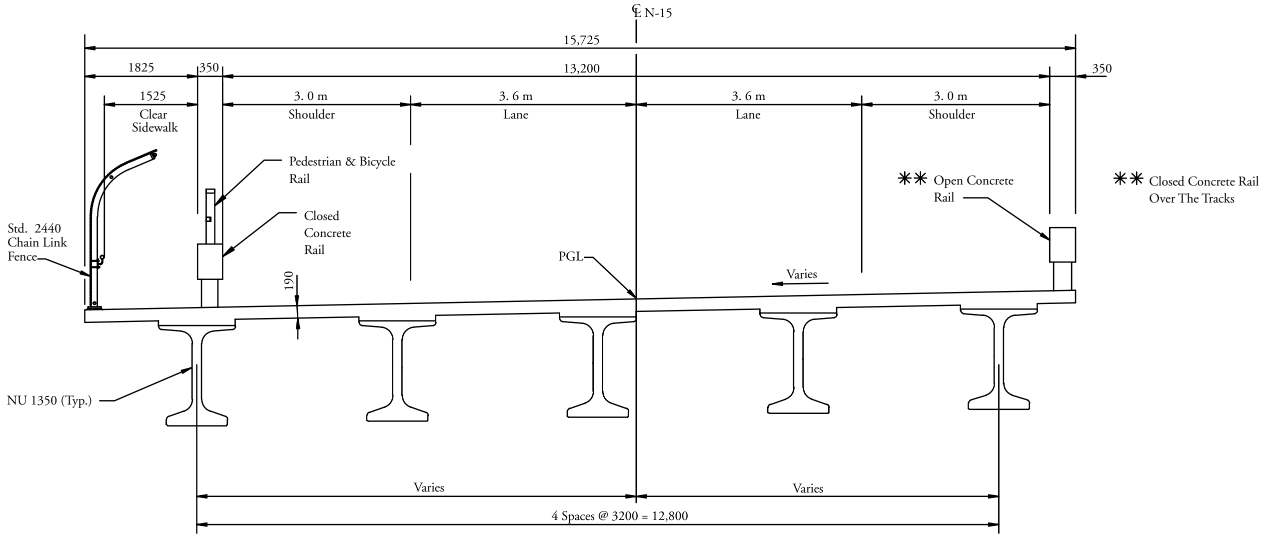
PLAN
No Scale

ALL DIMENSIONS ARE MILLIMETERS (mm)
UNLESS OTHERWISE NOTED.

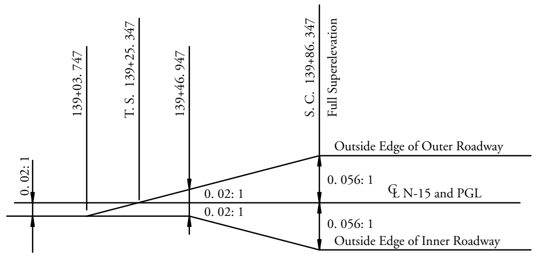
Horizontal Alignment and Vertical Profile shown are at Profile Grade Line.

Top of slab elevations are at Profile Grade Line.

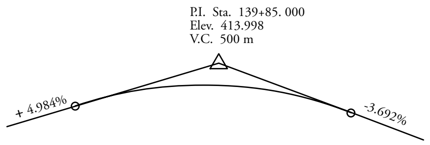
Pile size, type and capacity shall be provided in final design by the Nebraska Department of Roads.



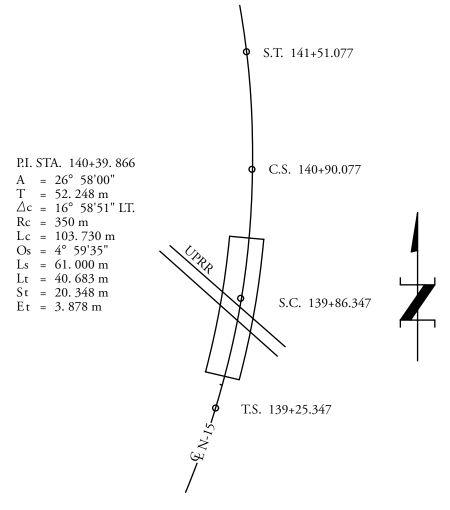
TYPICAL CROSS SECTION



SUPERELEVATION DIAGRAM
No Scale

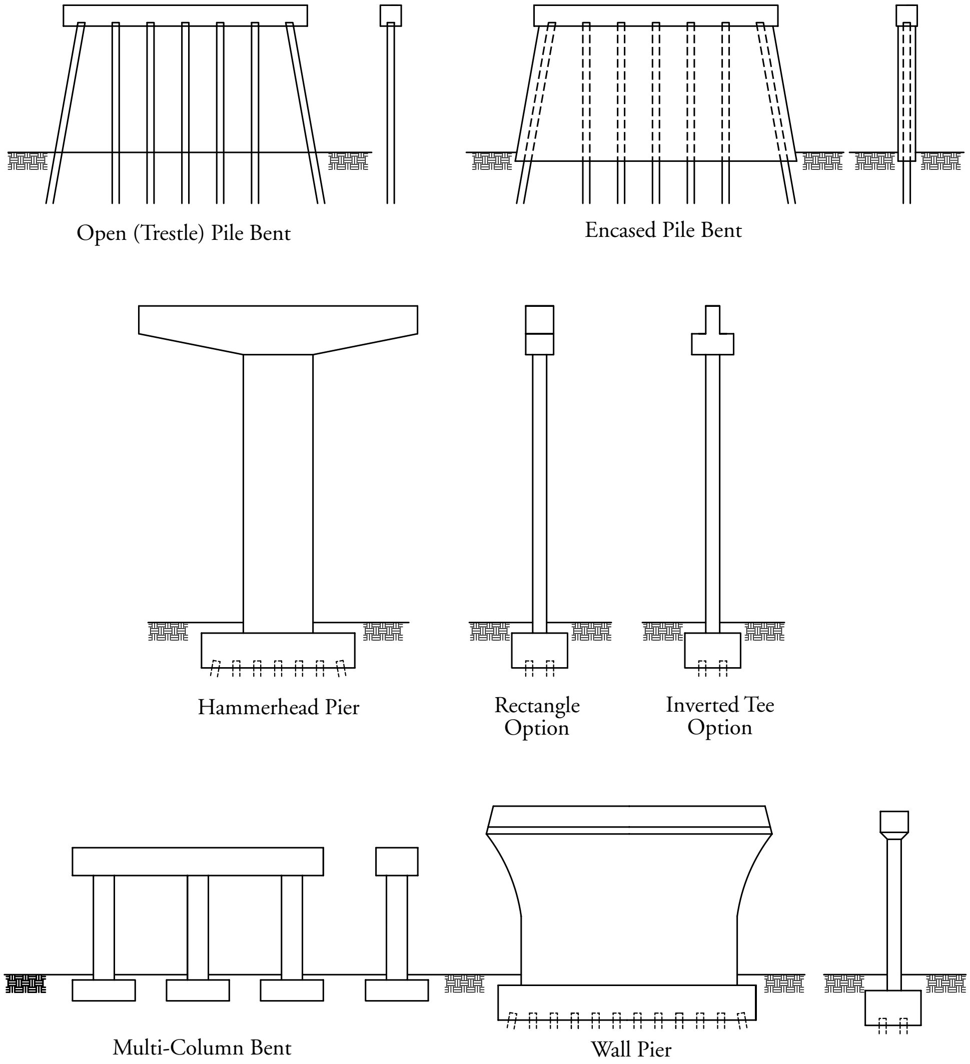


VERTICAL PROFILE - N-15
No Scale



HORIZONTAL ALIGNMENT
No Scale

Figure 6.3.1-1
Types of Commonly Used Piers



6.3.1.4 Multi-Column Bents

Multi-column bents are sometimes referred to as rigid frame piers. Basically, this pier type is a concrete beam supported on at least two columns. It is used for wide superstructures and longer spans. Generally, a round column is the simplest and the most economical shape since forms are commercially available and require no form ties. This reduces labor considerably. Forms for this type of pier are most likely found in a typical contractor's inventory. Columns may be extensions of piles or drilled shafts.

In situations where vertical clearance is a concern, a cap shaped like an inverted tee may be used to reduce the depth of cap beneath the superstructure.

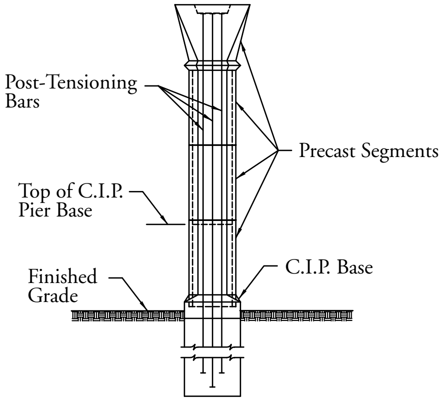
6.3.1.5 Wall Piers

Used primarily for river crossings, a wall pier is typically constructed as a combination of a solid shaft and hammerhead pier.

6.3.1.6 Segmental Precast Piers

Precast concrete segmental piers can be thin-walled hollow segments, match-cast or mass-produced with a thin mortar bed between segments. Shims can be used to maintain proper vertical alignment. The mortar bed should be designed to resist the anticipated loads, provide a thorough closure of the joint, and be designed with permissible creep and shrinkage characteristics.

Figure 6.3.1.6-1
Segmental Concrete Pier Column



Post-tensioned threaded bars are generally inserted in ducts cast in the segments and stressed. Later, ducts are grouted solid. Another alternative is the use of splice sleeves that couple reinforcing bars to provide full bar capacity. **Figure 6.3.1.6-1** shows a column designed and built with precast segments.

6.3.2 Abutments

Unlike piers, abutment types do not vary widely. The most common types of abutments are the backwall type and the integral type. For more information on integral abutments, see Chapter 13. Among the advantages of the integral type is the elimination of the deck joint, which often leaks and causes deterioration, and is therefore a maintenance problem. Integral abutments are flexible and tolerate movement caused by expansion and contraction of the superstructure due to temperature changes. It may be necessary, however, to use a backwall abutment if bridge length or skew dictate.

For precast abutment walls, full capacity may be accomplished by means of field welding of connecting steel plates, followed by corrosion protection of exposed steel.

Location of the abutments is a function of the profile grade of the bridge, the minimum vertical and horizontal clearances required, and the type and rate of end slope.

6.3.3 Hydraulics

Pier shapes that streamline flow and reduce scour are recommended. Consideration is based on the anticipated depth of scour at the bridge piers. Measures to protect the piers from scour activity (for example, riprap and pier alignment to stream flow) are recommended.

For bridges over navigable channels, piers adjacent to the channel may require pier protection as determined by the U.S. Coast Guard. The requirement is based on the horizontal clearance provided for the navigation channel and the type of navigation traffic using the channel. In many cases, piers over navigable waterways should be designed to resist vessel impact in accordance with AASHTO requirements.

6.3.4 Safety

Due to safety concerns, fixed objects should be placed as far from the edge of the roadway as economically feasible, maintaining minimum horizontal clearances to bridge piers and retaining walls.

Redundant supporting elements minimize the risk of catastrophic collapse. A typical guideline would recommend two columns minimum for roadways from 30 to 40 ft wide and three columns minimum for roadways over 40 to 60 ft wide. Also recommended is collision protection or design for collision loads on piers with one or two columns.

6.3.5 Aesthetics

The principal direction of view of the piers should be considered when determining their size, shape and spacing. The piers should be correctly sized to handle the structural loads required by the design and shaped to enhance the aesthetics of the overall structure. Column spacing should not be so small as to create the appearance of a "forest of columns." Chapter 5 discusses aesthetics in greater detail.

6.4 FOUNDATIONS

Typical foundation types include:

- Spread footings
- Prestressed concrete piles
- Drilled shafts
- Steel H-piles
- Steel pipe piles
- Timber piles

Round or square columns of multi-column bents, usually rest on single drilled shafts or on footings that cap multiple piles. Single columns usually rest on footings that cap multiple piles or drilled shafts.

Prestressed concrete piles are used extensively in the coastal regions, as well as other locations. For short bents on stream crossings, a line of piles may be extended into the cap, forming a trestle pile bent. These are economically competitive even when the soil is suitable for drilled shafts.

Prestressed piles can double as foundations and piers, thus reducing the amount of on-site forming and concreting. Precast, prestresseed concrete piles come in different sizes and shapes, ranging from 10 x 10-in.-square piles to 66-in.-diameter hollow cylinder piles.

6.5 PRELIMINARY MEMBER SELECTION

6.5.1 Product Types

The standard AASHTO-PCI sections given in Appendix B were used for development of the charts. It should be noted that neither the deck bulb-tee nor the double stemmed (double tee) sections are AASHTO standard products. However, they are included due to common use of these products in several states and due to their cost-effectiveness, particularly for bridges on secondary roads.

A number of states have their own standard products. Designers should check with their local precast concrete producers on product availability before they begin design. This manual may include similar design charts for local products which have been supplied by local or regional organizations. Until these design aids are available, the charts given here may be used as a convenient means of assessing the appropriateness of various product shapes, depths and spacings for a given span arrangement. This should, of course, be followed by detailed design incorporating locally available products.

6.5.2 Design Criteria

The design charts provided in this chapter were developed to satisfy flexure according to AASHTO *Standard Specifications.* However, use for preliminary member selection according to AASHTO *LRFD Specifications* should still be valid.

6.5.2.1 Live Loads

The live load considered for the charts is the HS25 truck which is 1.25 times the standard HS20 truck. This relatively heavy load is consistent with the practices of several states and generally produces designs that are similar to those produced by the AASHTO *LRFD Specifications.* The live load distribution factor is taken here as (spacing/5.5), the factor used in AASHTO *Standard Specifications* for I-beam systems. This factor is generally reasonable for the purpose of establishing the preliminary charts for all products. This assumption was made to simplify development of the charts for adjacent members such as box beams, deck bulb-tees, double-stemmed beams and voided slab beams.

6.5.2.2 Dead Loads

In many areas, adjacent members are constructed without a composite topping. In developing the charts, these members are assumed to have an additional dead load of 50 psf representing an overlay plus barriers and railing. Members with a cast-in-place composite topping are assumed to carry their own weight plus the topping weight as non-composite members, and 40 psf superimposed dead load as composite members. The 40 psf includes allowance for barriers, railing and 25 psf for a future wearing surface.

6.5.2.3 Composite Deck

In all cases of spread sections, an 8-in. thick composite topping (deck) is assumed to be used. Also for these spread sections, a 1/2 in. haunch is assumed placed directly over the top flange. It is accounted for in computing loads and section properties.

For adjacent sections which are recommended to have a composite topping, the topping thickness is assumed equal to 6 in. The topping weight is based on the indicated thickness. However, composite section properties were determined with the assumption that long-term wear reduces the thickness by 0.5 in. The 8-in. slab thickness should be adequate except for beam spacings larger than about 12 ft. The charts should still be valid for slightly wider spacings, say up to 14 ft, as the increased section properties should offset the increased weight.

6.5.2.4 Concrete Strength and Allowable Stresses

Except for the charts that include information on high strength concrete, concrete strength was kept the same for all charts. The precast concrete products are assumed to have $f'_{ci} = 5{,}500$ psi and $f'_c = 7{,}000$ psi, and the cast-in-place topping is assumed to have $f'_c = 4{,}000$ psi.

The allowable concrete tensile stresses are taken as $7.5\sqrt{f'_{ci}}$ at release and $6\sqrt{f'_c}$ at service. The allowable compression is taken as $0.6f'_{ci}$ at release and $0.6f'_c$ at service.

6.5.2.5 Strands and Spacing

One-half inch diameter, seven-wire, 270 ksi low-relaxation strands are used in all applications except where high strength concrete is indicated. The center-to-center strand spacing is assumed to be 2 in., although that spacing can be reduced to 1-3/4 in., according to Federal Highway Administration recommendations (1996).

All strands are assumed to have an initial tension of 202.5 ksi prior to release. Member end stresses are assumed to be controlled through debonding (shielding) and/or harping of some of the strands as needed. Prestress losses are calculated assuming 75% relative humidity. When the relative humidity is less as in several regions, prestress losses would be higher and member capacity reduced. Therefore, points on the charts above the dashed line ($f'_{ci} = 5{,}500$ psi) must be used with caution as values are highly sensitive to design assumptions.

Strand patterns used by producers vary. For the box beams in the charts in Section 6.9, two layers of strands are assumed in the bottom flanges.

6.5.2.6
Design Limits

The charts indicate whether tensile stress at service or strength controls the maximum span limit. A third controlling criterion may be compressive stress at release. If this is the case, the curves are continued until either tension at service or strength controls, with the ending point labeled with the minimum value of f'_{ci} required to allow its use. In no case was the end point allowed to have f'_{ci} greater than f'_c.

For the longer spans, camber and stability of the beams should also be evaluated.

6.5.3
High Strength Concrete

According to recent surveys, little difficulty is encountered anywhere in the country in obtaining 7,000 psi concrete on a consistent basis. Some owners and designers still specify 5,000 psi concrete, a practice which unnecessarily penalizes precast concrete bridge products.

6.5.3.1
Attainable Strengths

In recent years, higher strength concretes have been commercially achieved. The strength ranges from 10,000 to 15,000 psi. Use of such strengths is expected to increase in the future. Therefore, design charts for the AASHTO I-beams and bulb-tee beams include span capacities with concrete having f'_{ci} = 8,000 psi and f'_c = 12,000 psi. This limited coverage is intended to introduce the significant impact of using higher strength concrete on two common products. The increased span capacity should be weighed against the possible cost increase associated with producing higher strength concrete. Chapter 4 discusses many of these considerations.

6.5.3.2
Limiting Stresses

Where high strength concrete is used, the allowable tensile stress is increased to $10\sqrt{f'_{ci}}$ at release and $8\sqrt{f'_c}$ at service. Both of these limits have been justified by several recent studies. This 33 percent increase in allowable tensile stress, from 7.5 to 10 and from 6 to 8, has not been recognized by AASHTO specifications. Its impact on design is much less significant than that of the allowable concrete compressive stress, especially at release. The allowable compressive stresses are $0.6\,f'_{ci}$ and $0.6\,f'_c$ as assumed for normal strength concrete.

6.5.3.3
Larger Strands

In order to utilize the full potential of high strength concrete, it is sometimes necessary to use 0.6 in. diameter strands. These larger strands provide about 40 percent higher tensile capacity than for 1/2 in. diameter strands at only about 20 percent increase in diameter. The curves for high strength concrete were developed with 0.6 in. strands at 2 in. spacing. This spacing is in accordance with the provisions of a 1996 FHWA memorandum.

6.6
DESCRIPTION OF DESIGN CHARTS

6.6.1
Product Groups

The design charts provide preliminary design information for different products grouped into several types. These include:

CHARTS	**PRODUCTS**
Charts BB-1 through BB-10	AASHTO Box beams
BT-1 through BT-4	AASHTO-PCI Standard bulb-tees

DBT-1 through DBT-5	Deck bulb-tees
IB-1 through IB-7	AASHTO Standard I-beams
DT-1 and DT-2	Double-stemmed beams (double tees)
SB-1 through SB-3	AASHTO Voided slab beams

(Geometric properties for all products are given in Appendix B.)

6.6.2 Maximum Spans Versus Spacings

Within each group, the first chart, e.g. BB-1, BT-1,... etc., depicts the maximum attainable span versus member spacing for all member depths within the group. This type of chart is convenient to use in the early stages of design to identify product types, spacings and approximate depths for the span length being considered.

6.6.3 Number of Strands

The remainder of the charts within each group give the number of strands needed for specified span lengths and beam spacings. This type of information is needed to: (1) develop an estimate of the final design requirements, and (2) to determine if the number of strands needed is within the prestressing bed capacity of local producers. Otherwise, the member depth, or spacing if applicable, must be adjusted.

In developing the charts, no attempt was made to judge whether or not the number of strands given is feasible for local production. For example, in Chart IB-7 for AASHTO Type VI I-beams, only a very limited number of precast producers in the country are likely to have a prestressing bed capable of resisting 90 tensioned strands. The number of strands was strictly based on flexural stress requirements. In some cases, e.g. shallow I-beams at wide spacing, shear capacity may not meet maximum limits of the *Standard Specifications*. A complete check should be made during final design.

It should be noted that all charts were based on providing the lowest possible center of gravity of strands in the midspan section. This is accomplished by filling the first (bottom) row to capacity before any strands can be placed in the second row, and so on.

6.6.4 Controls

In certain situations, compressive strength at release controls the maximum span capacity. This is indicated in the charts by a crossing line labeled with the specified value of f'_{ci} . However, the line representing number of strands versus span is continued as a thinner line until another design criterion (usually tension at service) controls or until the required value of f'_{ci} reaches the assumed value of strength at service, f'_c .

Because of the recent increase in use of higher strength concrete, the charts of I-beams and bulb-tee beams include span capacities and number of strands required when the precast product strength, f'_c equals 12,000 psi. As can be seen, significant improvement in span capacity can be realized. Part of the improvement is due to the use of 0.6 in. diameter strands.

6.7 PRELIMINARY DESIGN EXAMPLES

6.7.1 Preliminary Design Example No. 1

Design a simple span for HS25 loading with a 95 ft design span. The total width of the bridge is 28'-0". The conditions do not allow for field forming of the concrete deck.

Referring to the preliminary design charts, the only applicable products would be adjacent box beams or deck bulb-tees in order to avoid deck forming. Using the charts, all possible solutions are summarized in **Table 6.7.1-1.**

Table 6.7.1-1 Product Options for Example No. 1[1]

Product		Depth, in.	Spacing, in.	Topping (Deck)	Number of strands	Design Chart
		35	48	No	36	DBT-2
Deck Bulb-Tee		53	48	No	20	DBT-3
		65	48	No	15	DBT-4
AASHTO	BII-36	33	36	No	27	BB-8
Box		33	36	Yes	28	BB-8
Beam	BIII-36	39	36	No	22	BB-9
		39	36	Yes	22	BB-9
	BIV-36	42	36	No	25	BB-10
		42	36	Yes	27	BB-10
	BI-48	27	48	Yes	50	BB-2
	BII-48	33	48	No	34	BB-3
		33	48	Yes	35	BB-3
	BIII-48	39	48	No	28	BB-4
		39	48	Yes	28	BB-4
	BIV-48	42	48	No	26	BB-5
		42	48	Yes	26	BB-5

Note 1. Refer to Section 6.5 for design assumptions.

From the table above, the deck bulb-tee generally requires more depth, but fewer strands. Please note that the product may not be available in all regions.

While the 36-in.-wide box beam is an option, the bridge width is not divisible by 3 ft. Further, unless weight of a single beam is a factor, wider units allow casting, transporting and installing fewer pieces. This usually results in lower cost.

Detailed Design Example 9.1, Chapter 9, has similar span, width and loading requirements. In that example, an AASHTO BIII-48 box beam was used. Considering **Table 6.7.1-1**, it is clear that a shallower section could be used, or, it could be reasoned, a lower concrete strength.

6.7.2 Preliminary Design Example No. 2

Design a simple span for HS20 loading with 120 ft design span. The total width of the bridge is 51'-0" with a cast-in-place deck slab 8-in. thick. **Table 6.7.2-1** shows the product options and the number of strands required for each product.

It is generally most beneficial to use the widest possible spacing to minimize the number of beam lines. Clearance requirements may dictate the structure depth. Assuming no maximum depth limitations, the most economical products will be the deepest in order to minimize the number of strands required. Accordingly, an AASHTO Type VI I-beam or 72 in. deep bulb-tee (BT-72) at a 9 ft spacing are recommended. However, since the bulb-tee is a lighter section and the number of strands required (47 strands) is about the same, a BT-72 at a 9 ft spacing will be a more efficient solution.

Table 6.7.2-1
Product Options for Example No. 2[1]

Products		Depth, in.	Spacing, ft	Topping (Deck)	Concrete strength, psi	Number of strands	Design Chart
AASHTO I-Beam	IV	54	6-8	Yes	12,000	34-45	IB-5
		54	6	Yes	7,000	53	IB-5
	V	63	6-12	Yes	12,000	22-52	IB-6
		63	6-10	Yes	7,000	40-68	IB-6
	VI	72	6-12	Yes	12,000	22-40	IB-7
		72	6-12	Yes	7,000	32-65	IB-7
AASHTO Bulb-Tee	BT-54	54	6-8	Yes	12,000	29-47	BT-2
	BT-63	63	6-10	Yes	12,000	22-45	BT-3
		63	6	Yes	7,000	38	BT-3
	BT-72	72	6-12	Yes	12,000	20-45	BT-4
		72	6-9.5	Yes	7,000	30-55	BT-4
Deck Bulb-Tee		53	4-6	Yes	7,000	29-48	DBT-3
		65	4-8	Yes	7,000	24-46	DBT-4
AASHTO Box Beam	BIII-36	39	3	No	7,000	36	BB-9
		39	3	Yes	7,000	43	BB-9
	BIV-36	42	3	No	7,000	40	BB-10
	BIII-48	39	4	No	7,000	45	BB-4
		39	4	Yes	7,000	50	BB-4
	BIV-48	42	4	No	7,000	41	BB-5
		42	4	Yes	7,000	45	BB-5

Note 1: Refer to Section 6.5 for design assumptions.

A deck bulb-tee can be utilized for this bridge if the product is locally available. An AASHTO box beam is also suitable if the superstructure depth needs to be relatively shallow.

Detailed Design Example 9.3, Chapter 9, has a 120 ft simple span, concrete strength of 6,500 psi and HS20 loading conditions. Referring to the above table, the BT-72 was chosen with 9 ft spacing.

6.8 REFERENCES

AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, First Edition, Washington, DC, 1994

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Bridge Design Manual, Washington State Department of Transportation, Program Development Division, Bridges and Structures, July 1996

Goodspeed, C.H., Vanikar, S., and Cook, R.A., “High-Performance Concrete Defined for Highway Structures,” Concrete International, February 1996, pp. 62-67

Gordon, Stanley, Federal Highway Administration, U.S. Department of Transportation, Memoranda dated October 26, 1988 and May 8, 1996

Precast/Prestressed Concrete Short Span Bridges – Spans to 100 Feet, Second Edition, Precast/Prestressed Concrete Institute, Chicago, IL, 1985

Prestressed Concrete Manual and Bridge Manual, Illinois Department of Transportation, Bureau of Bridges and Structures, Springfield, Illinois, January 1994

Standard Specifications for Highway Bridges, 16th Edition, American Association of State Highway and Transportation Officials, Washington, DC, 1996

6.9 PRELIMINARY DESIGN CHARTS

Chart BB-1 AASHTO Box Beams- 48 in. Wide

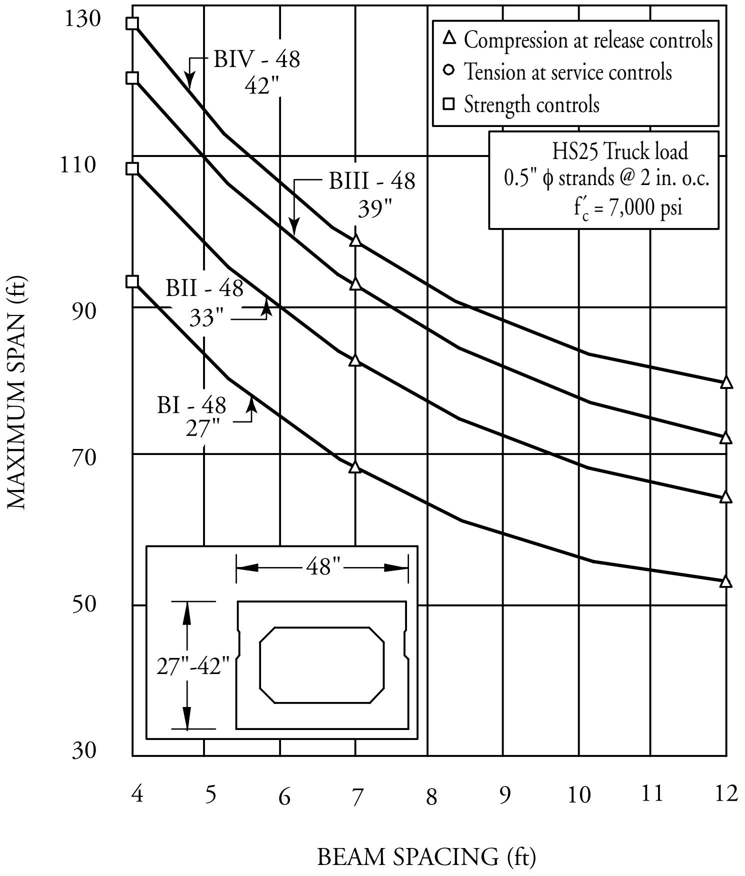

Chart BB-2 AASHTO Box Beams-BI-48

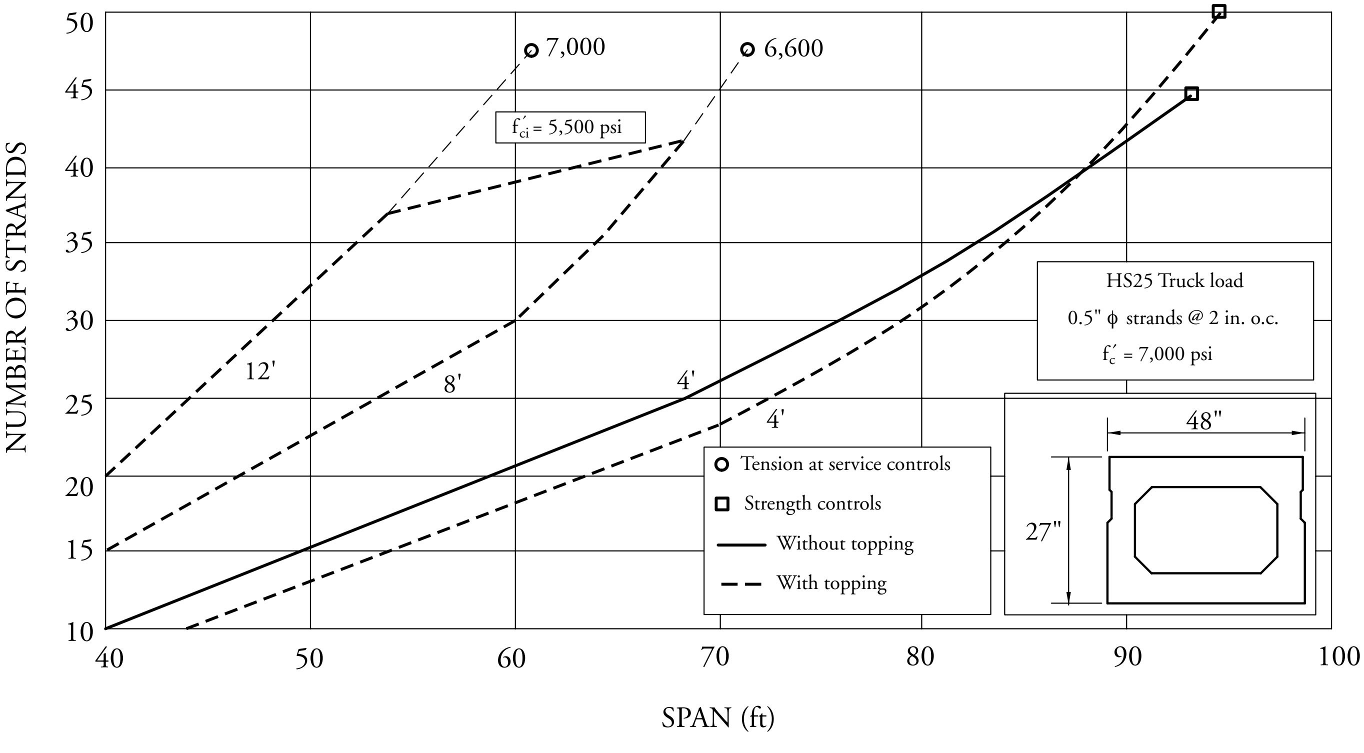

Chart BB-3
AASHTO Box Beams-BII-48

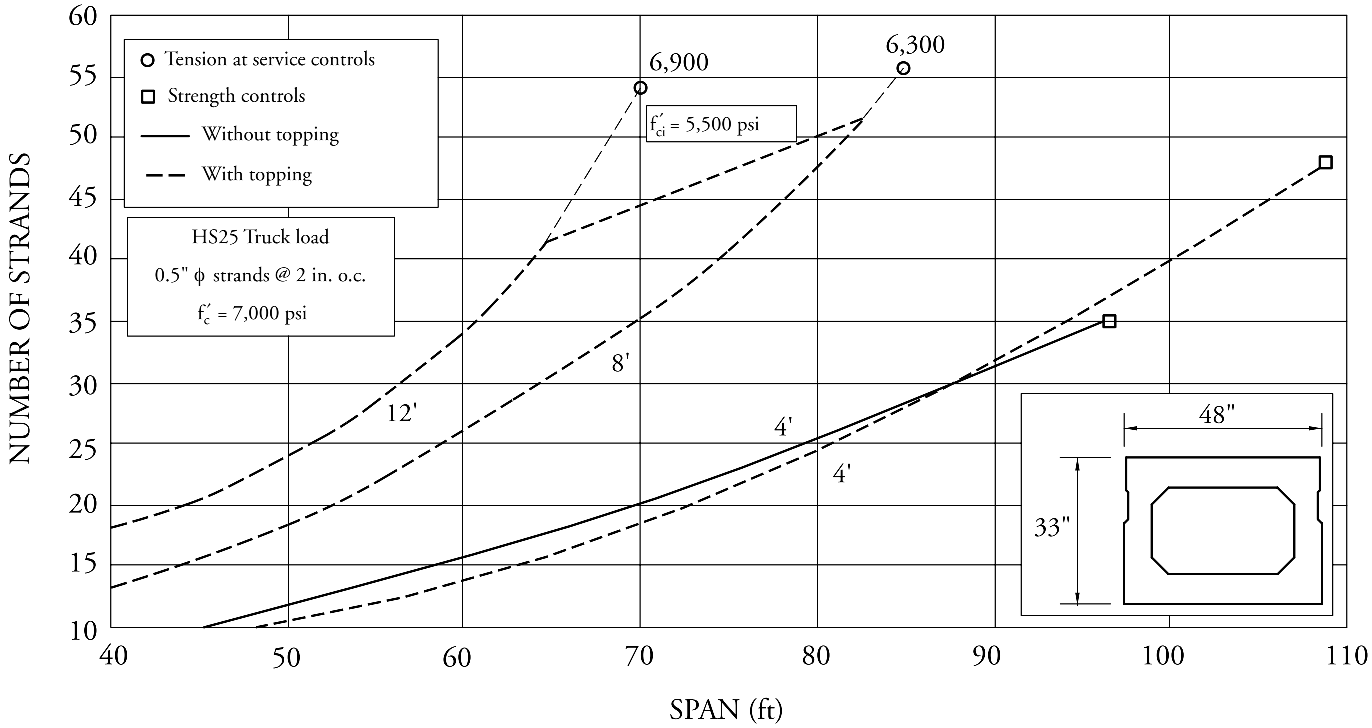

Chart BB-4
AASHTO Box Beams-BIII-48

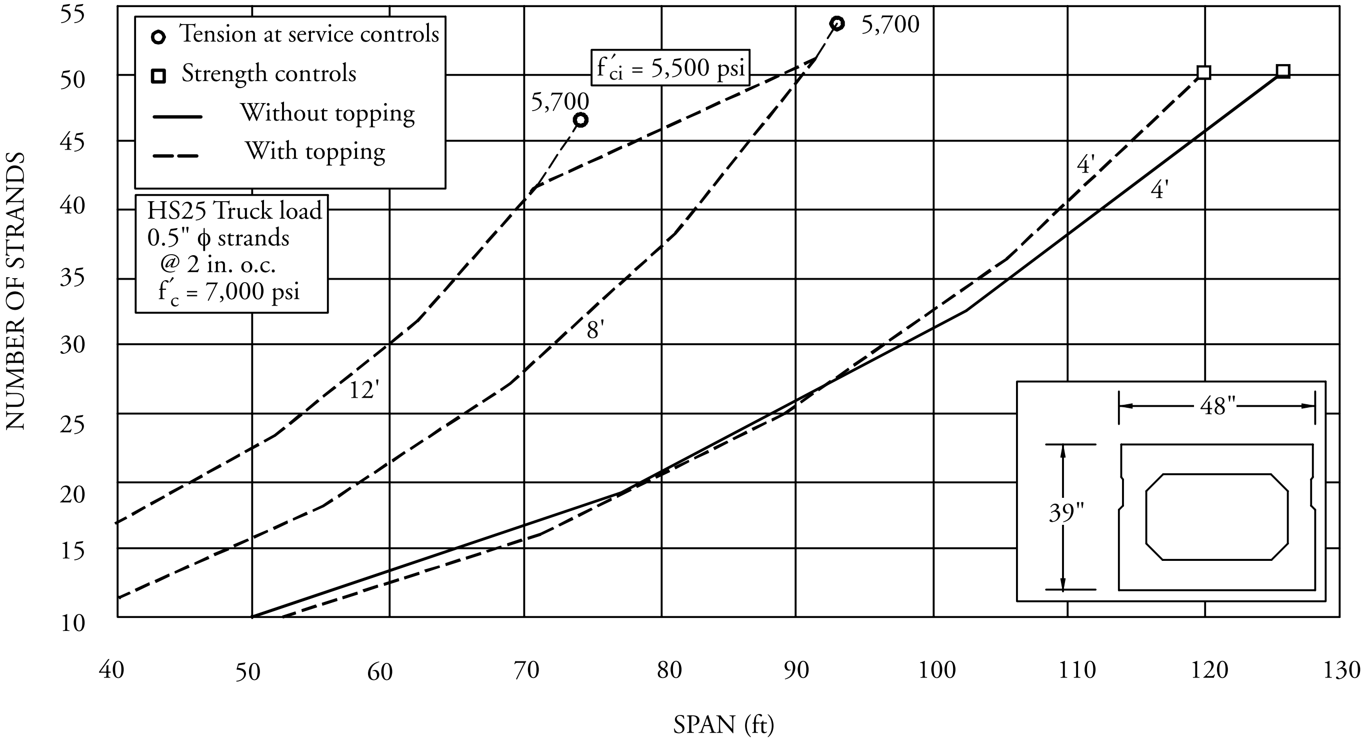

Chart BB-5
AASHTO Box Beams-BIV-48

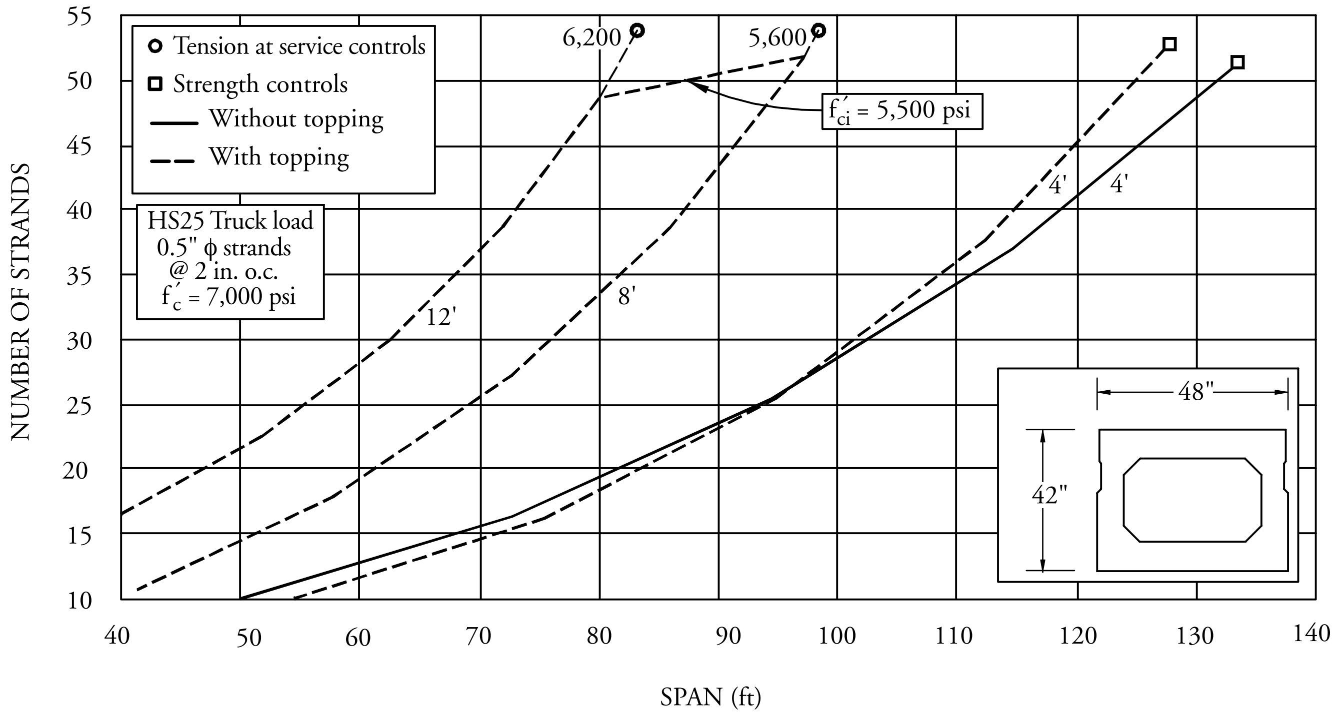

Chart BB-6
AASHTO Box Beams-
36 in. Wide

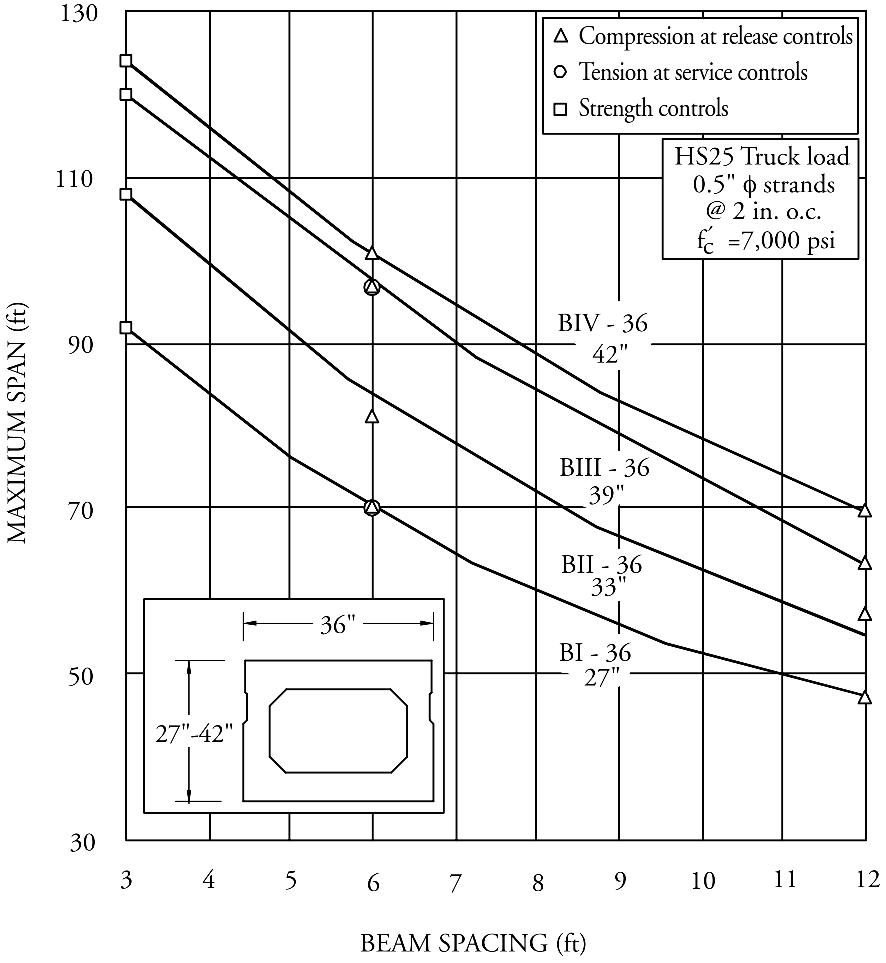

Chart BB-7
AASHTO Box Beams-BI-36

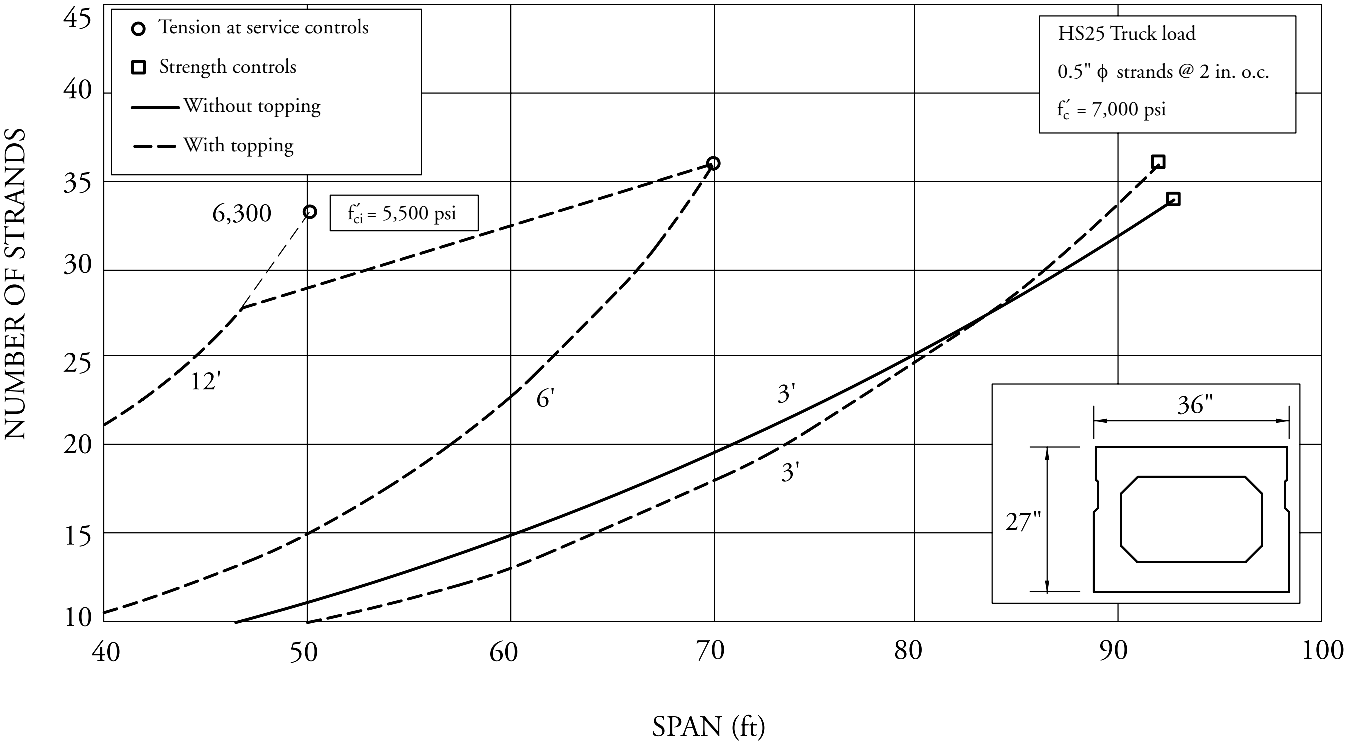

Chart BB-8
AASHTO Box Beams-BII-36

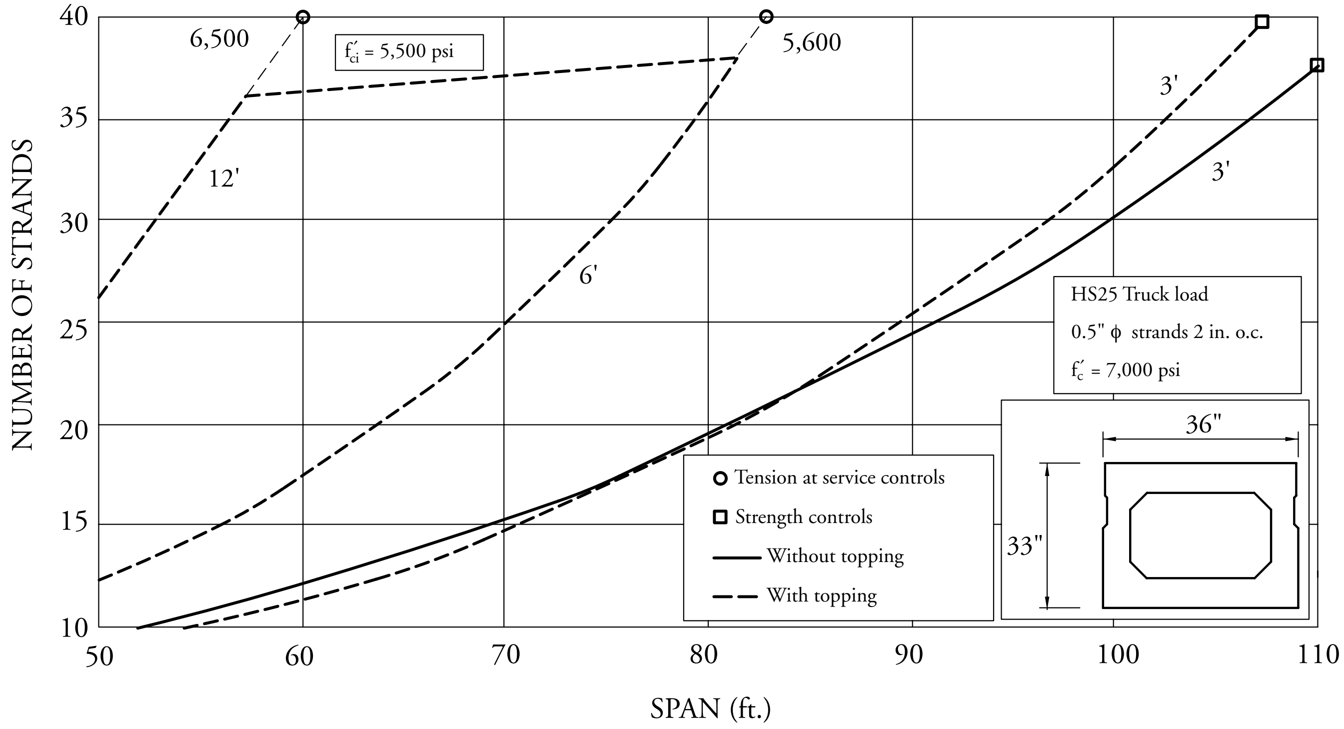

Chart BB-9
AASHTO Box Beams-BIII-36

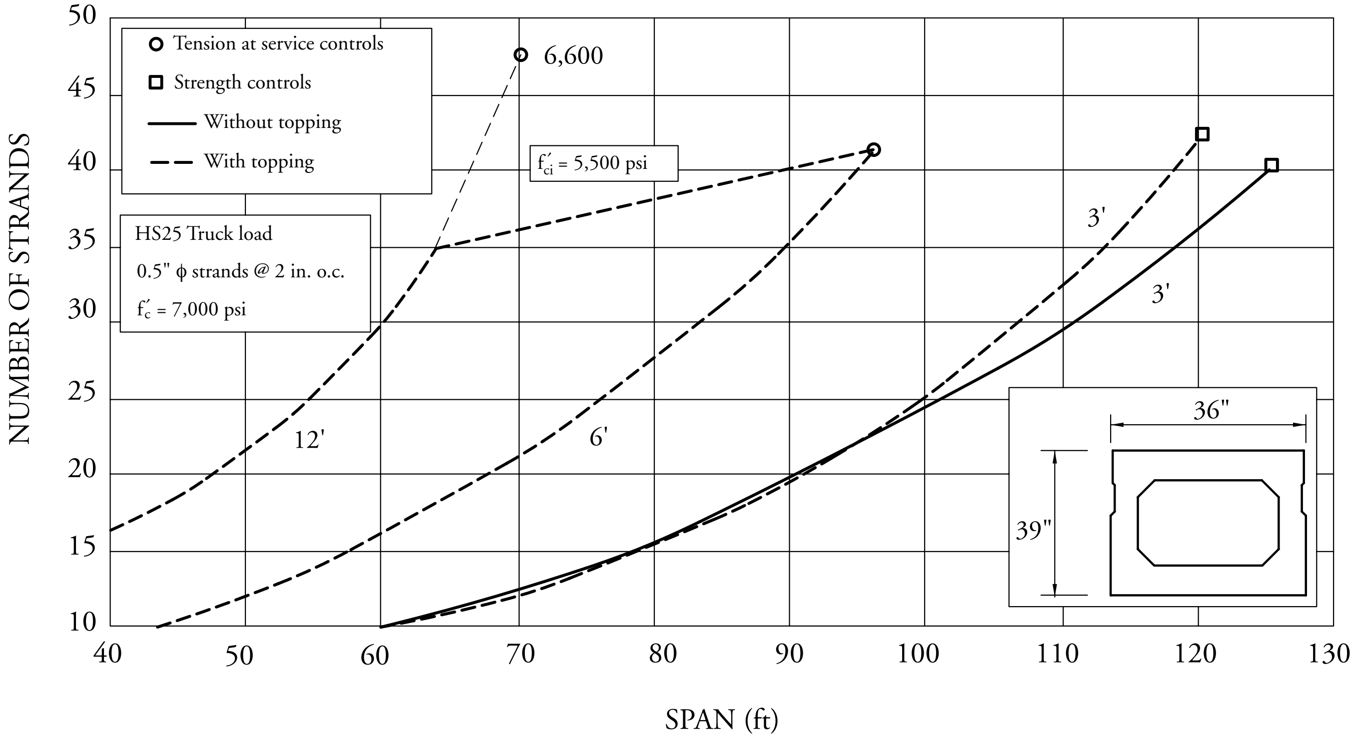

Chart BB-10
AASHTO Box Beams-BIV-36

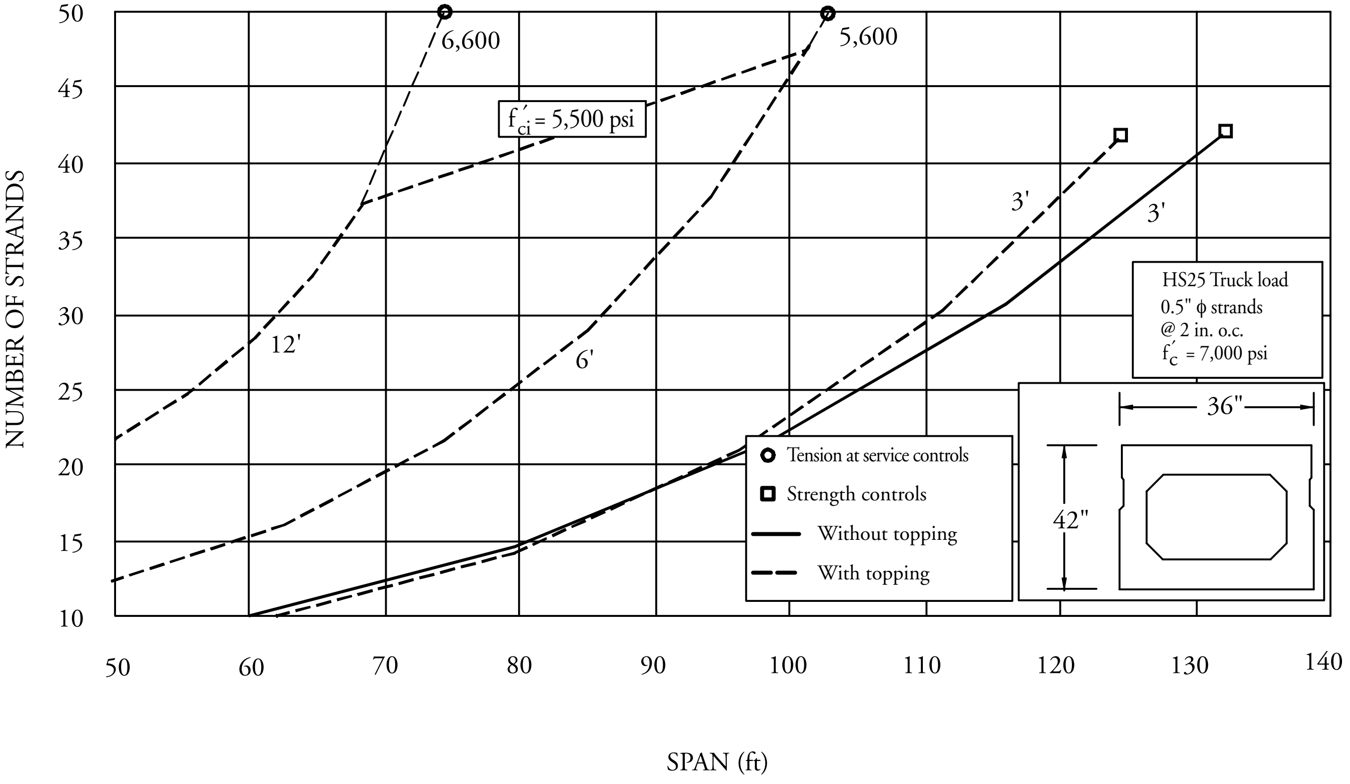

Chart BT-1
AASHTO-PCI Bulb-Tees

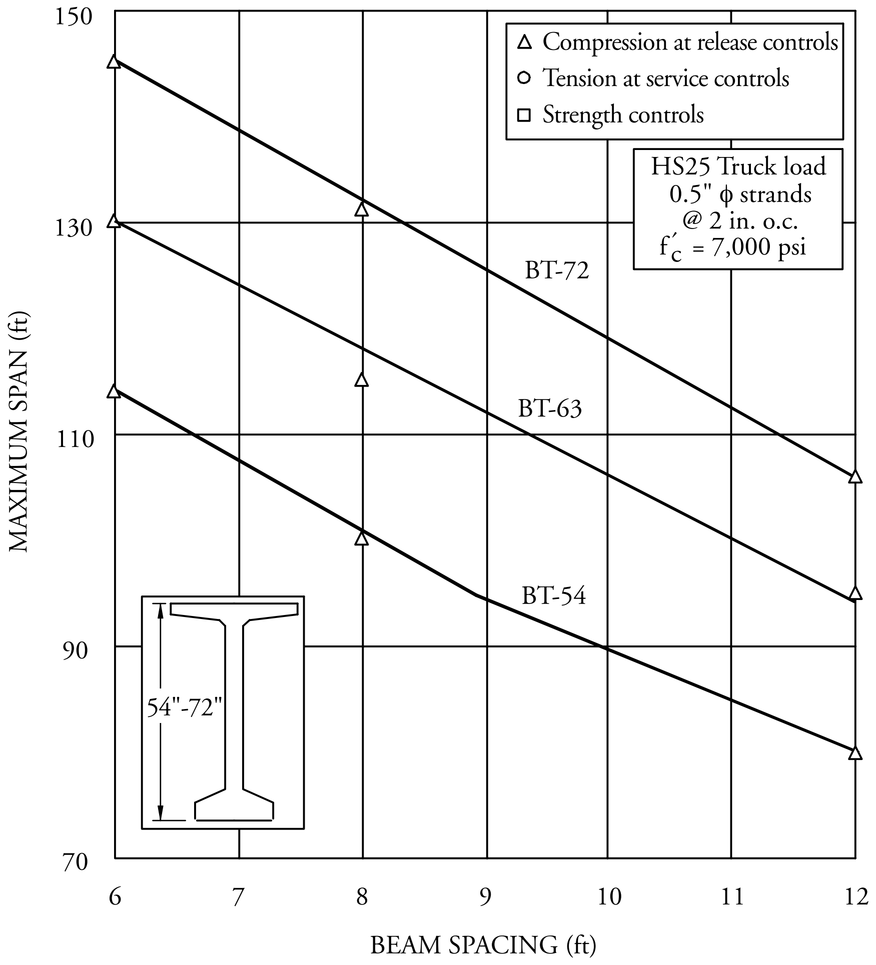

Chart BT-2
AASHTO-PCI Bulb-Tee – BT-54

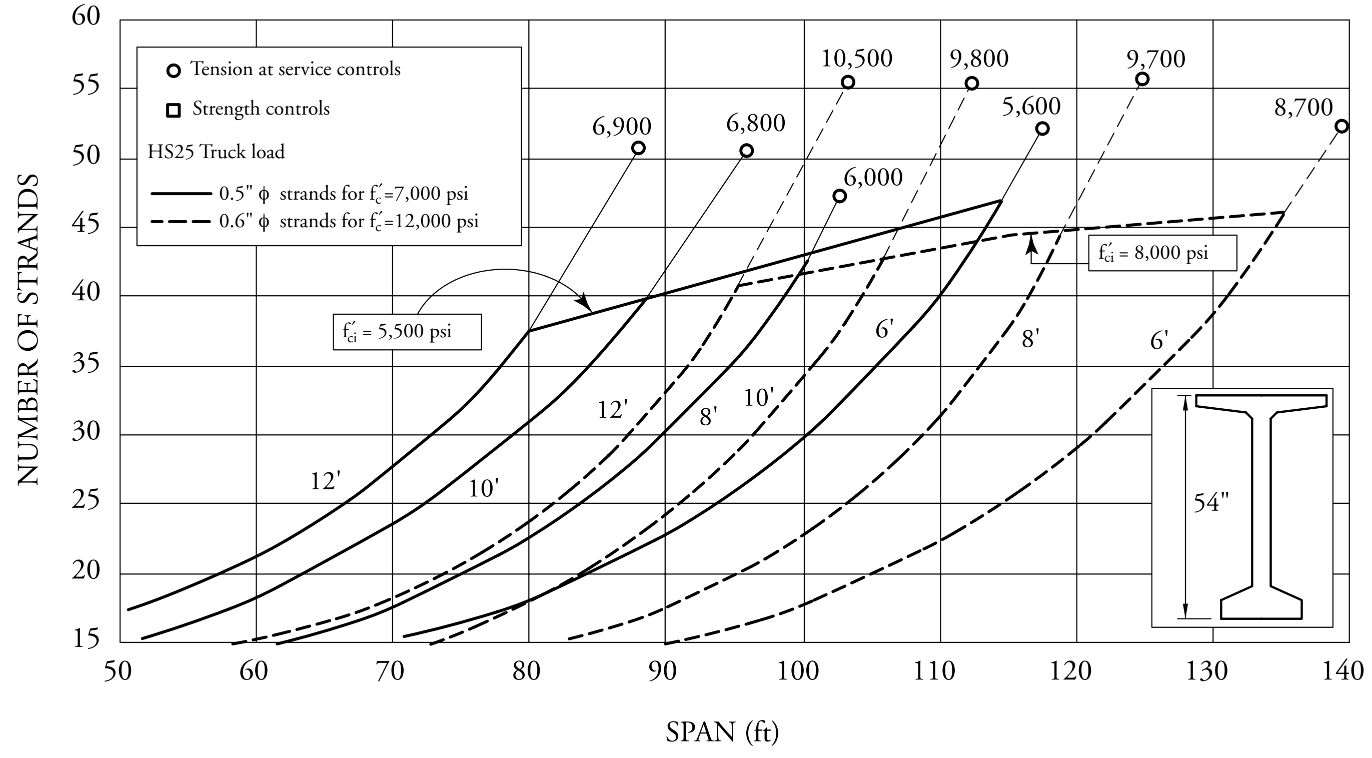

Chart BT-3
AASHTO-PCI Bulb-Tee – BT-63

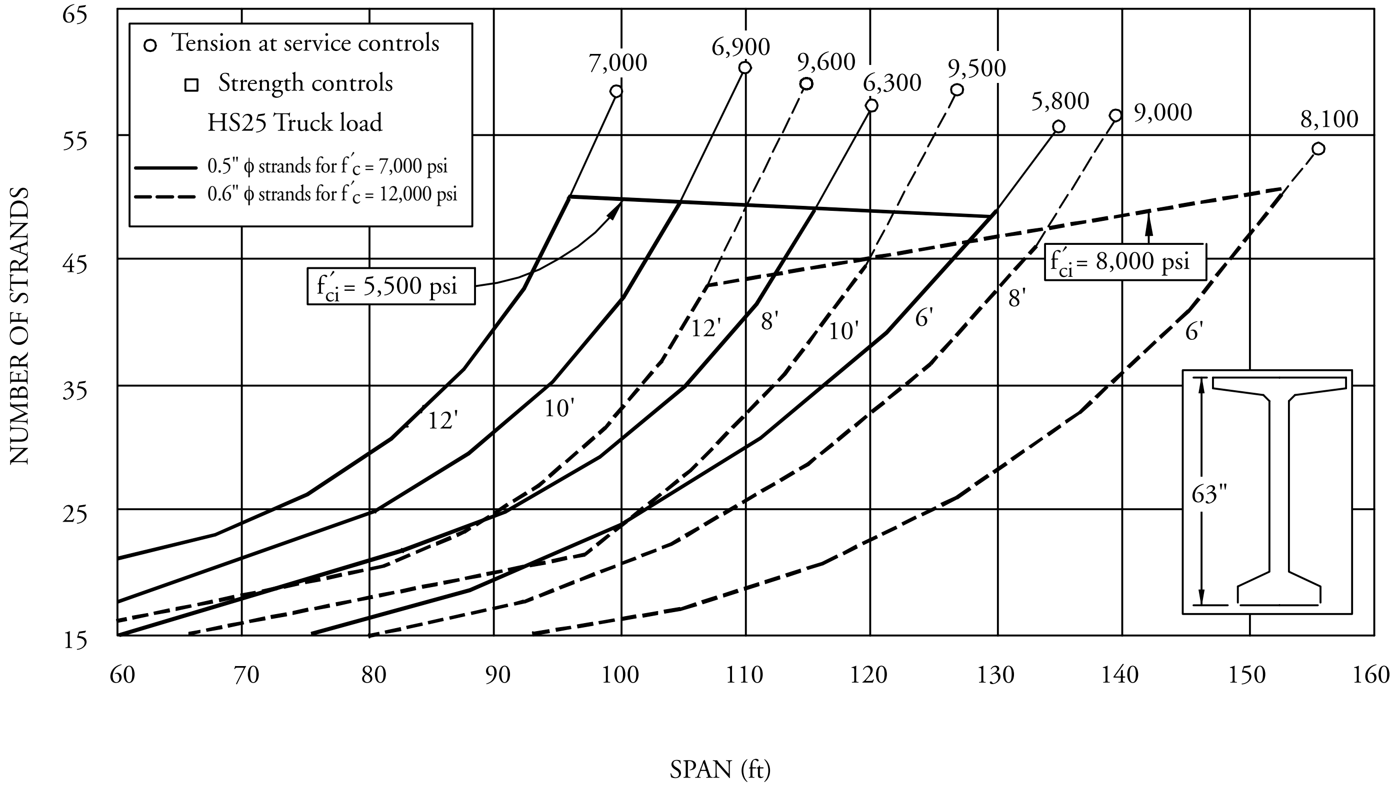

Chart BT-4
AASHTO-PCI Bulb-Tee – BT-72

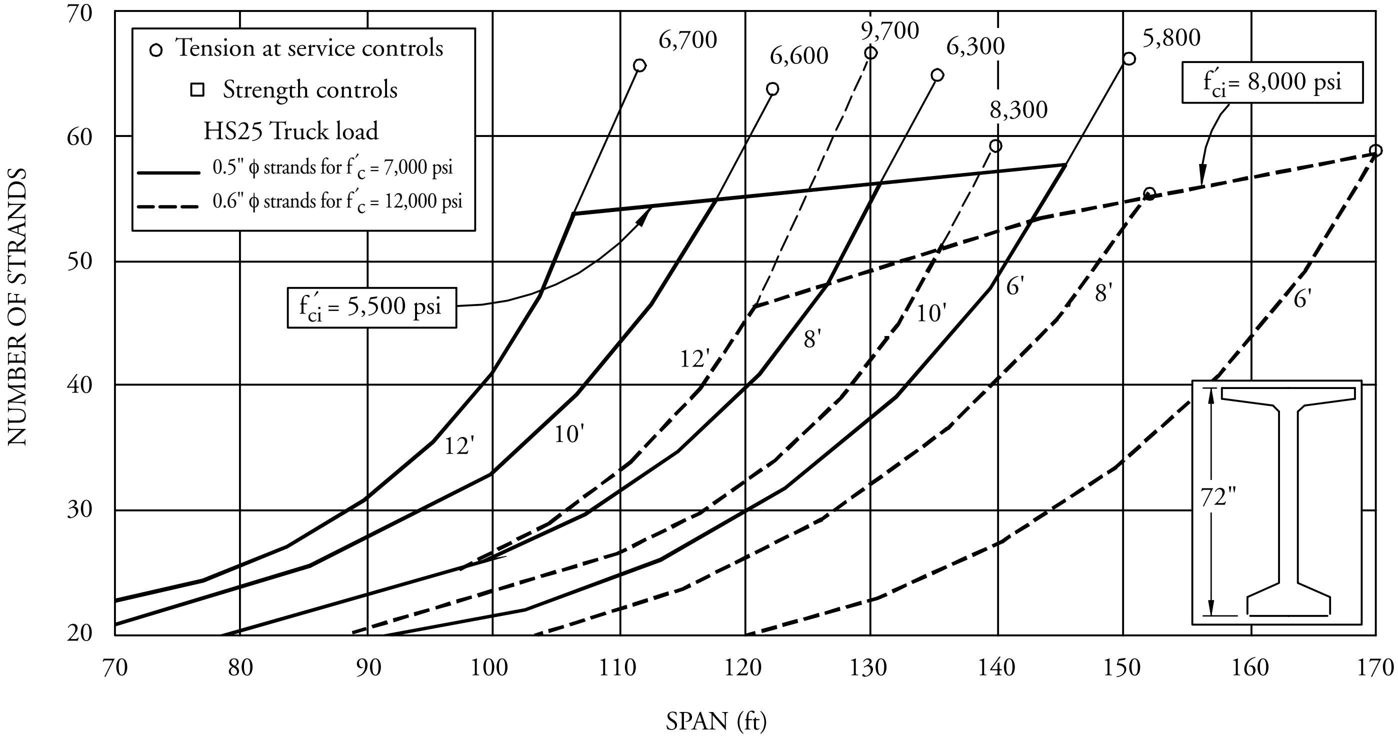

Chart DBT-1
Deck Bulb-Tees

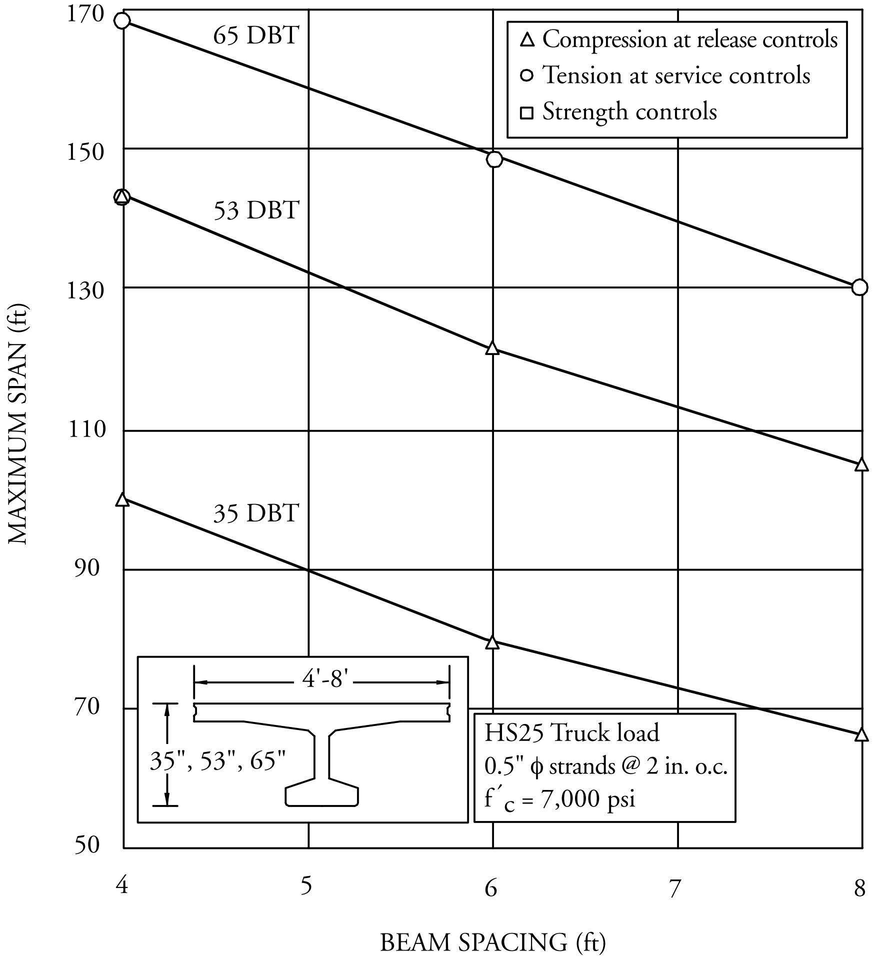

Chart DBT-2
35" Deck Bulb-Tees

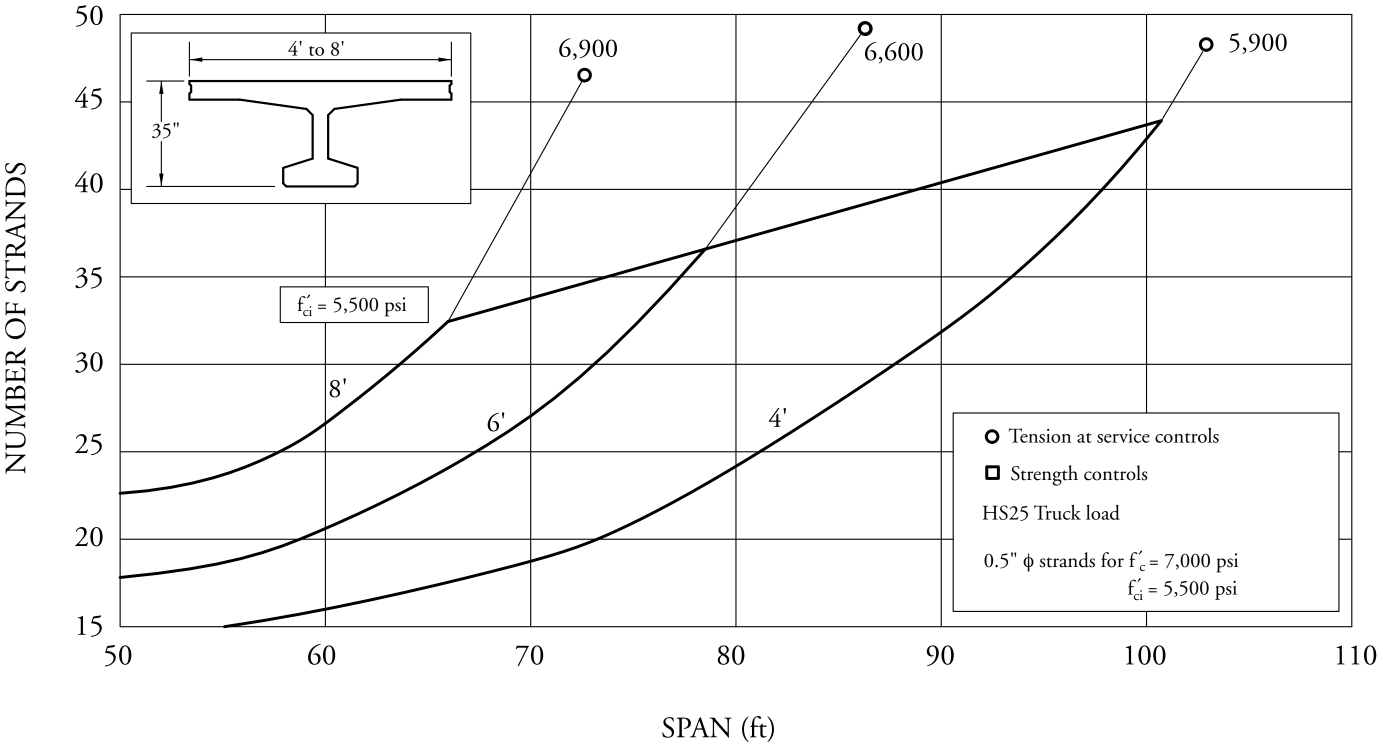

Chart DBT-3
53" Deck Bulb-Tees

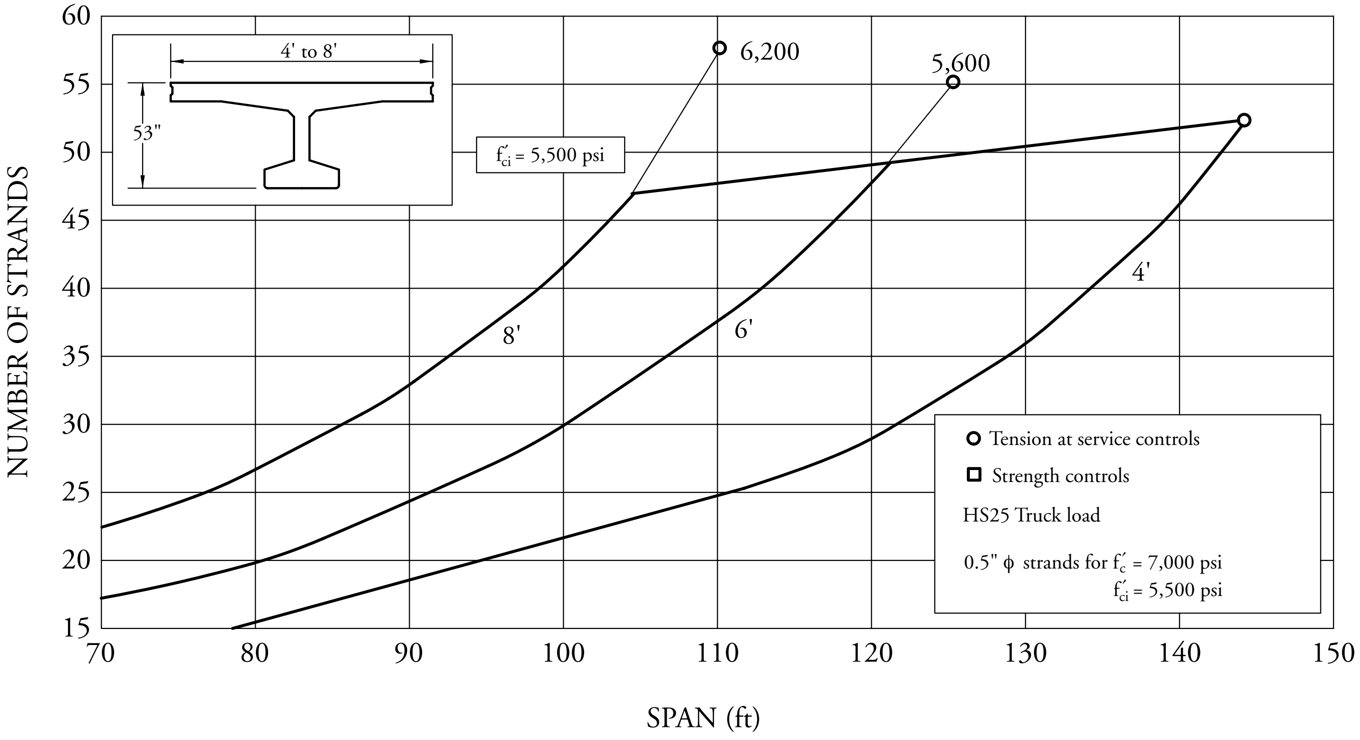

Chart DBT-4
65" Deck Bulb-Tees

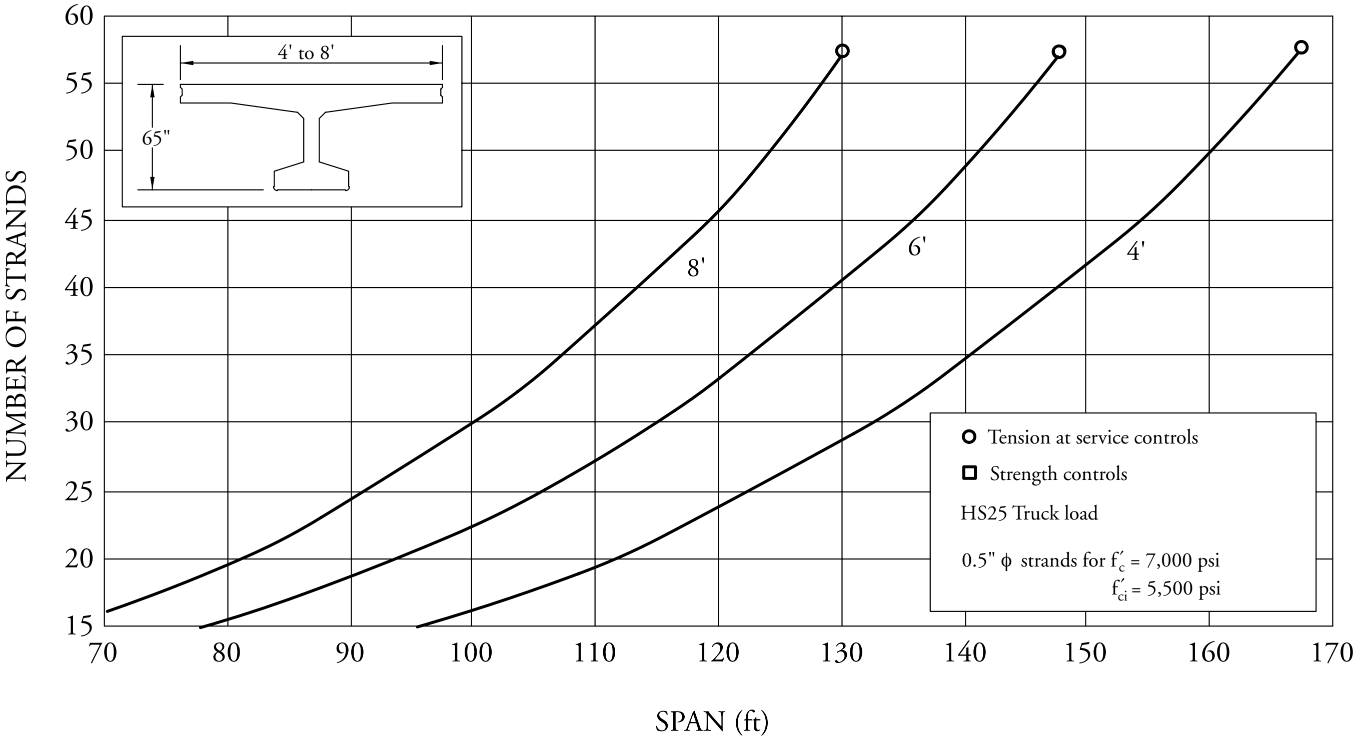

Chart DBT-5
Depth vs. Maximum Span

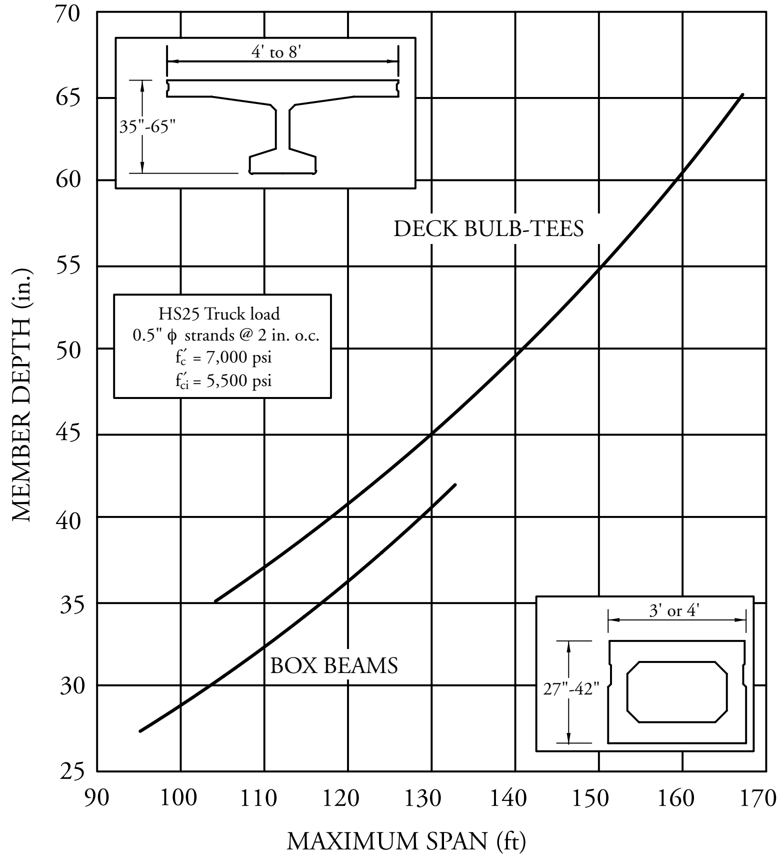

Chart IB-1
AASHTO I-Beams

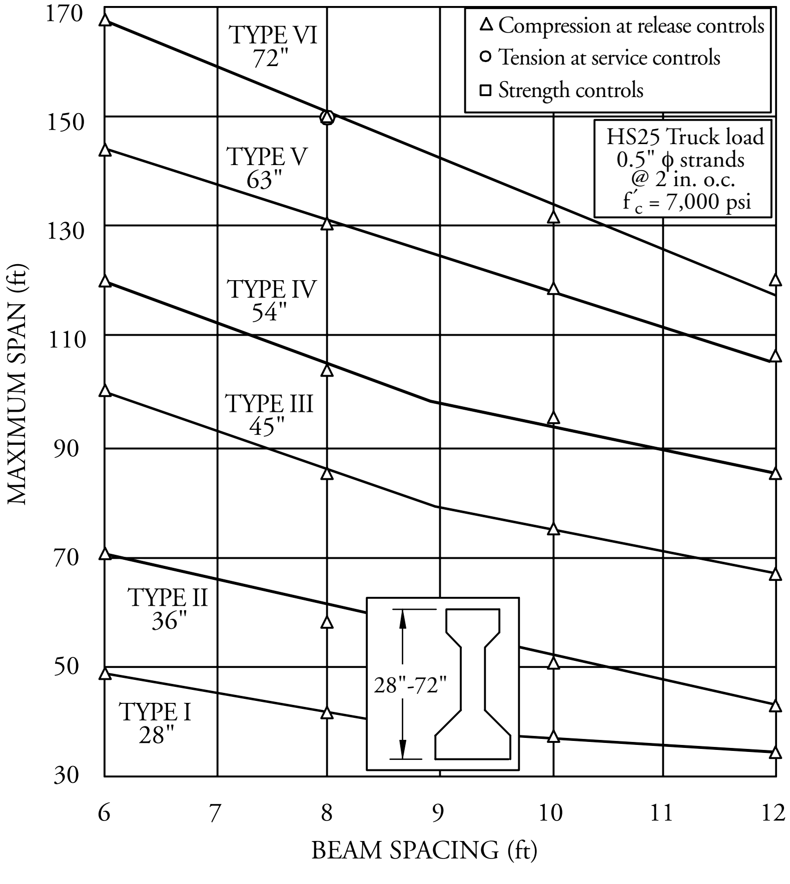

Chart IB-2
AASHTO I-Beams – Type I

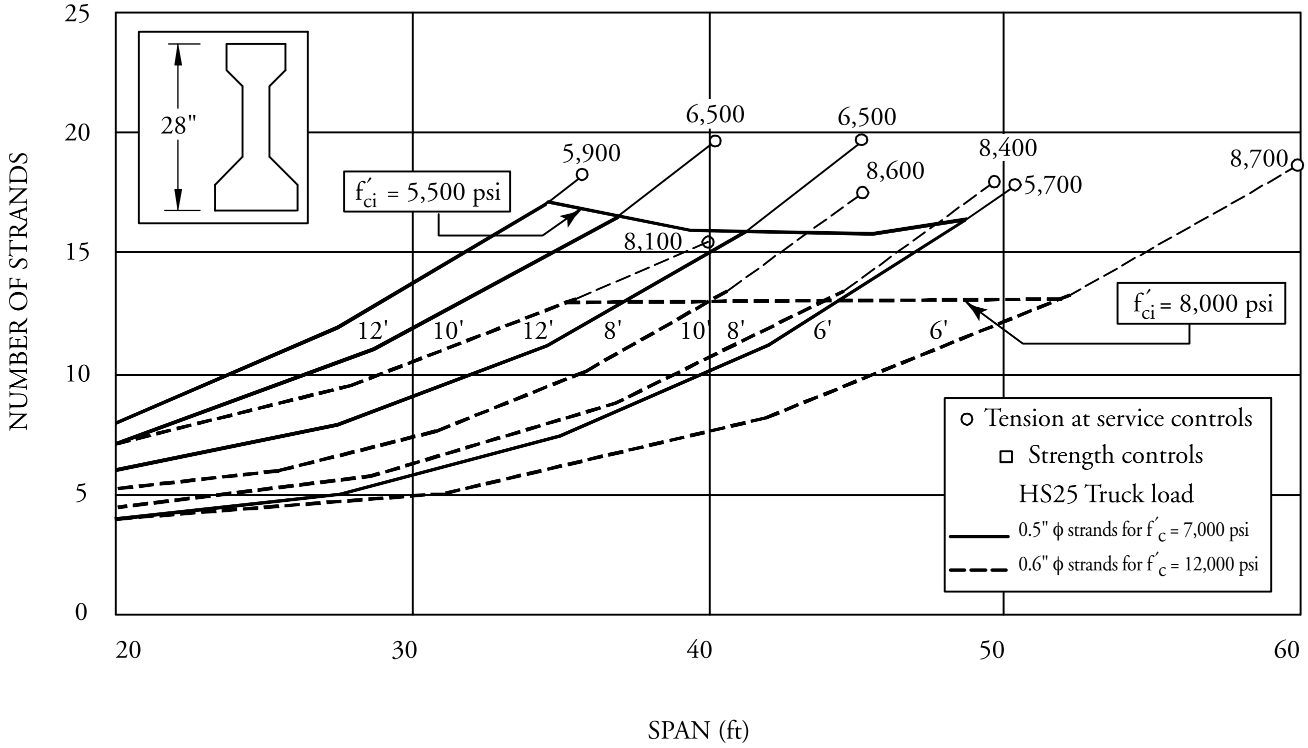

Chart IB-3
AASHTO I-Beams – Type II

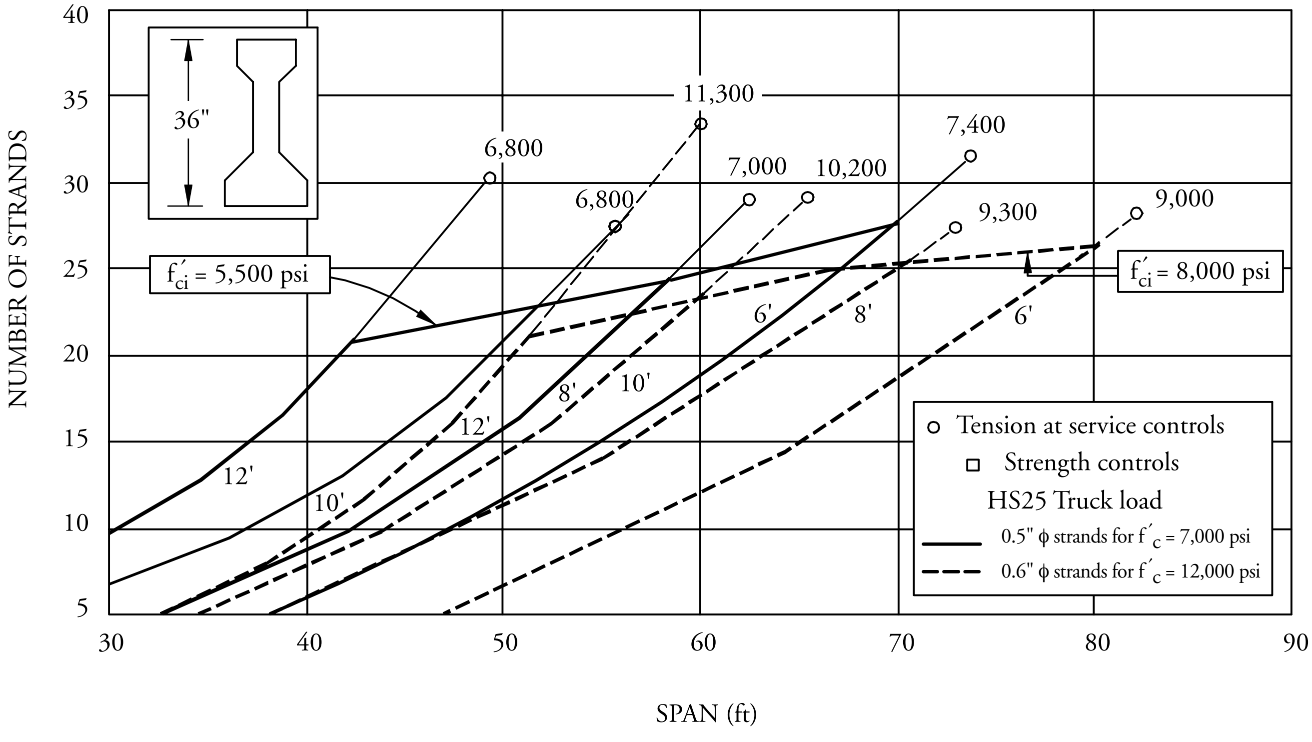

Chart IB-4
AASHTO I-Beams – Type III

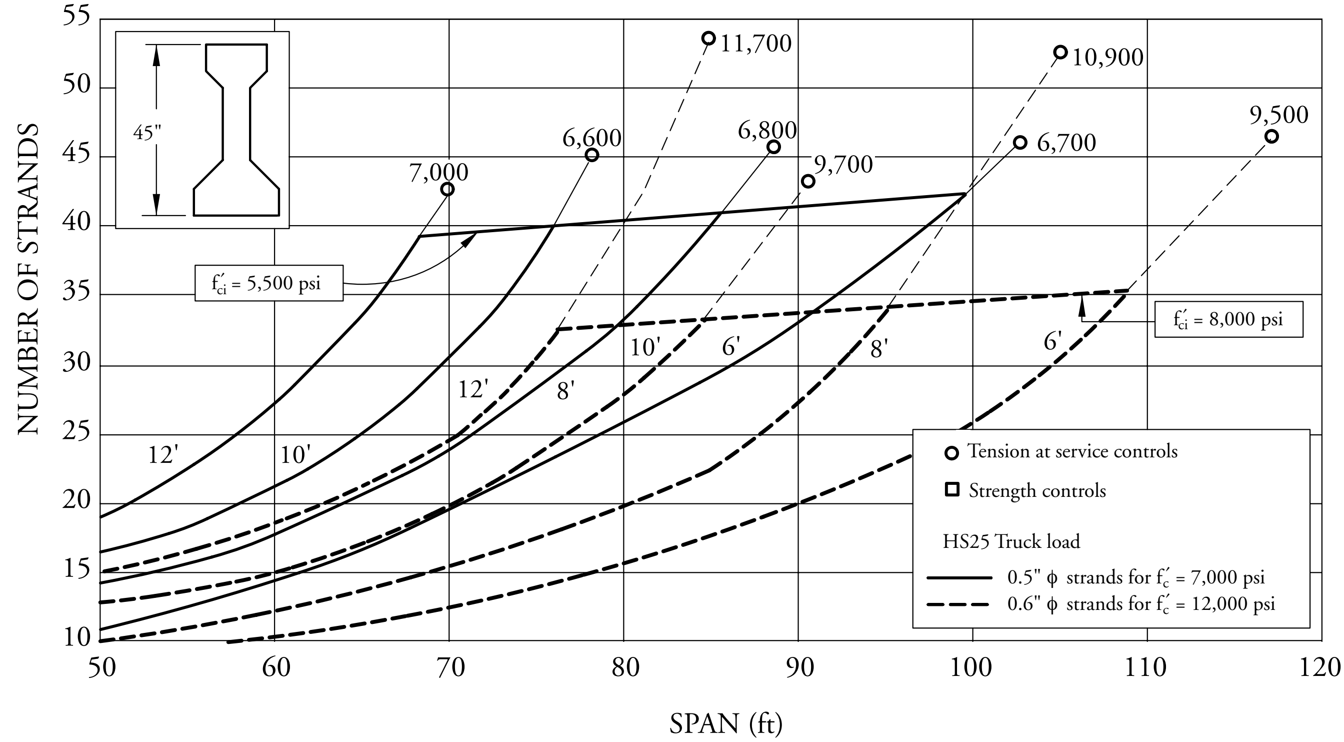

Chart IB-5
AASHTO I-Beams – Type IV

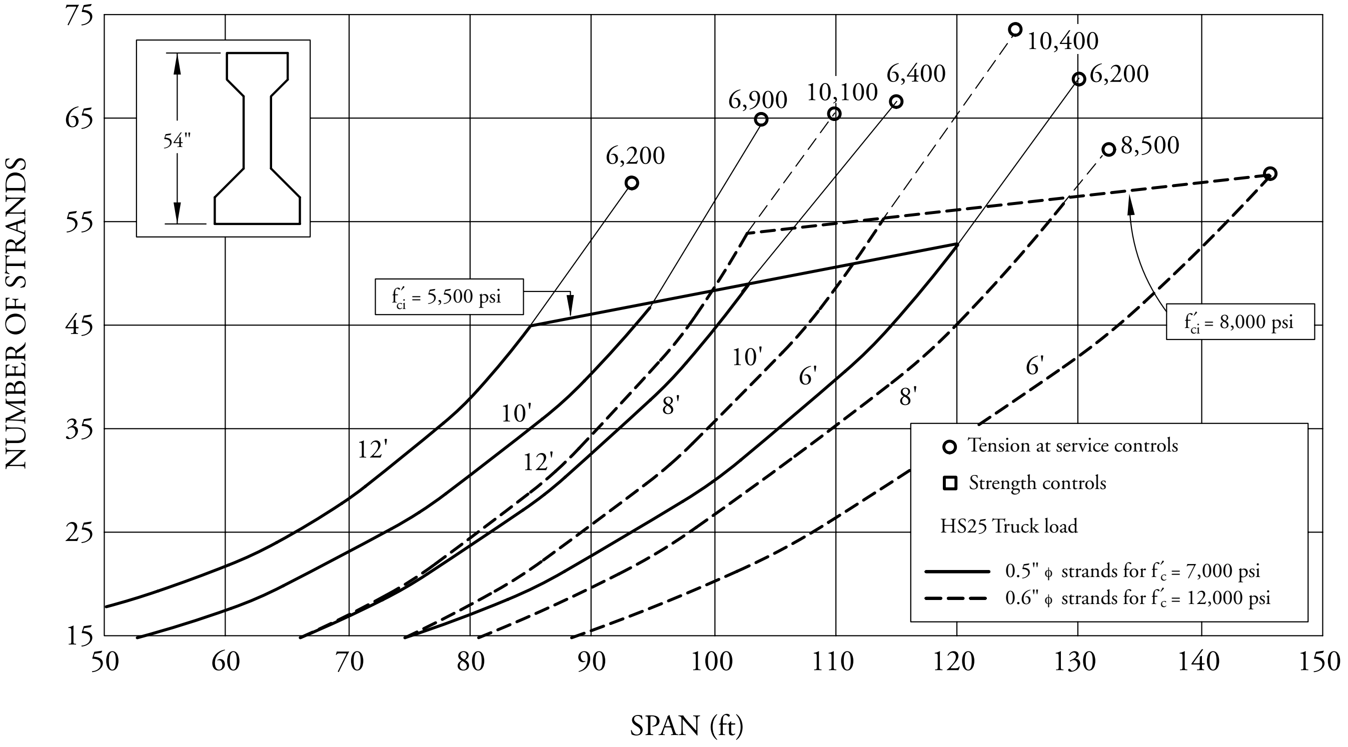

Chart IB-6
AASHTO I-Beams – Type V

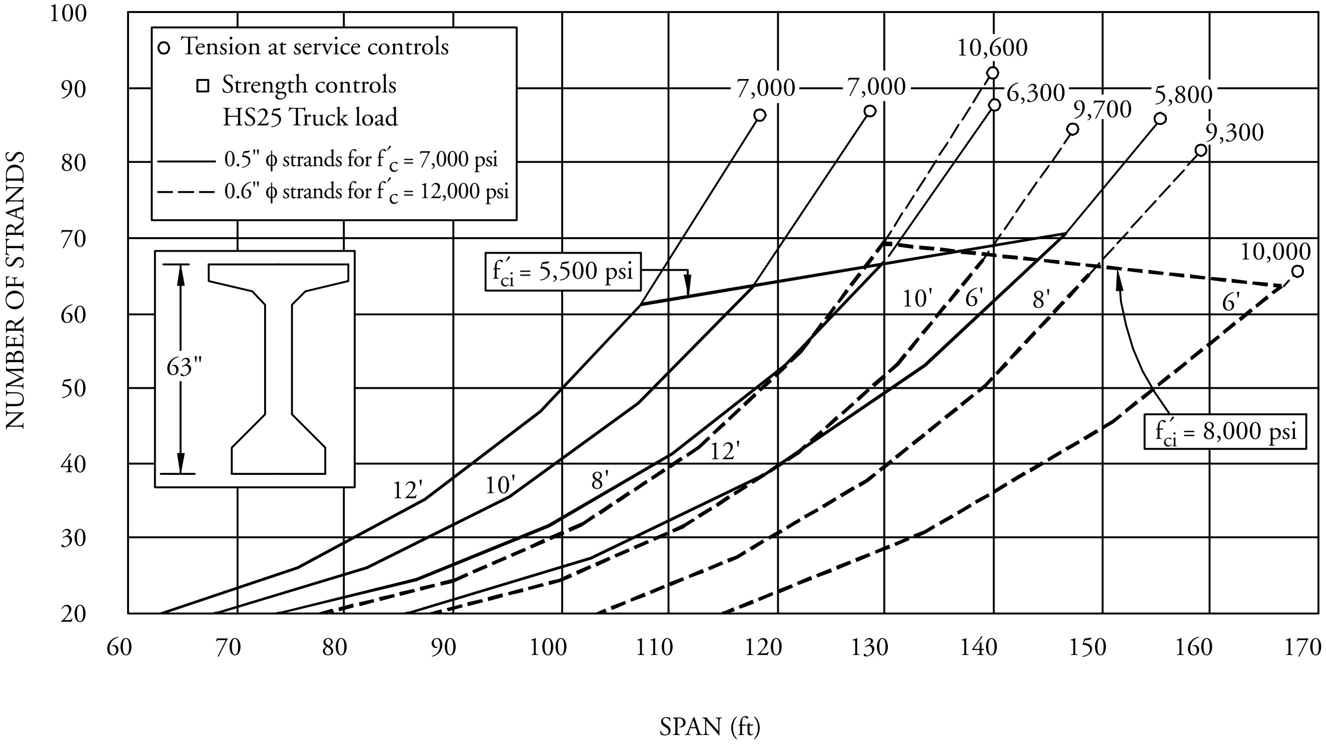

Chart IB-7
AASHTO I-Beams – Type VI

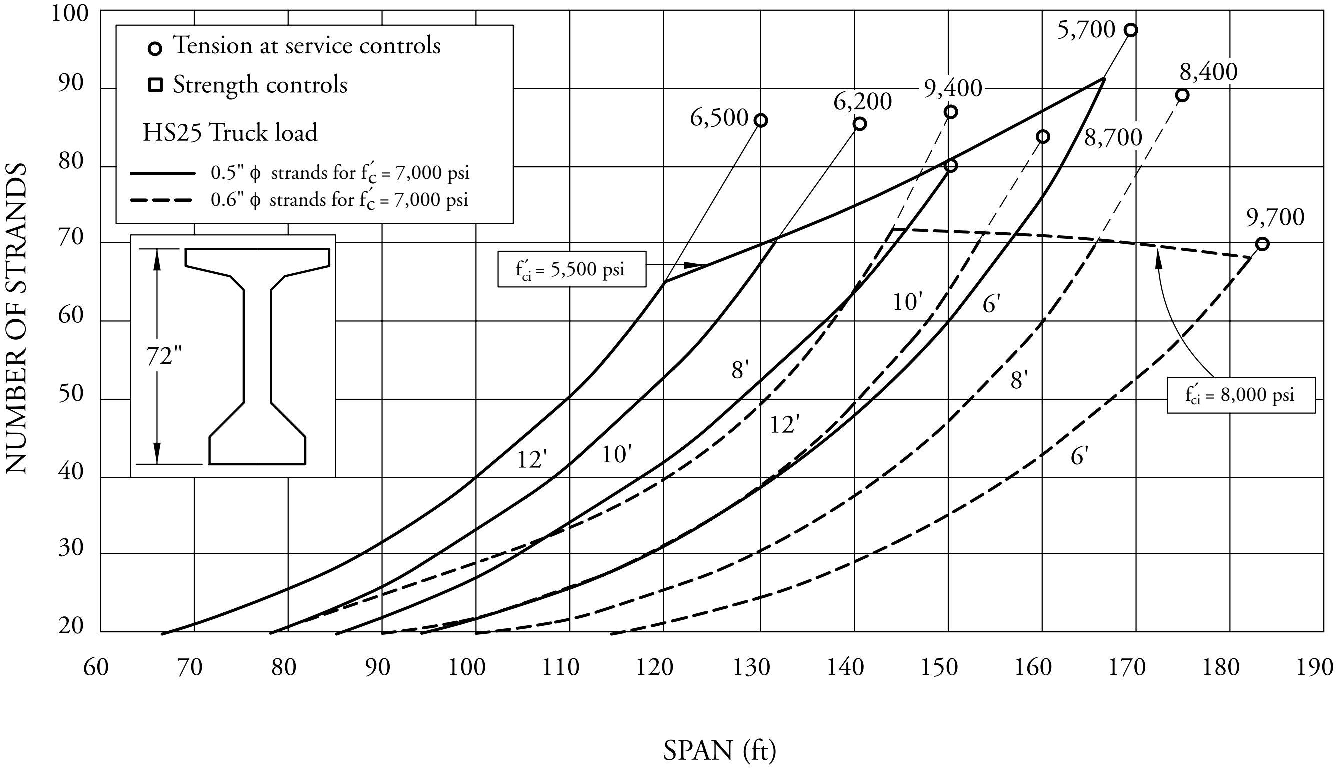

Chart DT-1
Untopped Double-Stemmed Beams

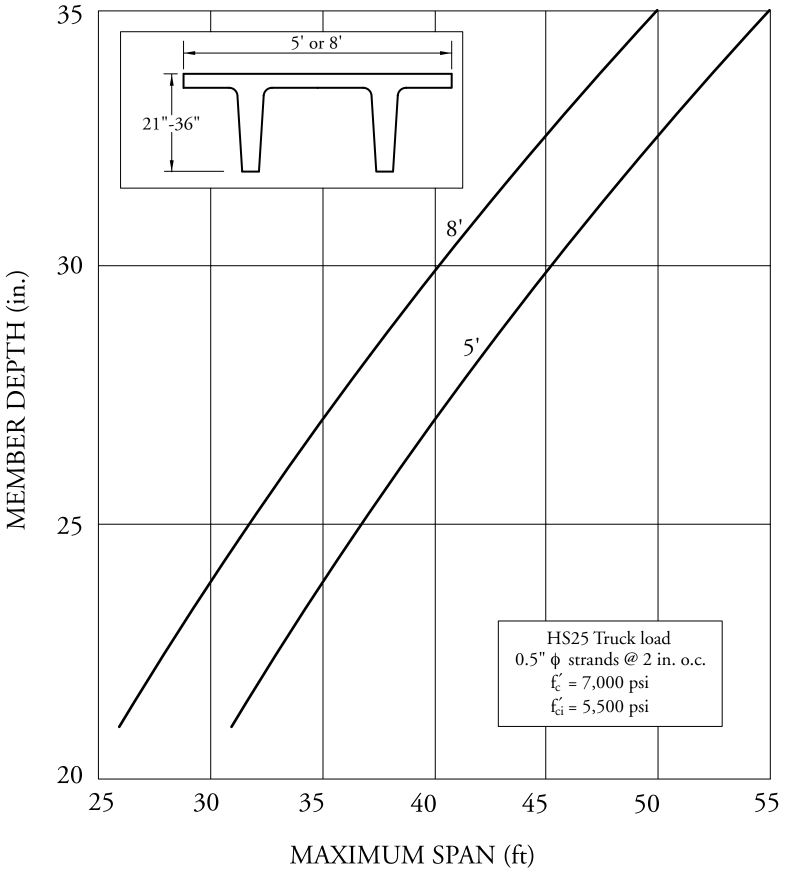

Chart DT-2
Topped Double-Stemmed Beams

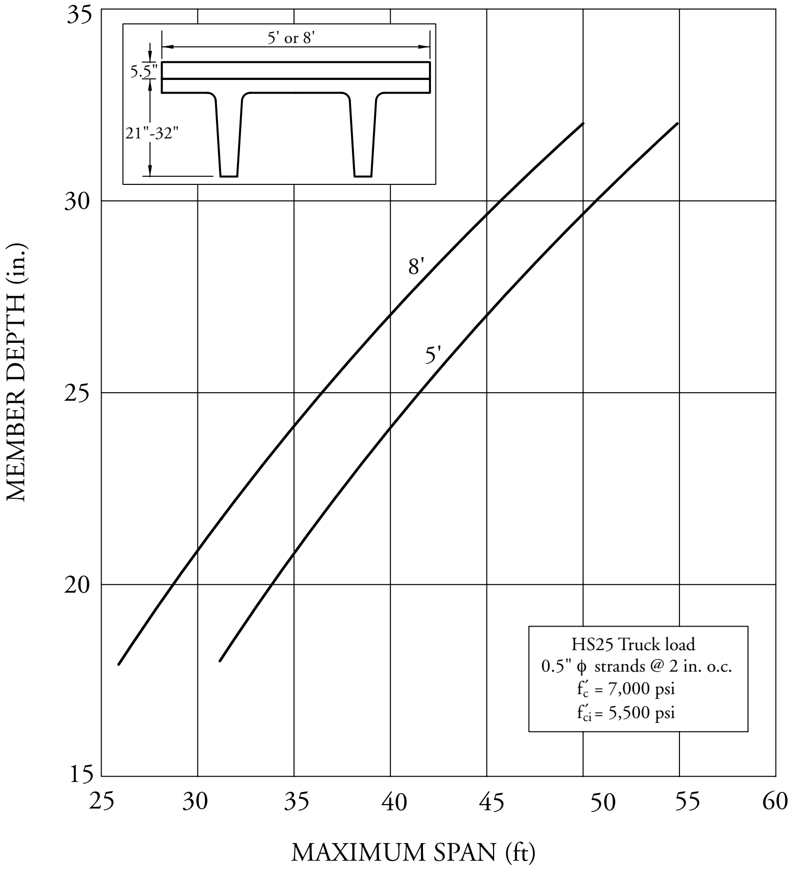

Chart SB-1
Voided Slab Beams

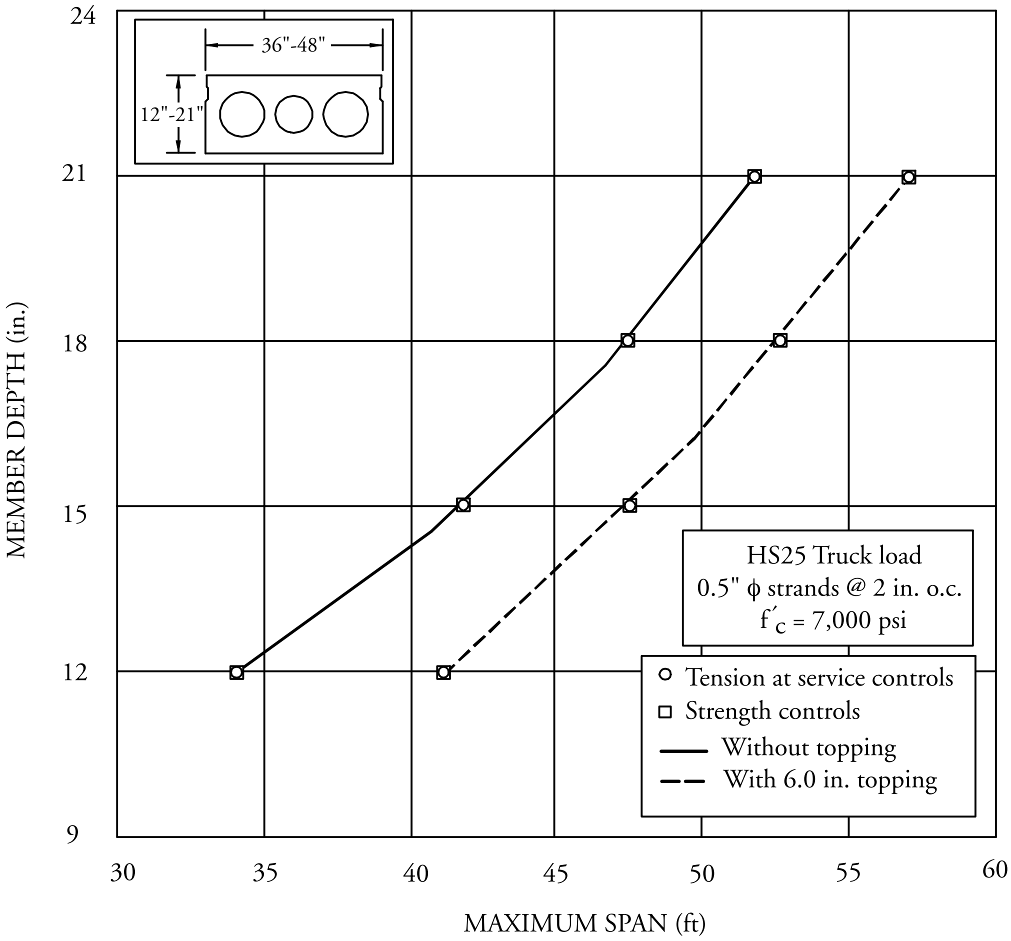

Chart SB-2
Voided Slab Beams – 48 in. Wide

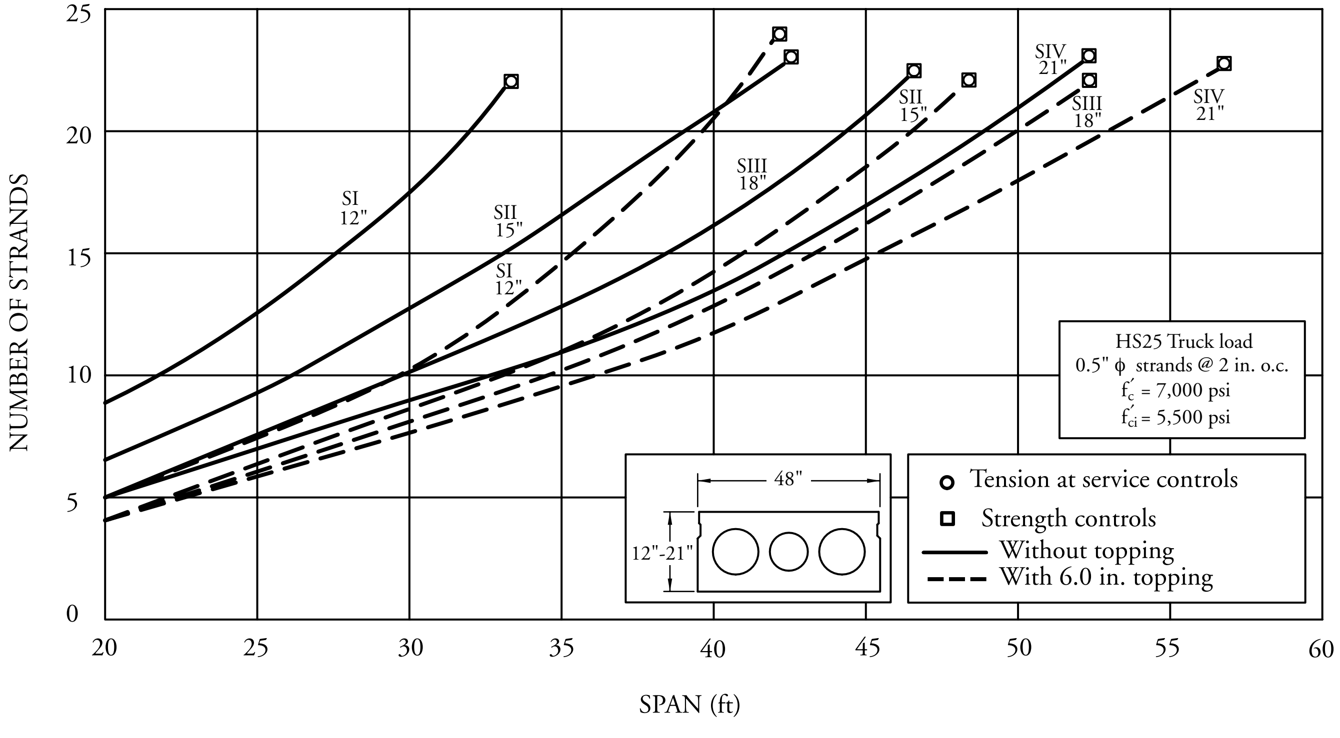

Chart SB-3
Voided Slab Beams – 36 in. Wide