PCI Precast/Prestressed Concrete Institute

BRIDGE DESIGN MANUAL

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PRELIMINARY DESIGN

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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 in 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 a minimum of two columns for roadways from 30 to 40 ft wide and three columns for roadways 40 to 60 ft wide. Also recommended is collision protection or design for collision loads in accordance with *LRFD Specifications* 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
- Drilled shafts
- Steel pipe piles
- Prestressed concrete piles
- Steel H-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, prestressed 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 preliminary design charts in **Section 6.9** are based on a blend of "national" and regional products. Data used to generate the design charts and basic information resulting from computer runs is provided in tables in **Section**

6.10. Traditional sections such as rectangular box beams, AASHTO I-beams and AASHTO-PCI Bulb-Tee sections are included because these are still commonly used for bridges with a wide range of configurations. Several other beam types are also included because they represent innovative design approaches and newer concepts gaining more widespread use. These include a non-composite deck bulb-tee family of shapes, various composite U-beams and a variation on traditional double-tee stemmed beams known as the NEXT beam.

The design charts are not an exhaustive summary of available products since many regional standards exist beyond those presented herein. There are dozens of additional beam types that have not been covered, yet are used successfully by individual states or regionally. States such as Washington, Utah, Texas, Nebraska, Florida, Pennsylvania, the New England states, and others have all produced many variations on traditional I-beams, wide-flange concrete beams, multi-web stemmed beams, solid and hollow plank sections, and others. Many of the states have design charts similar to those presented in this chapter indicating the span capability of local products. As with most design and construction decisions, knowledge of the local marketplace is important in determining the optimal configuration for a bridge.

6.5.2 Design Criteria

The design charts and graphs provided in this chapter were developed to satisfy flexure at the Strength I and Service III limit states according to the AASHTO *LRFD Specifications* Fifth Edition 2010, and the 2011 Interim Revisions. The following criteria were used to develop the various design data points used to make up the families of curves.

- Prestressed beam concrete design strength, f'_c up to 8 ksi and concrete strength at transfer of prestress f'_{ci} up to 6.8 ksi
- Allowable tension at transfer $= 0.24\sqrt{f'_{ci}}$ considering bonded auxiliary reinforcement is present to permit the use of the higher allowable stress
- Transformed section properties are used for all stress calculations
- The AASHTO LRFD Approximate Method is used for long-term prestress loss computations with an assumed relative humidity of 70%.
- Strands are 0.6-in.-diameter, Grade 270, low-relaxation type
- A standard single slope 42-in.-high barrier rail is assumed on each side of the bridge. The estimated weight of 0.500 kips/ft is shared equally by the exterior and first interior beams for all preliminary beam calculations.
- A 0.035 ksf future wearing surface allowance is included with the load effect distributed evenly to all beams.
- For bridges with a cast-in-place concrete deck, the concrete strength is 4.0 ksi. A minimum thickness of 8 in. is used with ½-in. deducted for long-term wear when determining structural properties. For larger beam spacings, an increased slab thickness is provided consistent with usual engineering practice. See Section 6.5.2.3.
- Shear design was checked for an assumed stirrup layout using the AASHTO LRFD general procedure.

Various trial designs were performed considering both an exterior and the first interior beam. For spread closed box, I-beam, and bulb-tee type cross sections, a standard overhang of 3.5 ft measured from the centerline of the exterior beam was used for all variations of the typical section. This is in the range of standard overhangs for closed box and I-beam bridges.

Beam spacings of 6, 8, 10, and 12 ft were chosen to represent a reasonable upper and lower bound of spacings in use today. Within that range of spacings, it is generally found that for the narrower beam spacings, the exterior beam governs—that is it requires more strands for a given span length than an interior beam or has a slightly shorter maximum span length. For wider beam spacings, the interior beam begins to control. This is a reflection of the LRFD live load distribution factor variations between exterior and interior beams.

Generally for the range of parameters studied, the controlling beam (interior or exterior) was found to require several more strands and only reduced the maximum possible span length on the order of 5 to10 ft. Therefore, it is not unnecessarily conservative to make all the beams of equal configuration. Due to the sensitivity of the exterior beam design to the weight of railing, method of distribution, actual overhang distance, and other assumptions that vary from state to state, the preliminary design charts presented herein are for a typical first interior beam. The engineer is cautioned to use these charts accordingly and also to check an exterior beam design for the specific bridge conditions to make sure that the governing member is identified.

For composite U-beams, the overhang measured from the centerline of the exterior beam was selected as 6 ft. With precast section widths of 6 to 8 ft for common U-beams, this results in a physical overhang beyond the exterior web on the order of 2 to 3 ft, a reasonable dimension. The spacing of U-beams was chosen to vary from 10 to 18 ft. The minimum spacing of 10 ft reflects a reasonable minimum spacing given that the precast section will be 6 to 8 ft wide typically at its top. This is a near practical minimum beam spacing. At the upper end, a beam spacing of 18 ft was selected. This is the upper end of the limit of the empirical AASHTO live load distribution factors and results in a clear deck span between boxes of about 10 to 12 ft, still a reasonable slab span for conventionally reinforced decks and easily accommodated by traditional deck forming systems including stay-in-place precast deck panels.

Two NEXT beam types were chosen for evaluation, Type D and Type F. The Type D section has a thick top flange (8 in.) that can serve directly as the structural slab for the bridge. The design considers that a 3-in.-thick asphalt wearing surface is used. The other beam type, Type F, has a 4-in.-thick top flange that primarily serves as a continuous stay-in-place form for a traditional 8-in.-thick composite cast-in-place deck with a future overlay allowance.

6.5.2.1 Live Loads

The live load considered for the charts is the HL-93 loading with all designs based on a single span bridge. A random check of selected designs for the Type 3, 3S2 and 3-3 rating loads indicated that the HL-93 designs governed the design and resulted in designs with inventory and operating rating factors greater than 1.0 for the various notional rating vehicles. Live load moment and shear are distributed to the beams in accordance with the AASHTO empirical equations for live load distribution found in LRFD Section 4.6.2.2 with the exception that the rigid rotation model for exterior beams is not considered. The rigid rotation model is only stipulated for bridges with diaphragms and cross frames that are sufficient to induce a load distribution mechanism analogous to the rigid body distribution usually assumed for elements like pile groups or footings. For a prestressed concrete I-beam or bulb-tee section such as cross-section (k) in LRFD Table 4.6.2.2.1-1, the designer should consider whether the exterior diaphragms required by the specifications or agency policy are sufficient in number and stiffness to produce such behavior. If so, the design charts may prove to be unconservative for exterior beams in some instances and the designer should be aware that three potential exterior beam distribution factors might apply—the simple beam, AASHTO empirical, and rigid rotation model.

Since various types of beams and cross sections have been studied, a unique approach to live load distribution is required for each solution. The following load distribution models from LRFD Table 4.6.2.2.1-1 were considered in the development of the design graphs.

- For AASHTO I-beam and bulb-tee sections, cross-section Type (k) was used.
- For spread box beams, cross-section Type (b) was used.
- For U-beams, cross-section Type (c) was used.
- For adjacent box beams with a cast-in-place concrete overlay, Type (f) was used. All adjacent box beams were assumed to have a composite, cast-in-place concrete slab. Charts for non-composite box beams with an asphalt overlay were not developed.
- For deck bulb-tee bridges without transverse post-tensioning in the flanges, cross-section Type (j) was used.
- For double-tee NEXT Type D and F beams, cross-section (k) was used to be consistent with the PCI Northeast Chapter assumptions in developing the section and details. (see **Appendix C**)

6.5.2.2 Dead Loads

The design of the first interior beam was performed assuming that the beam carries 50% of the weight of the barrier rail. A 42-in.-high single slope barrier rail was assumed, weighing approximately 0.500 kips/ft, with half of this load carried by the exterior beam and half by the first interior beam. The practice of distributing the parapet load to exterior and interior beams varies widely amongst engineers and agencies from even distribution to all beams to rules requiring a larger share of this load be carried by the exterior beam(s). For purposes of developing the design charts, it was assumed that the exterior beam carries 50% of the barrier rail and the first interior beam carries the remaining 50%. With heavy parapet loads, stiff beams, and relatively short overhangs, this approach is considered a reasonable approximation. Cast-in-place slab loads are assigned on a tributary basis. An allowance of 0.035 ksf is provided between gutter lines, uniformly carried by all beams, to provide for an additional wearing surface (*DW*) loading.

6.5.2.3 Composite Deck

For all spread beam designs (box, I-beam, U-beam, etc.), a composite deck section is used with the thickness as shown in Table 6.5.2.3-1.

Table 6.5.2.3-1
Assumed Deck Thickness

Beam Type	Beam Spacing ft	C.I.P Deck Thickness in.
Box Beams 48 in. wide	Adjacent	6.0
	6, 8, 10, 12	8.0
Box Beams 36 in. wide	Adjacent	6.0
	6, 8, 10	8.0
	12	8.5
Bulb-Tees BT-54, BT-63, BT-72	6, 8, 10	8.0
	12	9.0
Deck Bulb-Tees	Adjacent	None
I-Beams Types II, III, IV	6, 8	8.0
	10	8.5
	12	9.5
I-Beams Types V, VI	6, 8, 10	8.0
	12	9.0
NEXT Beams Type D	Adjacent	None
NEXT Beams Type F	Adjacent	8.0
U-Beams	10, 14	8.0
	18	10

See **Appendix C** for spliced U-Beams and curved spliced U-Beams from PCI Zone 6.

The deck comprises 4.0 ksi compressive strength concrete in all cases. A haunch thickness of 2 in. was typically used to provide additional dead load on the section as well as to slightly offset the deck from the top of the precast section. The use of the haunch to offset the composite slab is a practice that varies throughout the country. Some agencies consider the slab to sit on top of the precast section while still providing for a haunch load. Others use the minimum haunch as typical for the entire span length (approach taken herein). There are other approaches as well.

For all design cases, a ½ in. reduction in slab thickness is included for wear.

For adjacent sections that are considered to have a composite topping, the topping thickness is assumed equal to 6 in. for box beams and 8 in. for NEXT Type F beams. The topping weight is based on the indicated thickness. However, composite section properties were determined with the assumption that long-term wear and/or longitudinal profiling (deck grinding) reduces the thickness by ½ in.

6.5.2.4 Concrete Strength and Allowable Stresses

The precast concrete products are assumed to have $f'_{ci} = 6.8$ ksi and $f'_c = 8.0$ ksi , and the cast-in-place topping is assumed to have $f'_c = 4.0$ ksi. These material properties are in keeping with readily available concrete mixes around the country. Substantially higher precast concrete transfer strengths have been achieved and are available on a regional basis.

6.6 DESCRIPTION OF DESIGN CHARTS

6.6.1 Product Groups

The design charts in **Section 6.9** 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 bulb-tees
DBT-1 through DBT-2	Deck bulb-tees
IB-1 through IB-6	AASHTO I-beams
NEXT-1 and NEXT-6	NEXT Double-tee beams
U-1 through U-5	U-Beams

(Geometric properties for 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. The number of strands was strictly based on flexural stress or strength requirements. In some cases, e.g., shallow I-beams at wide spacing, shear capacity may require an unreasonable stirrup arrangement. 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

For each scenario, various potential controls were checked. In general, the maximum span was first established by satisfying the Strength I and Service III limit states. When strands could no longer be added to the section, or doing so did not increase span capacity, the practical maximum span was established. However this was usually a large number of strands for a particular beam section. Checks of stress at transfer were also performed. To mitigate the high stresses in the transfer region, the use of harping (with a hold down at $0.4L$) or debonding was used to control the beam end stresses. Maximum debonding limits of 40% of the strands in a row and 25% of the total number of strands were enforced with the exception that if the number of debonded strands was only one strand over the maximum due to rounding, that was considered an acceptable solution. The charts do not indicate the nature of the control but generally for narrower beam spacings the trend was for Service III to govern and for wider spacing, longer spans, Strength I was a common control. Most of the intermediate to longer spans required some debonding or harping to control the end zone stresses.

6.7 PRELIMINARY DESIGN EXAMPLES

6.7.1 Preliminary Design Example No. 1

Design a simple span for HL-93 loading with a 95 ft design span. The total width of the bridge is 36 ft 0 in. 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, possible solutions are summarized in **Table 6.7.1-1**.

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

<table>
<tr><th colspan="2">Product</th><th>Depth
in.</th><th>Spacing
in.</th><th>Topping
(Deck)</th><th>Number
of Strands</th><th>Design
Chart</th></tr>
<tr><td colspan="2" rowspan="3">Deck Bulb-Tees
6-ft-Wide Flange</td><td>41</td><td>72</td><td>No</td><td>26</td><td>DBT-2</td></tr>
<tr><td>53</td><td>72</td><td>No</td><td>20</td><td>DBT-2</td></tr>
<tr><td>65</td><td>72</td><td>No</td><td>18</td><td>DBT-2</td></tr>
<tr><td rowspan="6">AASHTO
Box
Beams</td><td>BII-36</td><td>33</td><td>36</td><td>Yes</td><td>22</td><td>BB-7</td></tr>
<tr><td>BIII-36</td><td>39</td><td>36</td><td>Yes</td><td>18</td><td>BB-7</td></tr>
<tr><td>BIV-36</td><td>42</td><td>36</td><td>Yes</td><td>16</td><td>BB-7</td></tr>
<tr><td>BII-48</td><td>33</td><td>48</td><td>Yes</td><td>27</td><td>BB-2</td></tr>
<tr><td>BIII-48</td><td>39</td><td>48</td><td>Yes</td><td>23</td><td>BB-2</td></tr>
<tr><td>BIV-48</td><td>42</td><td>48</td><td>Yes</td><td>19</td><td>BB-2</td></tr>
</table>

Note 1. Refer to **Section 6.5** for design assumptions.

From the table above, the deck bulb-tee generally requires more depth, but fewer beams and, therefore, fewer total strands. Please note that the product may not be available in all regions. 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 Examples 9.3, 9.4, and 9.5, Chapter 9, have similar spans and loading requirements. In those examples, AASHTO BIII-48 box beams and DBT-53s are used. Considering **Table 6.7.1-1**, it is clear that a shallower section could be used.

6.7.2 Preliminary Design Example No. 2

Design a simple span for HL-93 loading with 120 ft design span. The total width of the bridge is 51 ft 0 in. 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.

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

Products		Depth in.	Spacing ft	Deck Thickness in.	Number of Strands	Design Chart
AASHTO I-Beams	I V	54	8	8.0	42	IB-4
		54	6	8.0	36	IB-4
	V	63	12	9.0	46	IB-5
		63	10	8.0	48	IB-5
	VI	63	8	8.0	42	IB-5
		63	6	8.0	32	IB-5
		72	12	9.0	40	IB-6
		72	10	8.0	42	IB-6
		72	8	8.0	36	IB-6
		72	6	8.0	26	IB-6
AASHTO-PCI Bulb-Tees	BT-54	54	6	8.0	34	BT-2
	BT-63	63	6	8.0	28	BT-3
	BT-72	72	6	8.0	24	BT-4
		72	8	8.0	34	BT-4
		72	10	8.0	38	BT-4
		72	12	9.0	36	BT-4
Deck Bulb-Tees 6-ft-Wide Flange		53	6	None	30	DBT-2
		65	6	None	23	DBT-2
AASHTO Box Beams	BIV-36	39	3	6.0	27	BB-7
	BIV-48	42	4	6.0	31	BB-5
Washington U-Beams	U66G5	66	10	8.0	47	U-4
	U78G5	78	14	8.0	49	U-5
		78	10	8.0	43	U-5

Note 1. Refer to **Section 6.5** for design assumptions.

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 12 ft spacing are recommended. However, since the bulb-tee is a lighter section and the number of strands required (36 strands) is less, a BT-72 at 12 ft spacing is a more efficient solution.

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 6.5 ksi and HL-93 loading conditions. Referring to the above table, the BT-72 was chosen with 9 ft spacing.

6.8 REFERENCES

1. AASHTO. 2010. *AASHTO LRFD Bridge Design Specifications*, Fifth Edition with 2011 Interim Revisions. American Association of State Highway and Transportation Officials, First Edition, Washington, DC. https://bookstore.transportation.org (Fee)

2. Hawkins, N. M. and D. A., Kuchma, 2007. *Application of LRFD Bridge Design Specifications to High-Strength Concrete: Shear Provisions.* NCHRP Report 579. Transportation Research Board. Washington, DC. 197 pp. http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_579.pdf

3. Rizkalla, S., A. Mirmiran, P. Zia, et al. 2007. *Application of the LRFD Bridge Design Specifications to High-Strength Structural Concrete: Flexure and Compression Provisions*. NCHRP Report 595. Transportation Research Board. Washington, DC. 28 pp. http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_595.pdf

4. Ramirez, J. A. and B. W. Russell. 2008. *Transfer, Development, and Splice Length for Strand/Reinforcement in High-Strength Concrete*. NCHRP Report 603. Transportation Research Board. Washington, DC. 122 pp. http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_603.pdf

6.9 PRELIMINARY DESIGN CHARTS

The design charts listed in Table 6.9-1 are included in this section, Section 6.10 provides tables that correspond to each of these charts that show input and output data from which the charts were developed.

Table 6.9-1
Design Charts

Chart No.	Beam Type	Chart Type
BB-1	AASHTO Box Beams 48 in. Wide	Maximum span versus beam spacing
BB-2	AASHTO Adjacent Box Beams 48 in. Wide	No. of strands versus span length
BB-3	AASHTO Spread Box Beams BII-48	No. of strands versus span length
BB-4	AASHTO Spread Box Beams BIII-48	No. of strands versus span length
BB-5	AASHTO Spread Box Beams BIV-48	No. of strands versus span length
BB-6	AASHTO Box Beams 36 in. Wide	Maximum span versus beam spacing
BB-7	AASHTO Adjacent Box Beams 36 in. Wide	No. of strands versus span length
BB-8	AASHTO Spread Box Beams BII-36	No. of strands versus span length
BB-9	AASHTO Spread Box Beams BIII-36	No. of strands versus span length
BB-10	AASHTO Spread Box Beams BIV-36	No. of strands versus span length
BT-1	AASHTO-PCI Bulb-Tees	Maximum span versus beam spacing
BT-2	AASHTO-PCI Bulb-Tees BT-54	No. of strands versus span length
BT-3	AASHTO-PCI Bulb-Tees BT-63	No. of strands versus span length
BT-4	AASHTO-PCI Bulb-Tees BT-72	No. of strands versus span length
DBT-1	Deck Bulb-Tees	Maximum span versus section depth
DBT-2	Deck Bulb-Tees	No. of strands versus span length
IB-1	AASHTO I-Beams	Maximum span versus beam spacing
IB-2	AASHTO I-Beams Type II	No. of strands versus span length
IB-3	AASHTO I-Beams Type III	No. of strands versus span length
IB-4	AASHTO I-Beams Type IV	No. of strands versus span length
IB-5	AASHTO I-Beams Type V	No. of strands versus span length
IB-6	AASHTO I-Beams Type VI	No. of strands versus span length
NEXT-1	NEXT Type D Beams	Maximum span versus section depth
NEXT-2	NEXT Type D x 96 Beams	No. of strands versus span length
NEXT-3	NEXT Type D x 120 Beams	No. of strands versus span length
NEXT-4	NEXT Type F Beams	Maximum span versus section depth
NEXT-5	Next Type F x 96 Beams	No. of strands versus span length
NEXT-6	Next Type F x 144 Beams	No. of strands versus span length
U-1	U-Beams	Maximum span versus beam spacing
U-2	Texas U-40 Beams	No. of strands versus span length
U-3	Texas U-54 Beams	No. of strands versus span length
U-4	Washington U66G5 Beams	No. of strands versus span length
U-5	Washington U78G5 Beams	No. of strands versus span length

Chart BB-1
AASHTO Box Beams 48 in. Wide

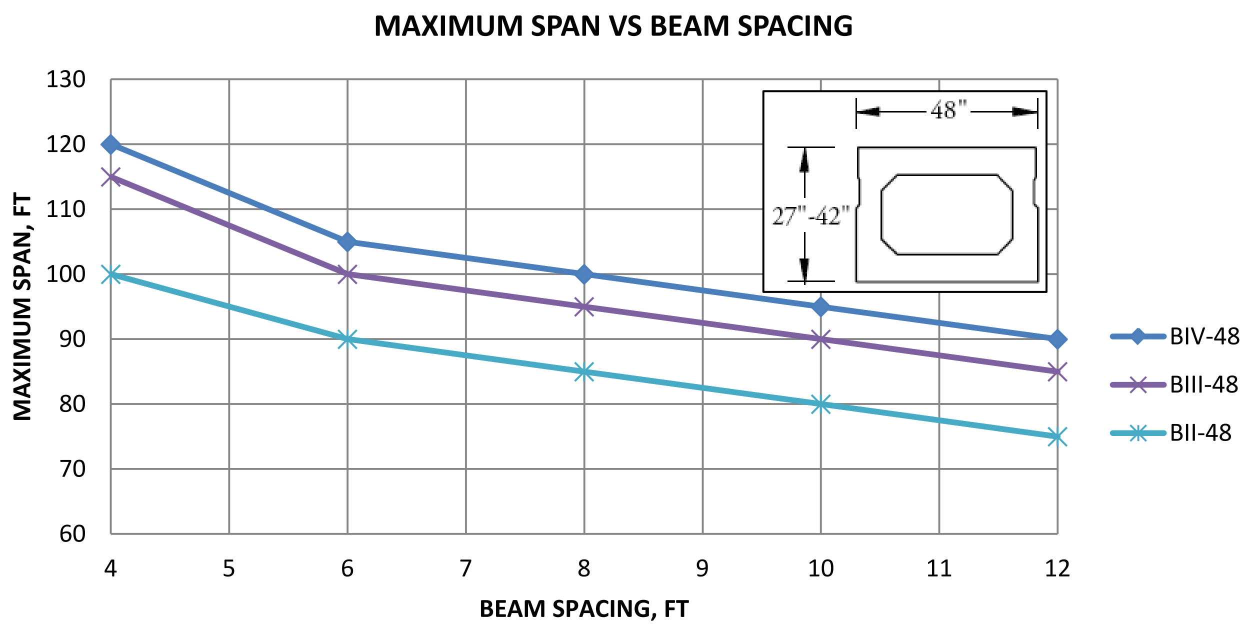


Chart BB-2
AASHTO Adjacent Box Beams 48 in. Wide

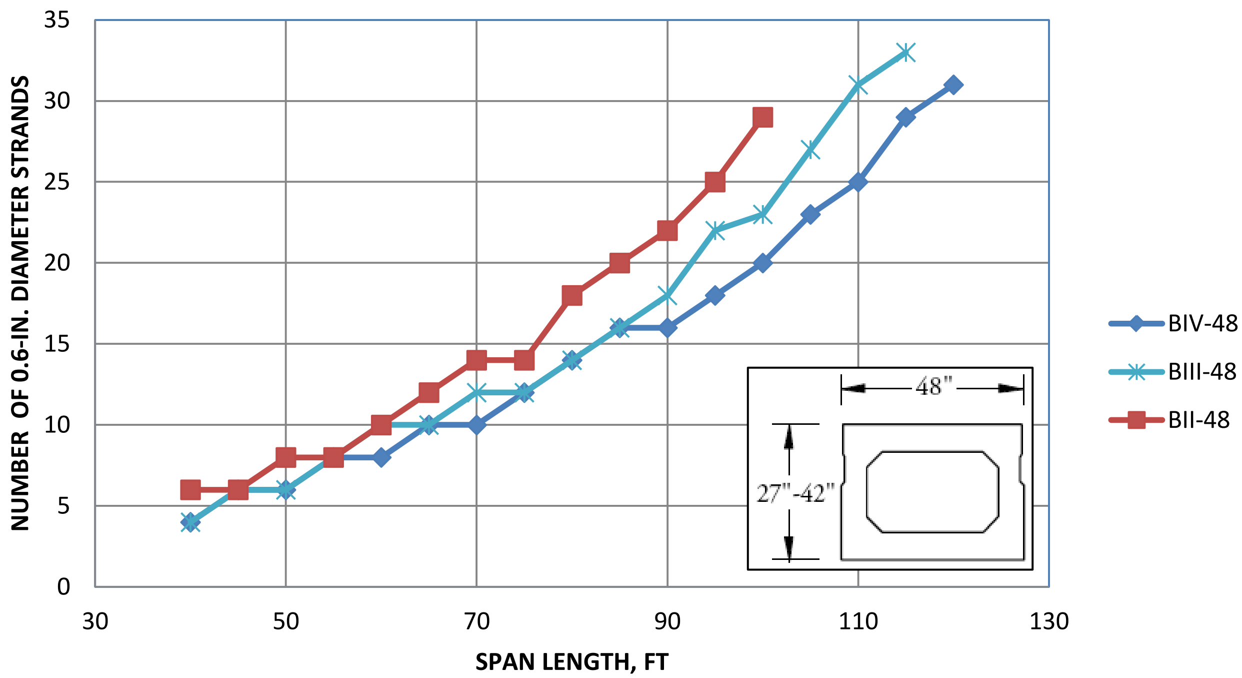


Chart BB-3
AASHTO Spread Box Beams BII-48

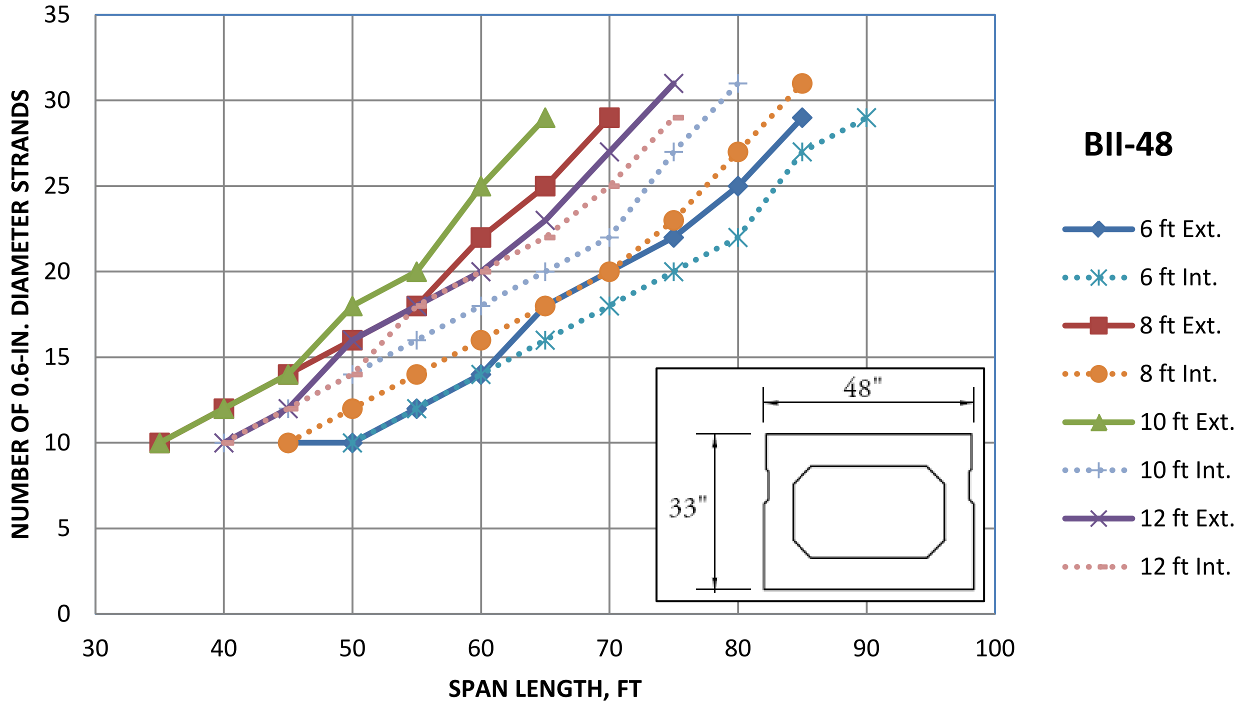


Chart BB-4
AASHTO Spread Box Beams BIII-48

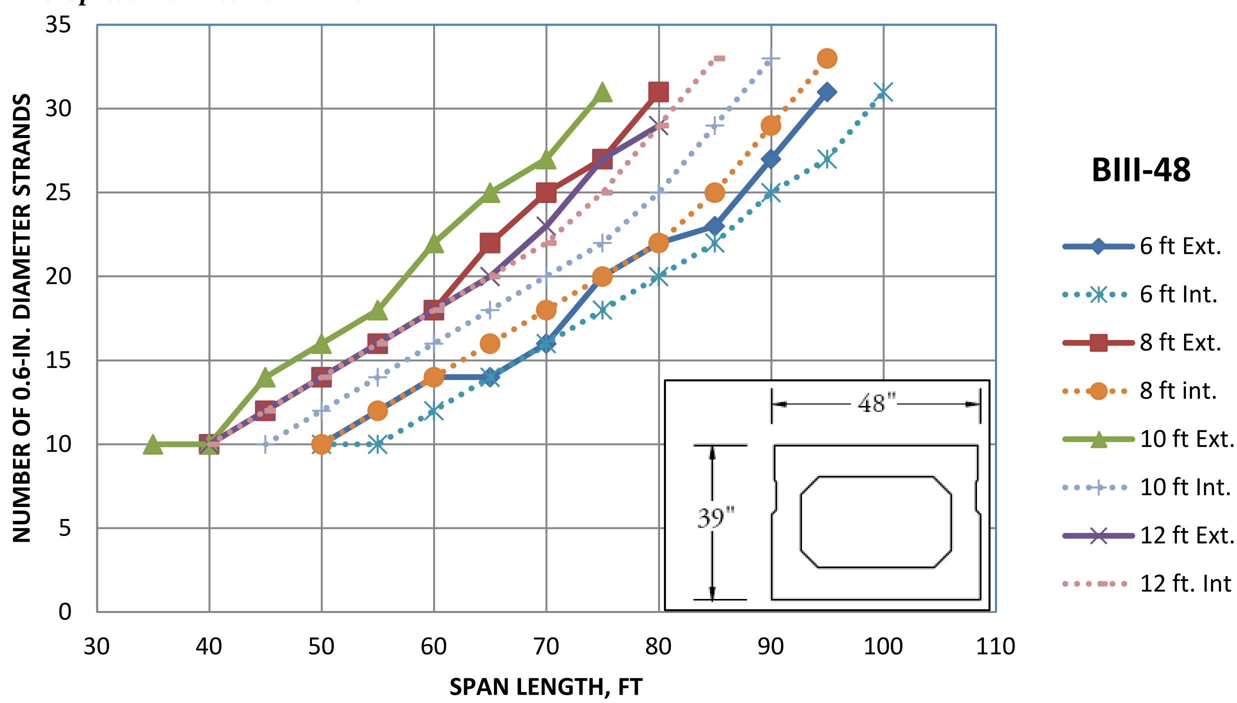


Chart BB-5
AASHTO Spread Box Beams BIV-48

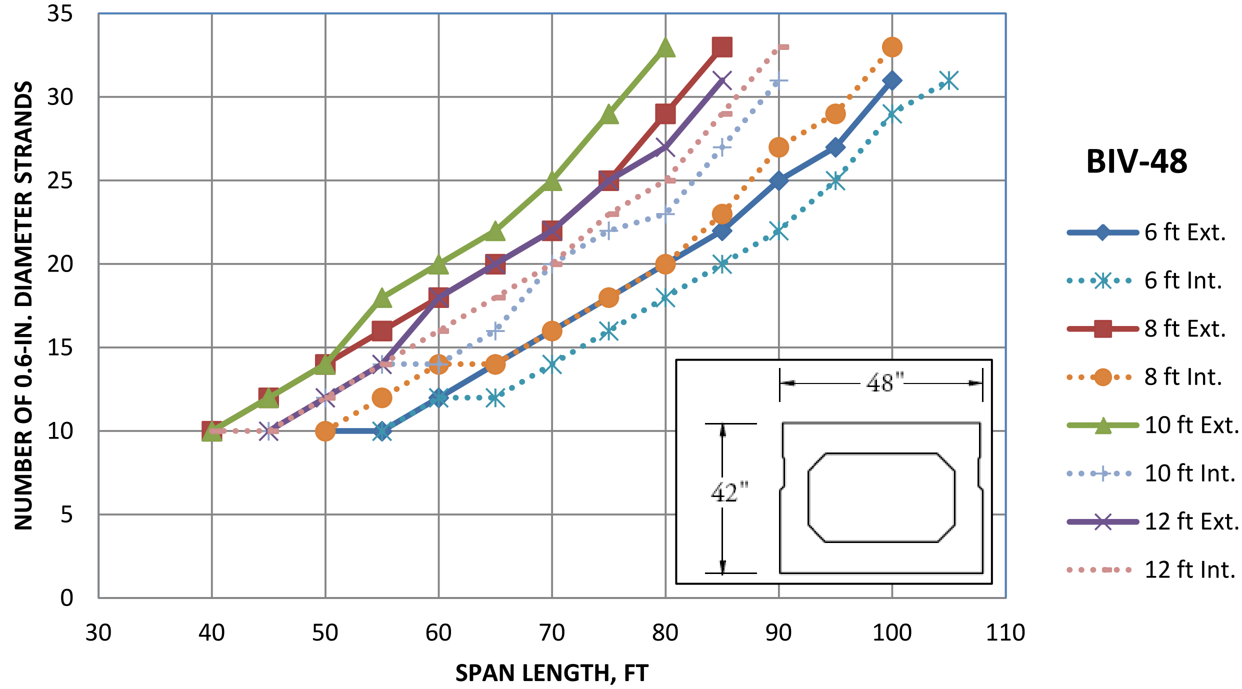


Chart BB-6
AASHTO Box Beams 36 in. Wide

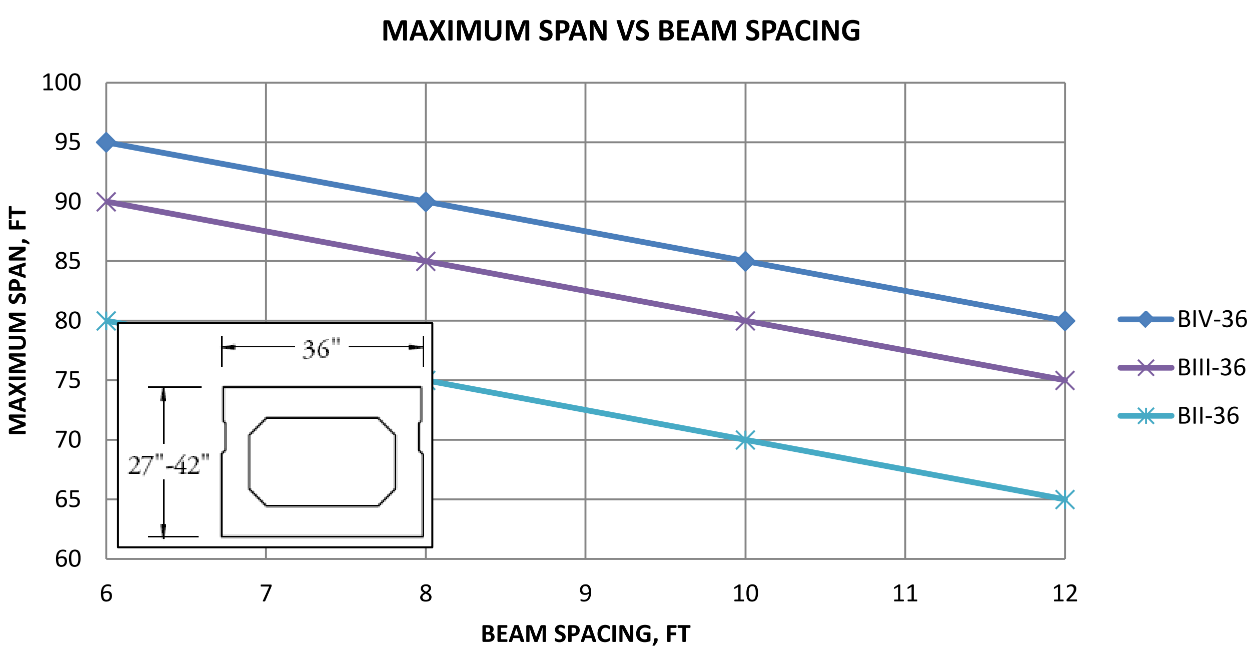


Chart BB-7
AASHTO Adjacent Box Beams 36 in. Wide

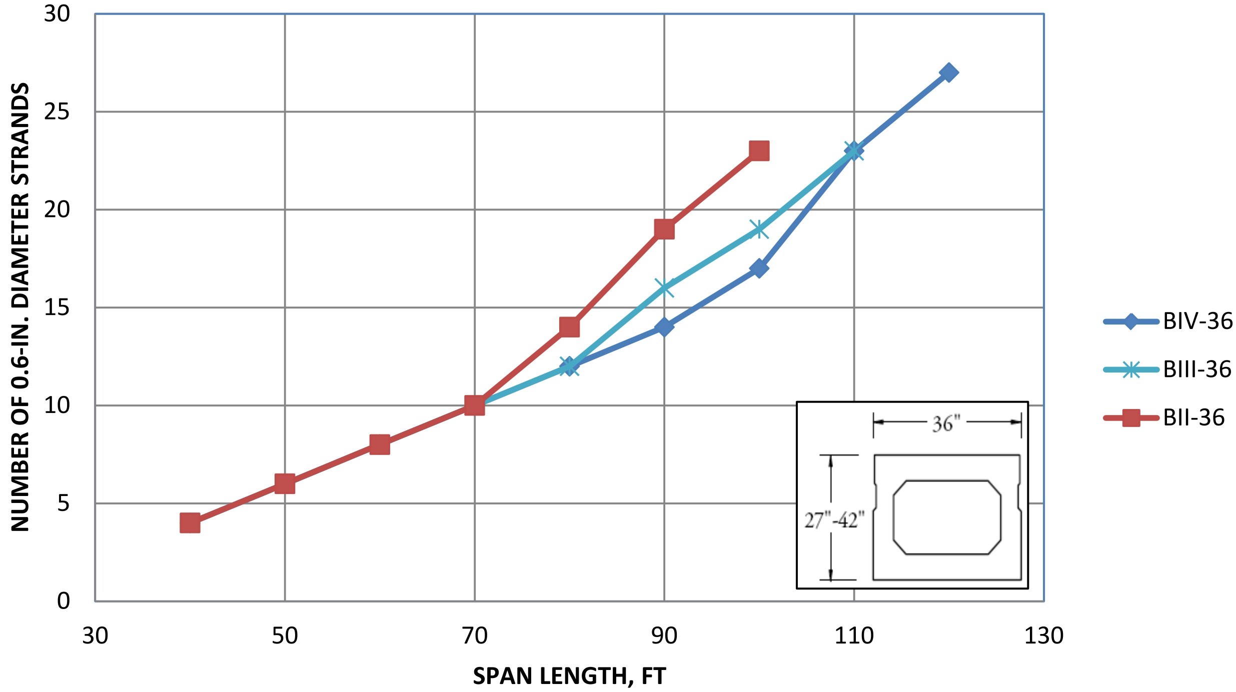


Chart BB-8
AASHTO Spread Box Beams BII-36

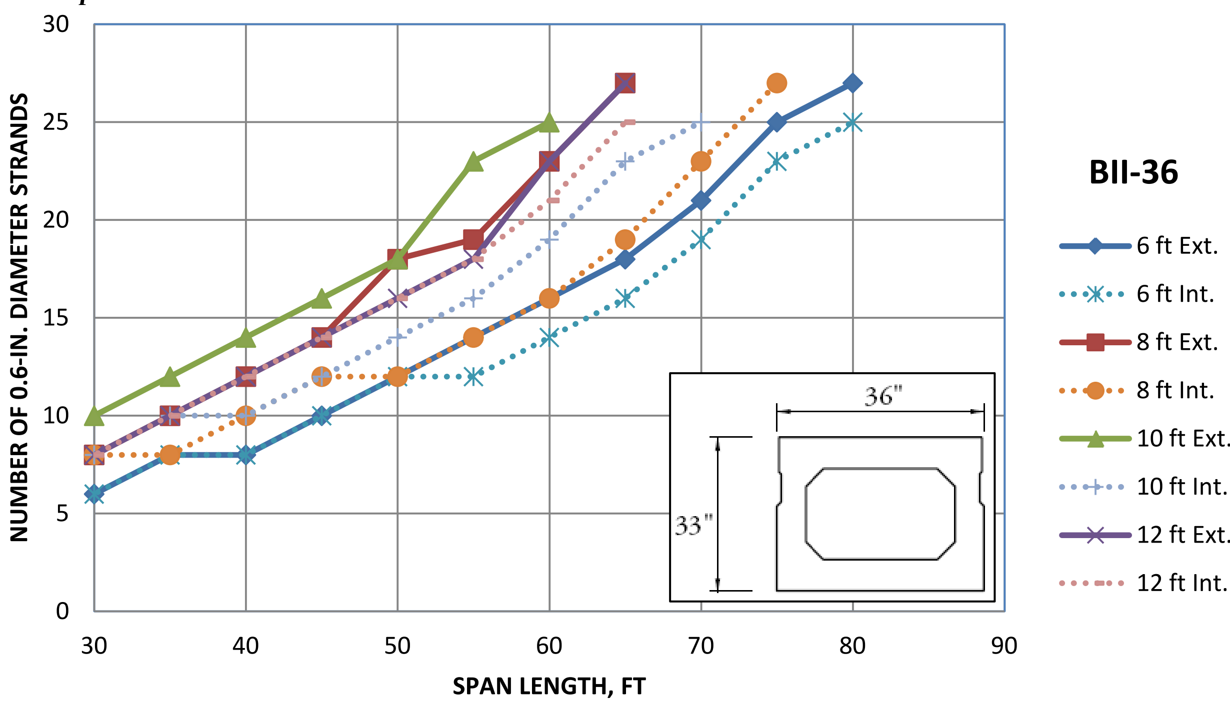


Chart BB-9
AASHTO Spread Box Beams BIII-36

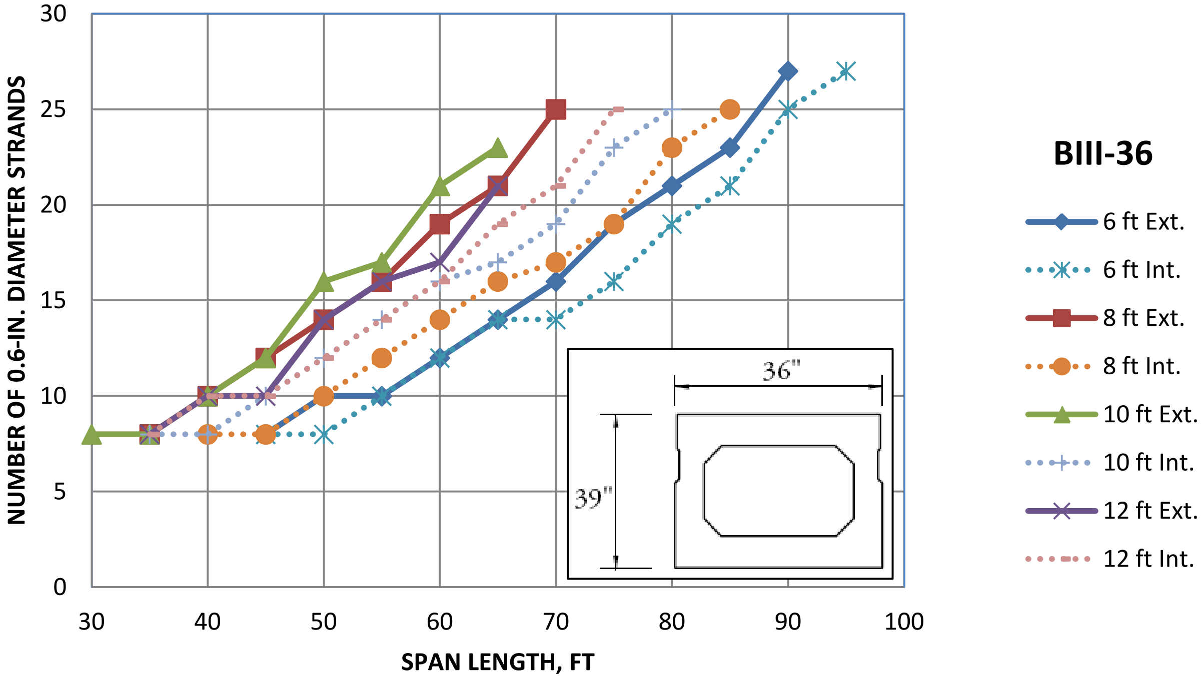


Chart BB-10
AASHTO Spread Box Beams BIV-36

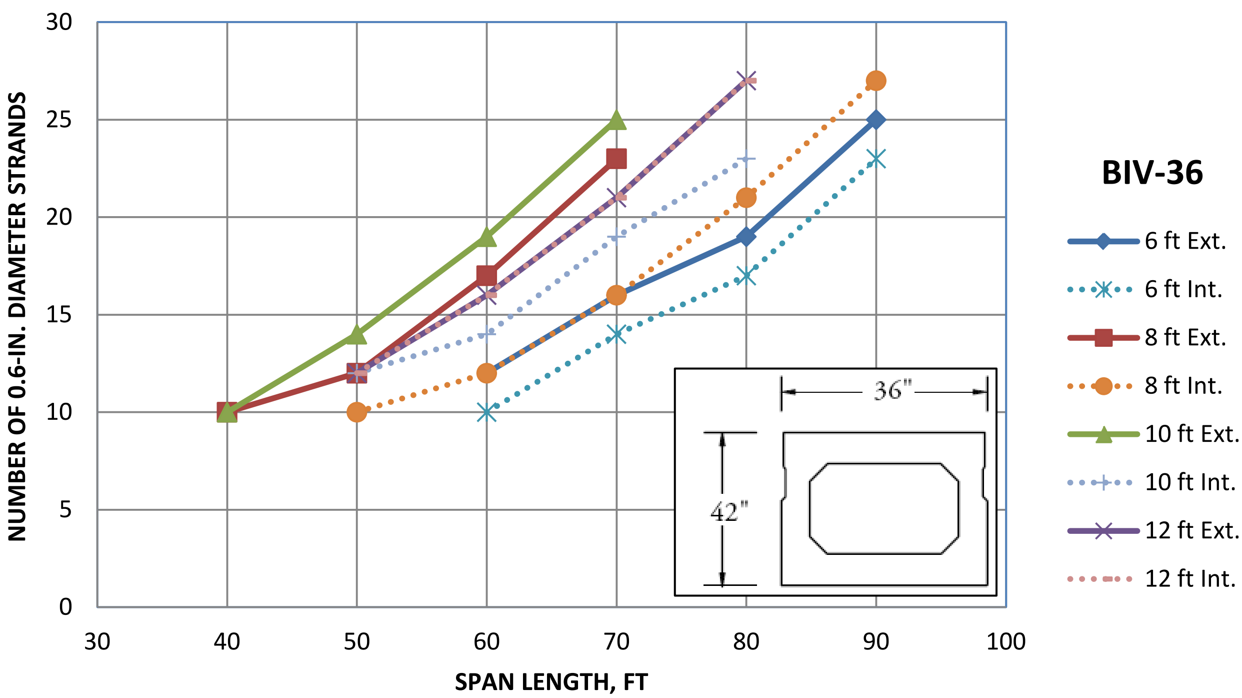


Chart BT-1

AASHTO-PCI Bulb-Tees

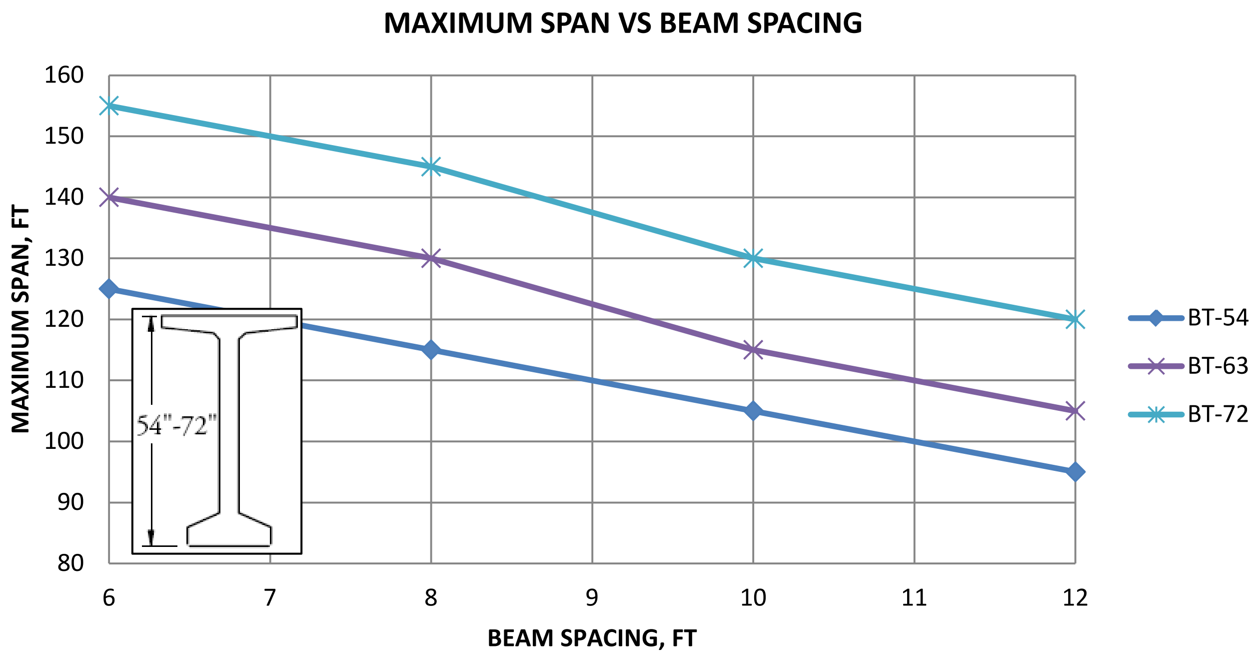


Chart BT-2
AASHTO-PCI Bulb-Tees BT-54

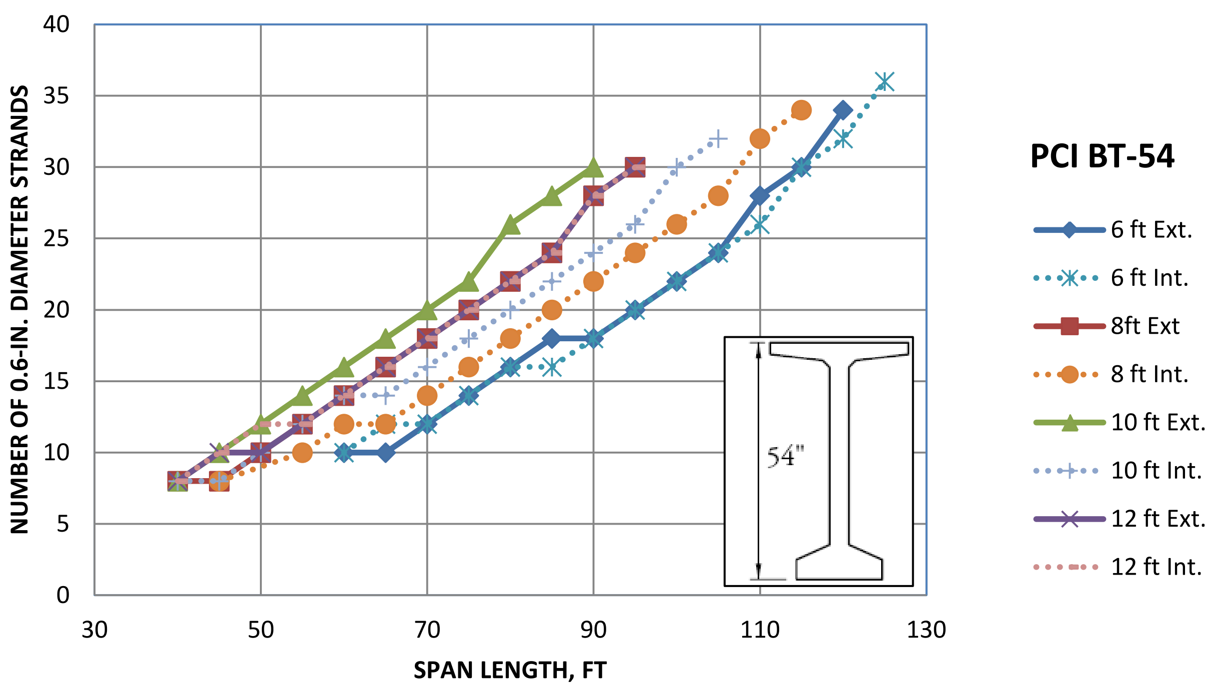


Chart BT-3
AASHTO-PCI Bulb-Tees BT-63

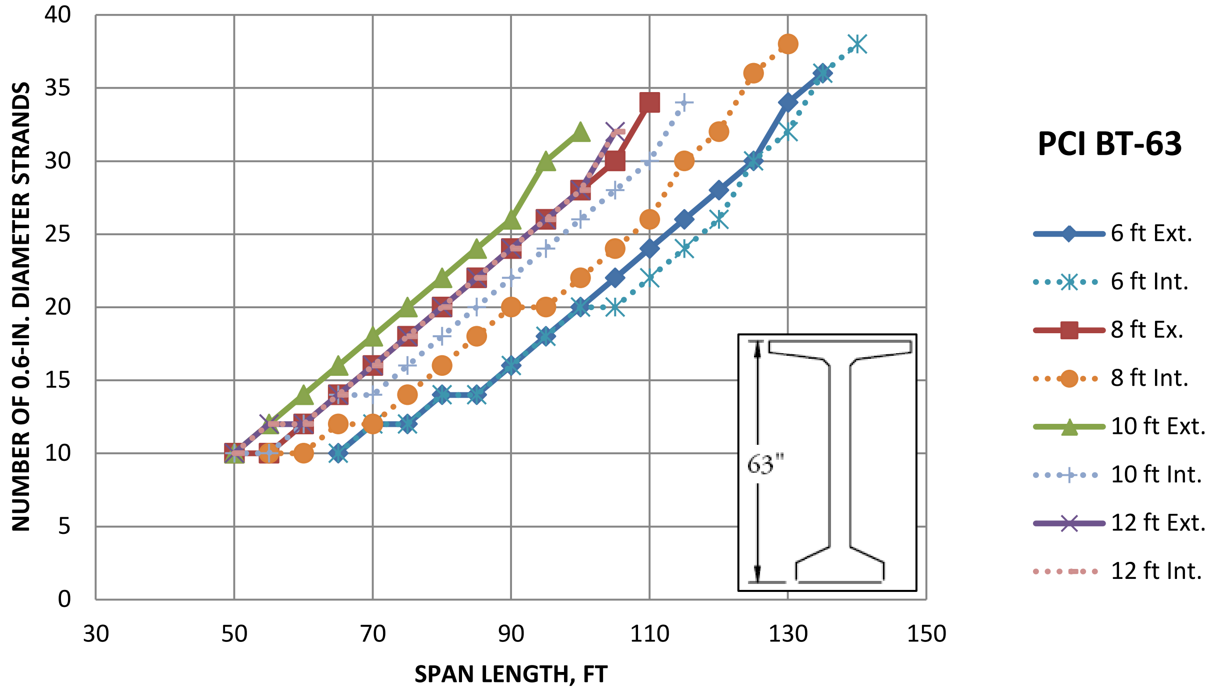


Chart BT-4
AASHTO-PCI Bulb-Tees BT-72

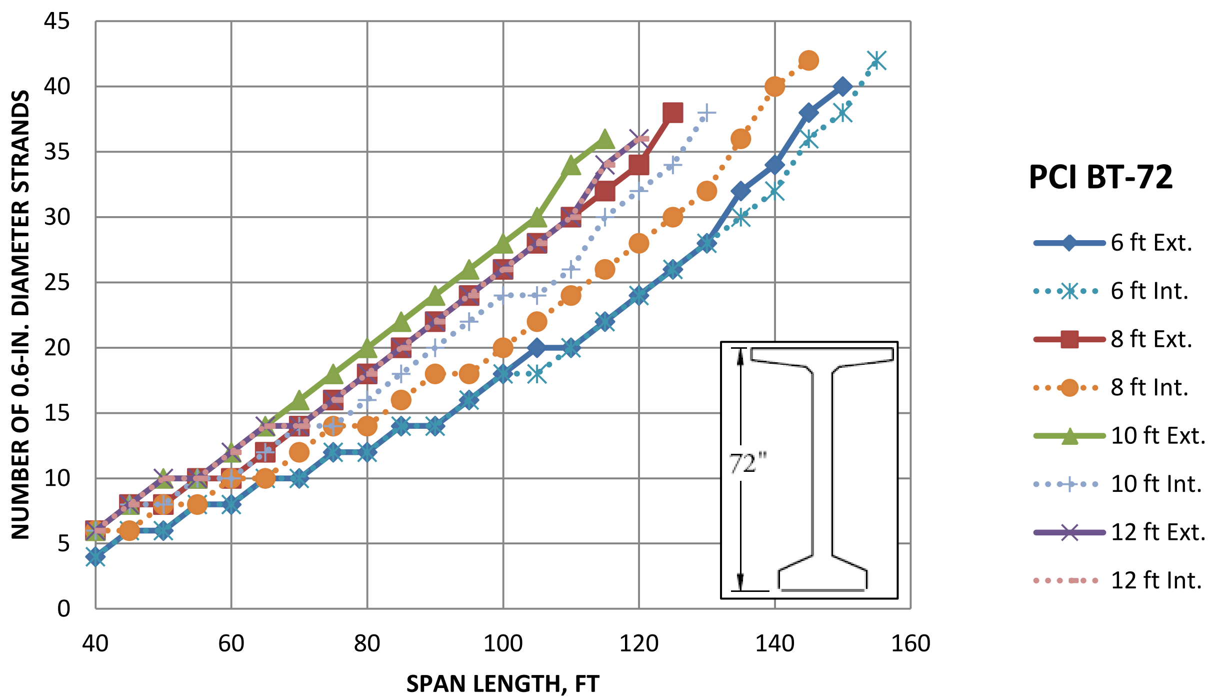


Chart DBT-1
Deck Bulb-Tees

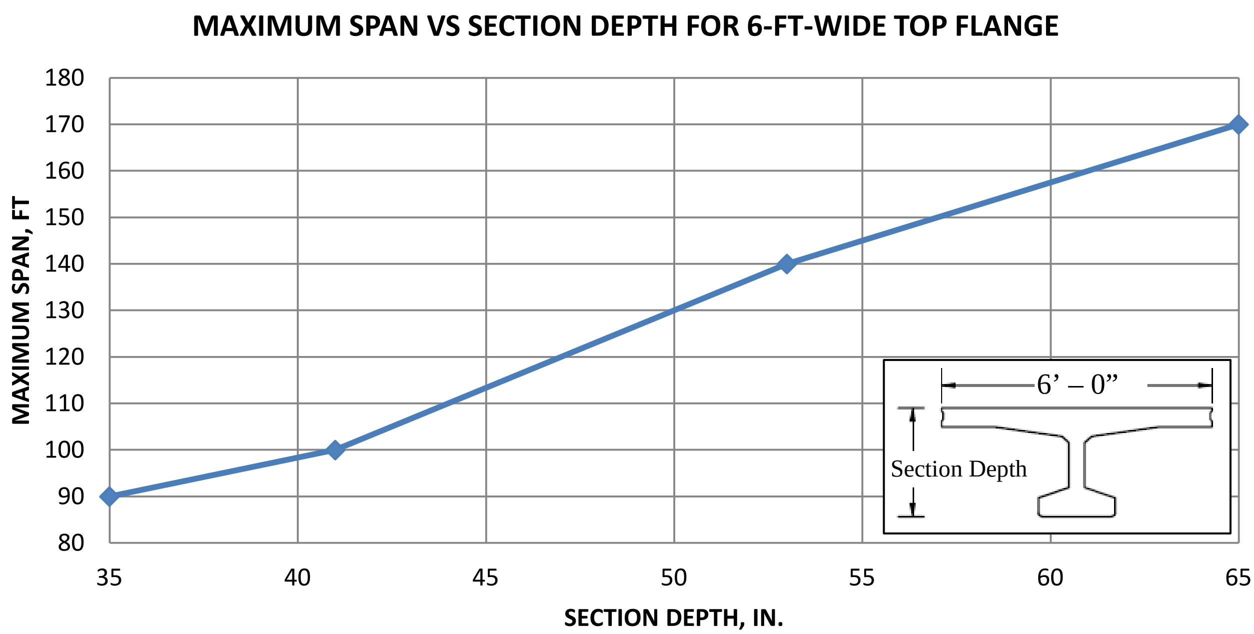


Chart DBT-2
Deck Bulb-Tees

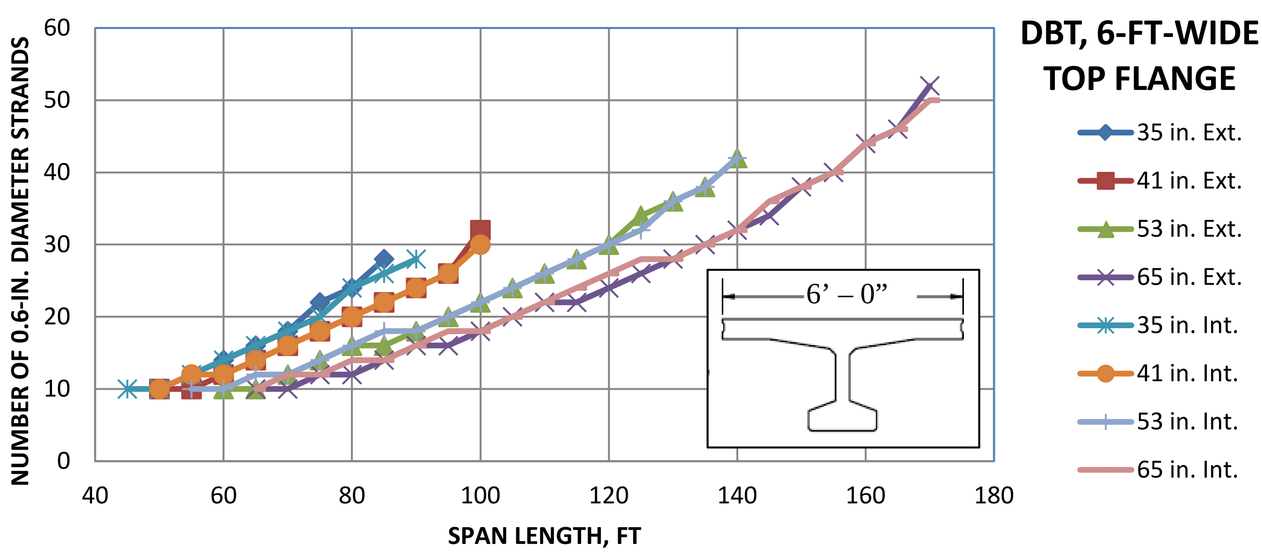


Chart IB-1
AASHTO I-Beams

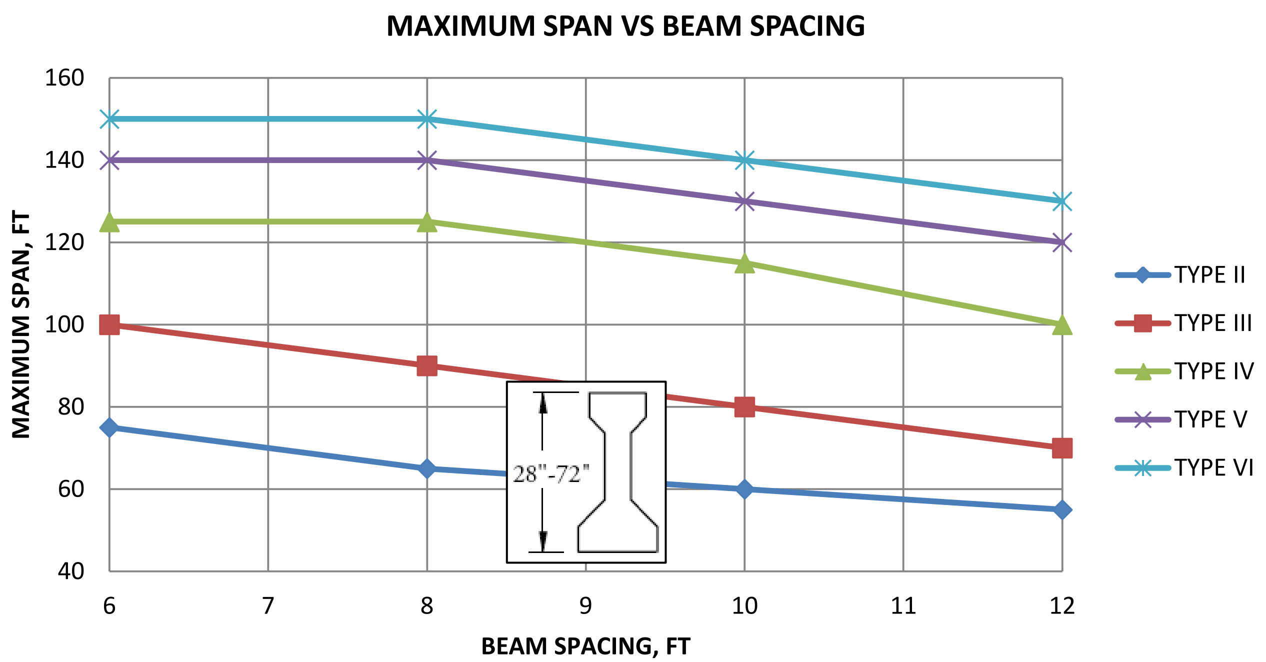


Chart IB-2
AASHTO I-Beams Type II

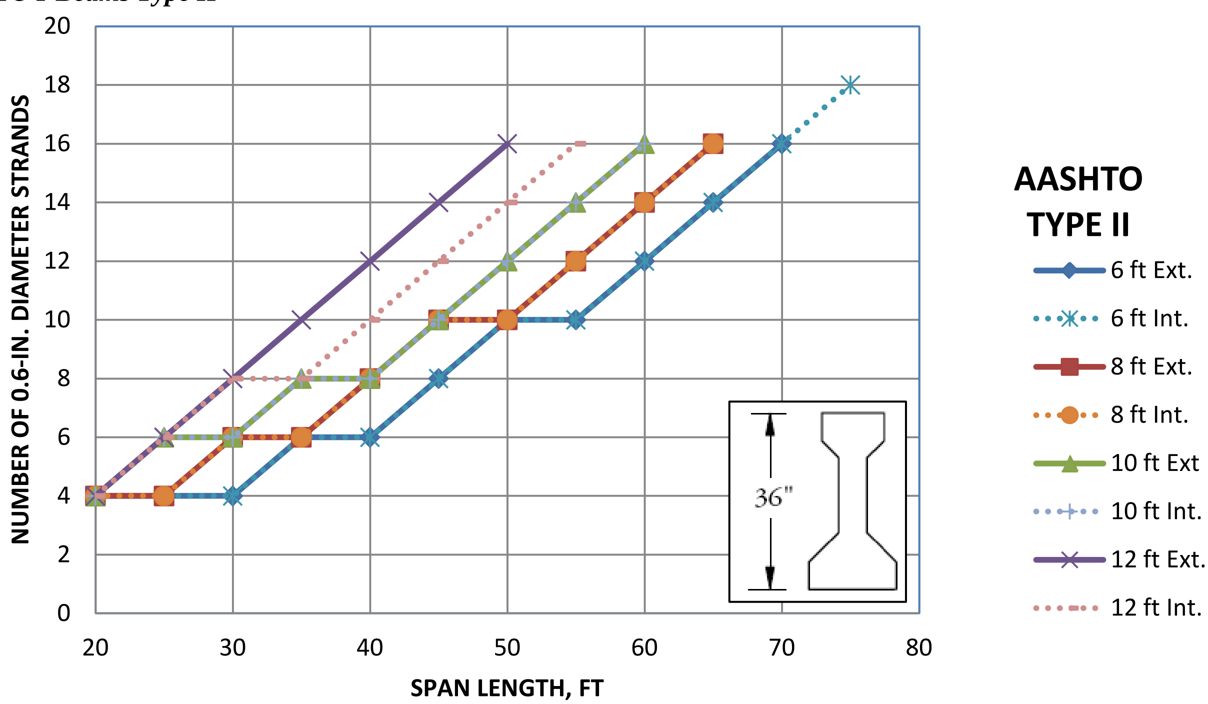


Chart IB-3
AASHTO I-Beams Type III

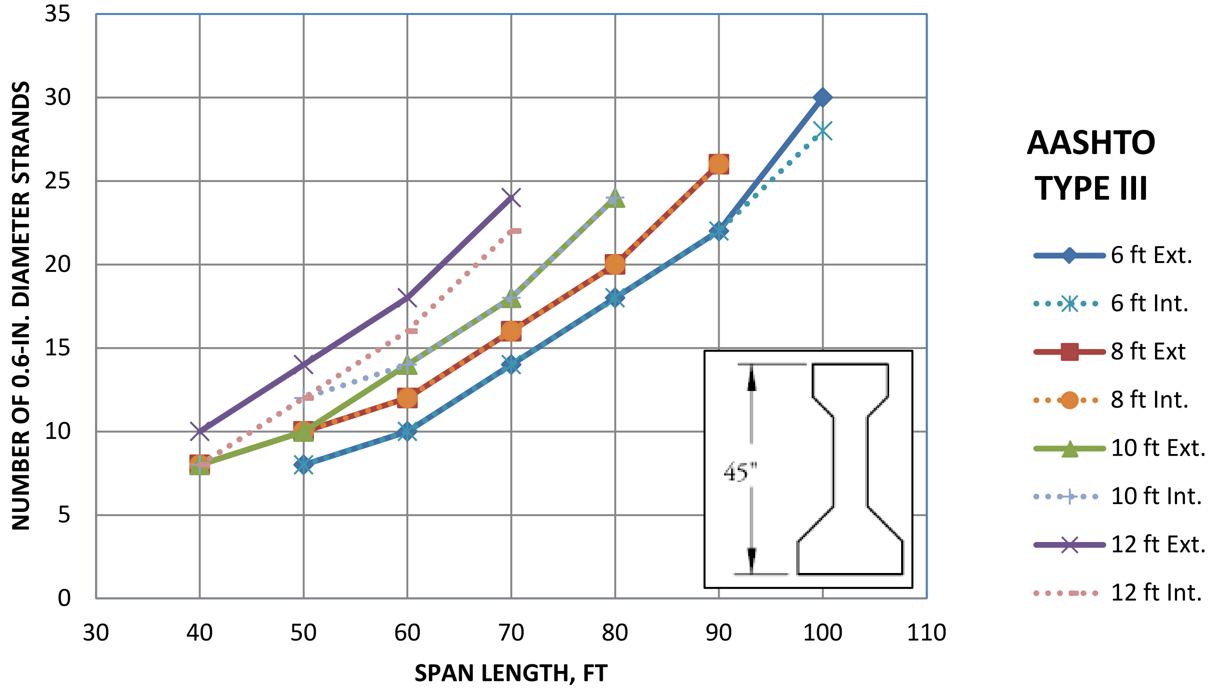


Chart IB-4
AASHTO I-Beams Type IV

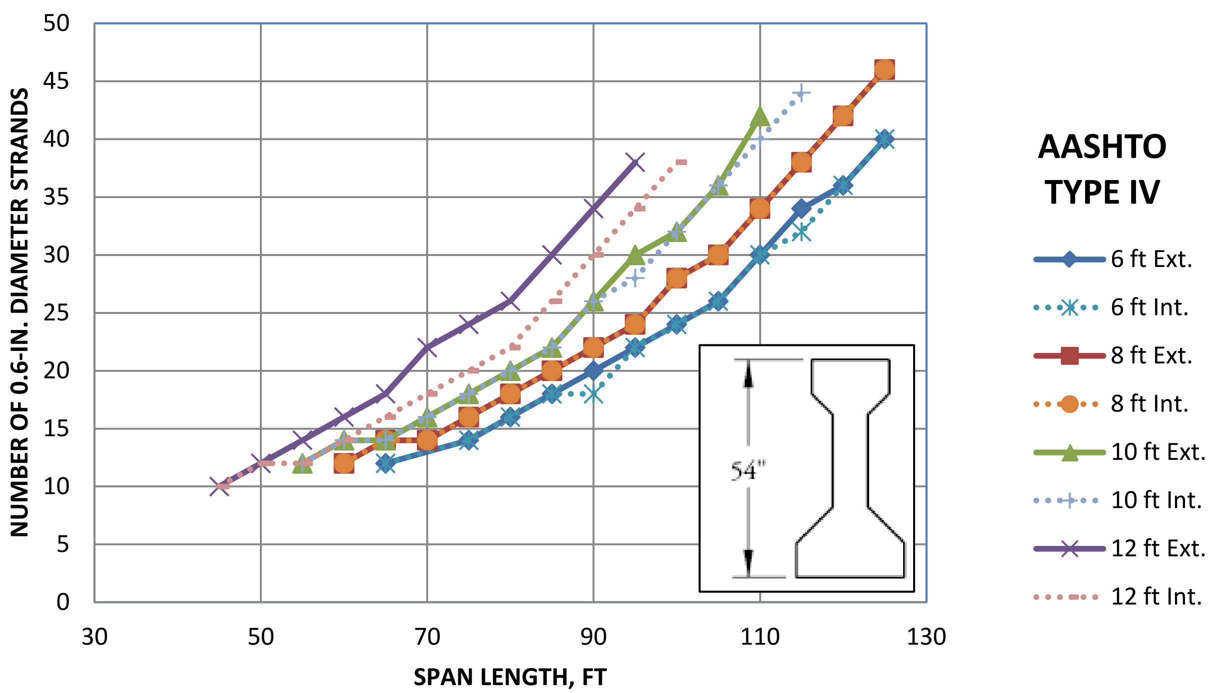


Chart IB-5
AASHTO I-Beams Type V

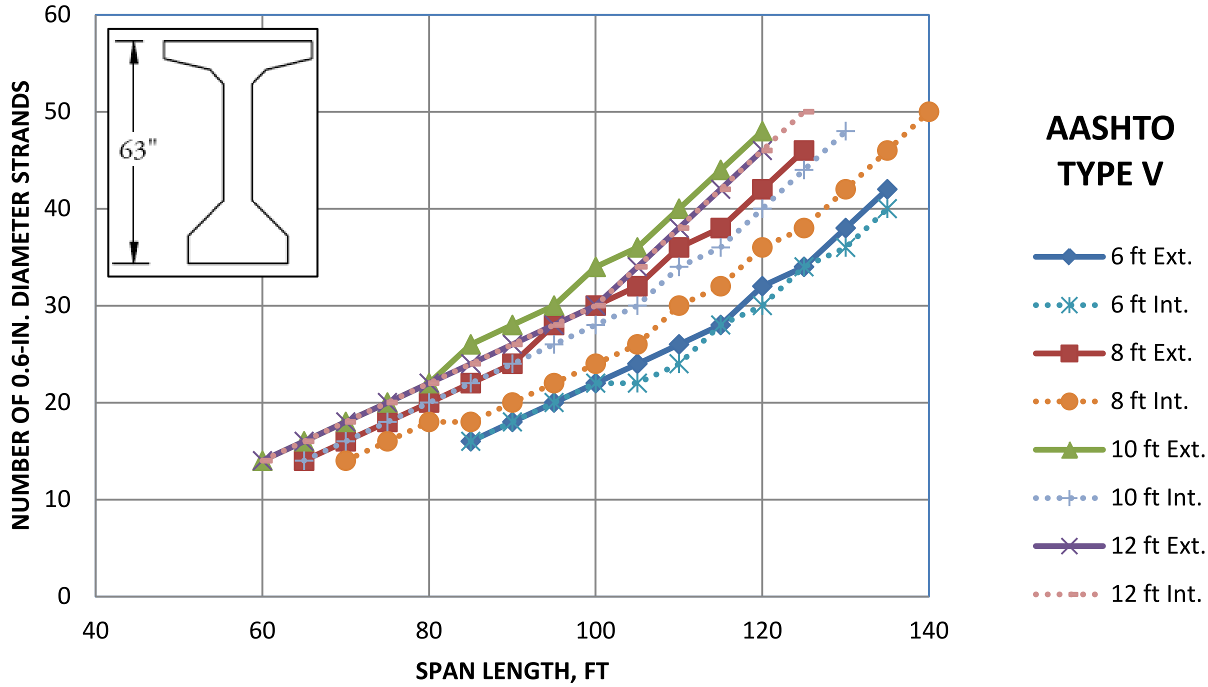


Chart IB-6
AASHTO I-Beams Type VI

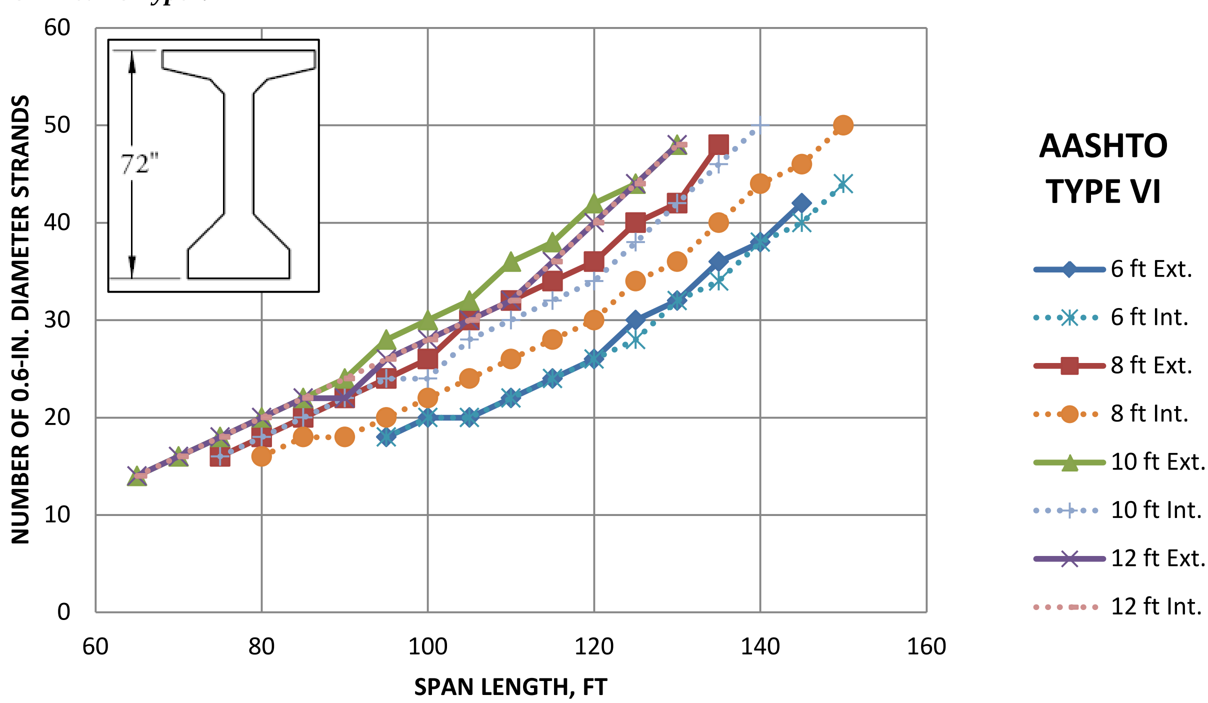


Chart NEXT-1
NEXT Type D Beams

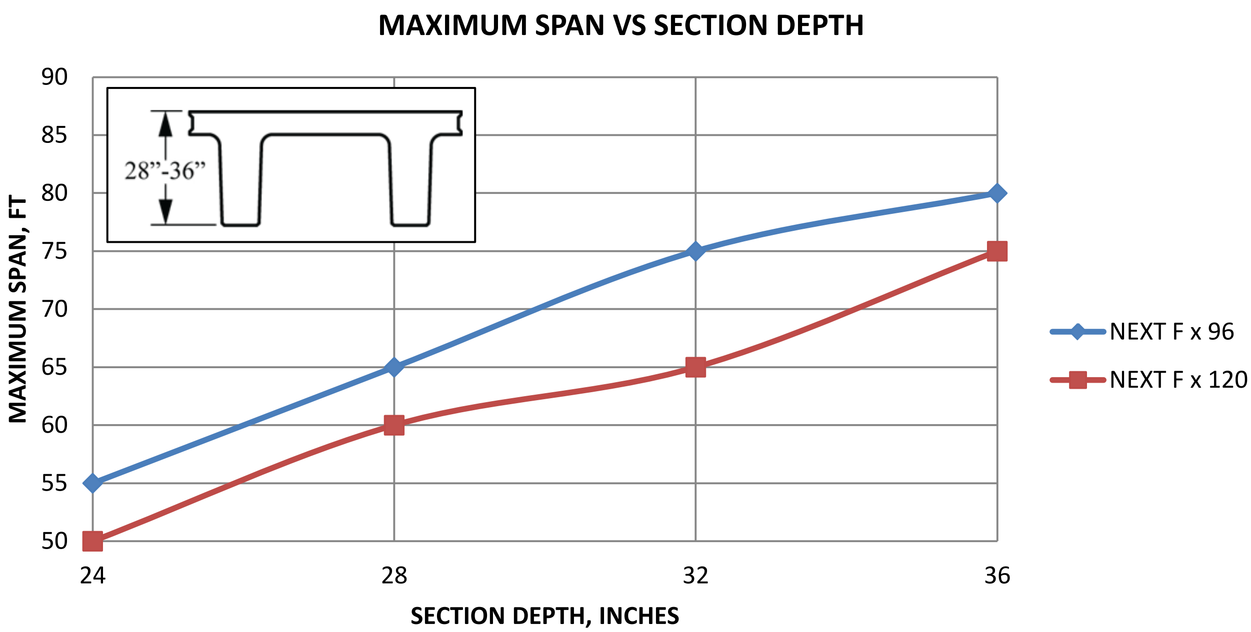


Chart NEXT-2
NEXT Type D x 96 Beams

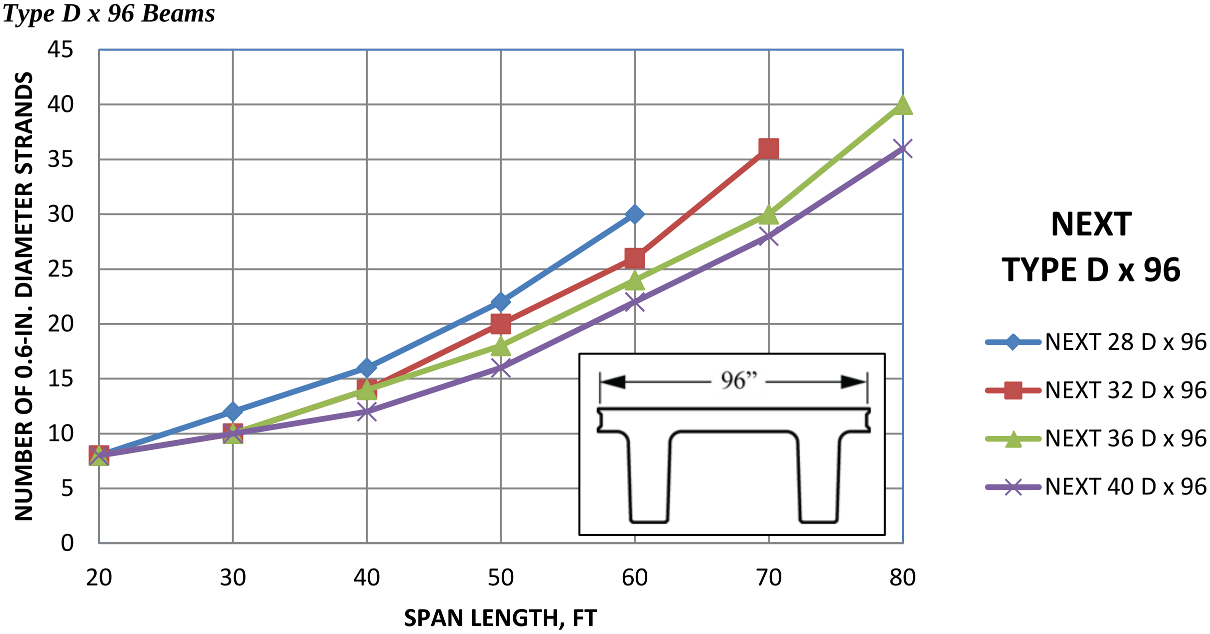


Chart NEXT-3
NEXT Type D x 120 Beams

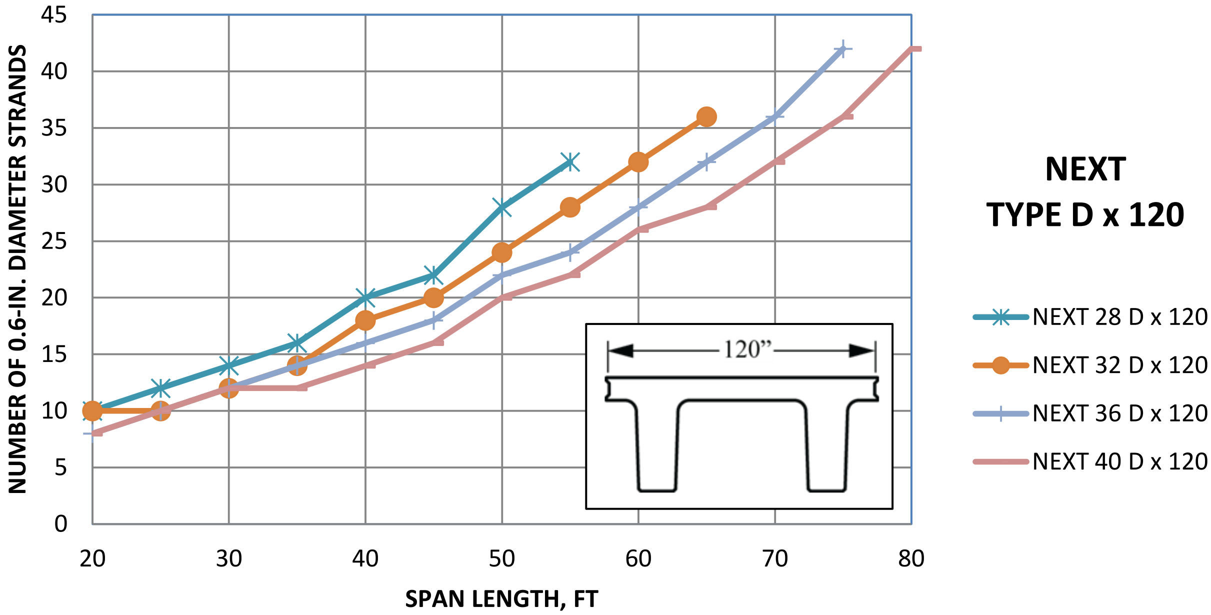


Chart NEXT-4
NEXT Type F Beams

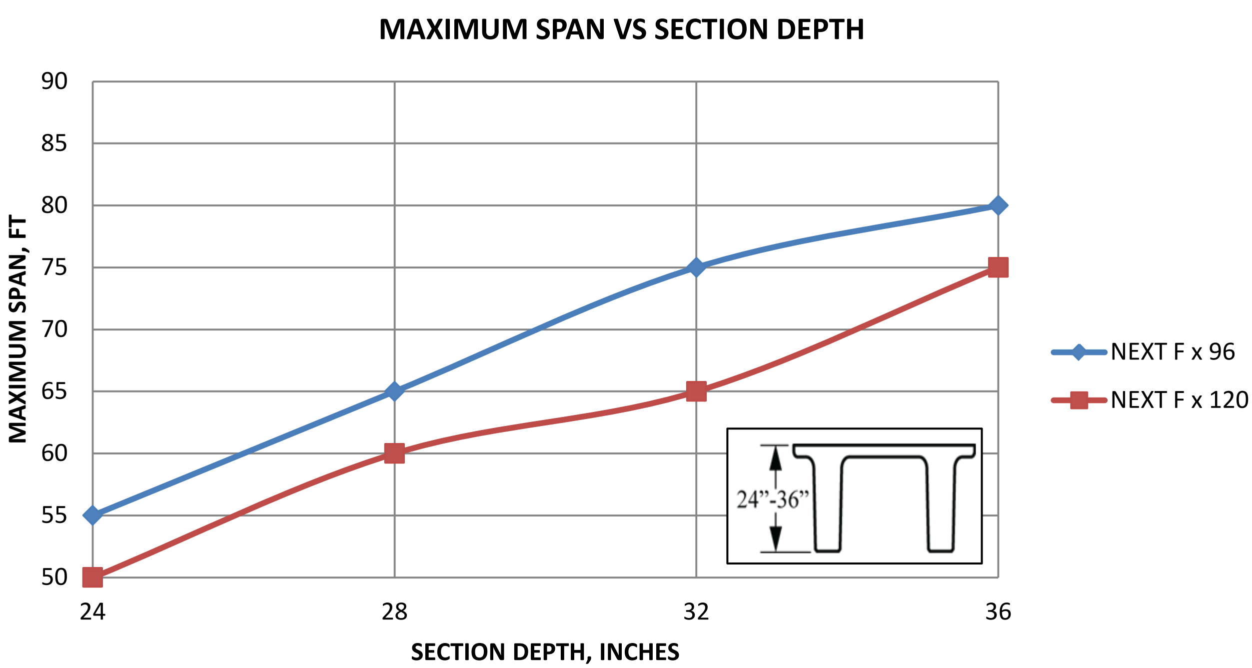


Chart NEXT-5
NEXT Type F x 96 Beams

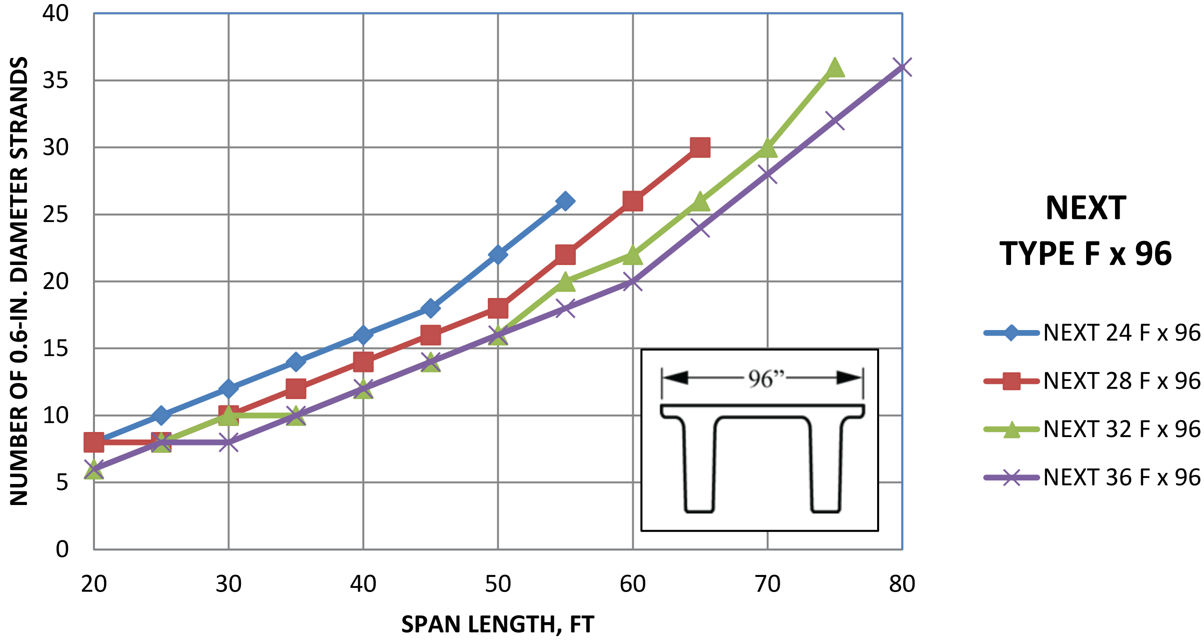


Chart NEXT-6
NEXT Type F x 144 Beams

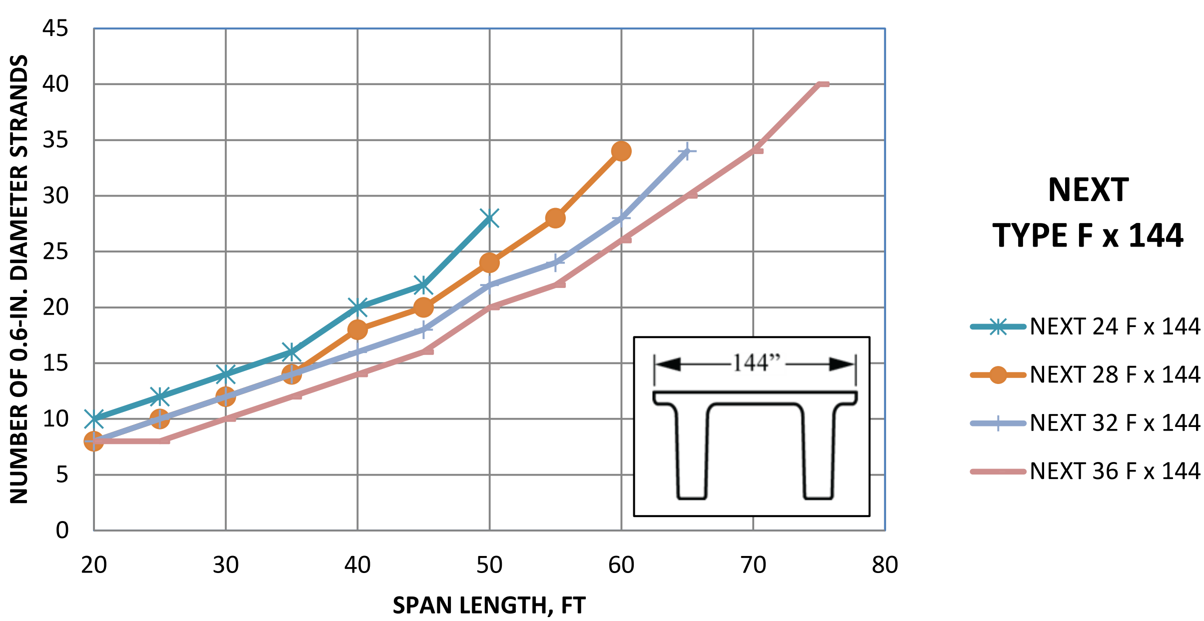


Chart U-1
U-Beams

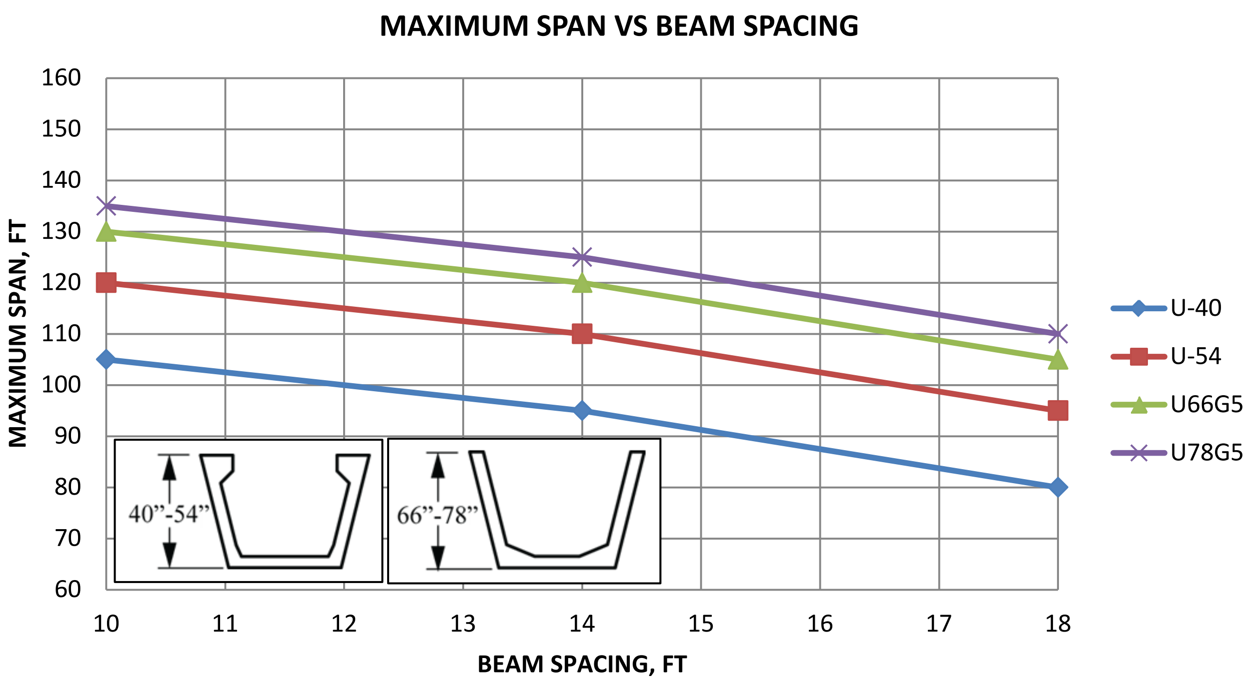


Chart U-2
Texas U-40 Beams

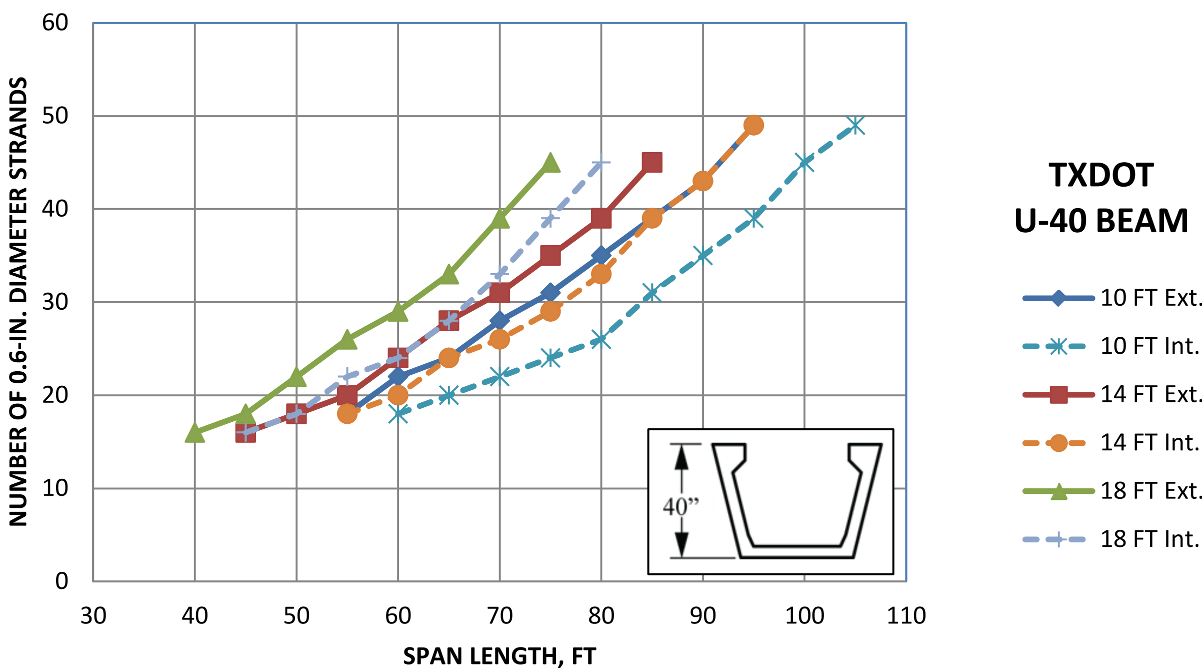


Chart U-3
Texas U-54 Beams

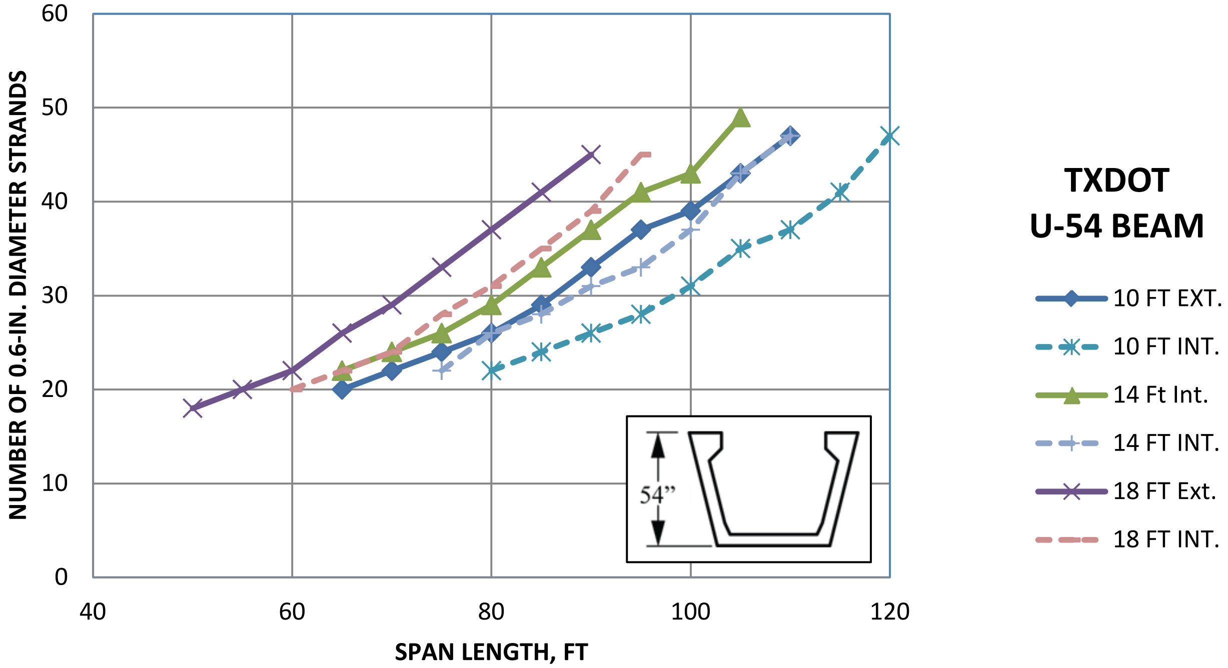


Chart U-4
Washington U66G5 Beams

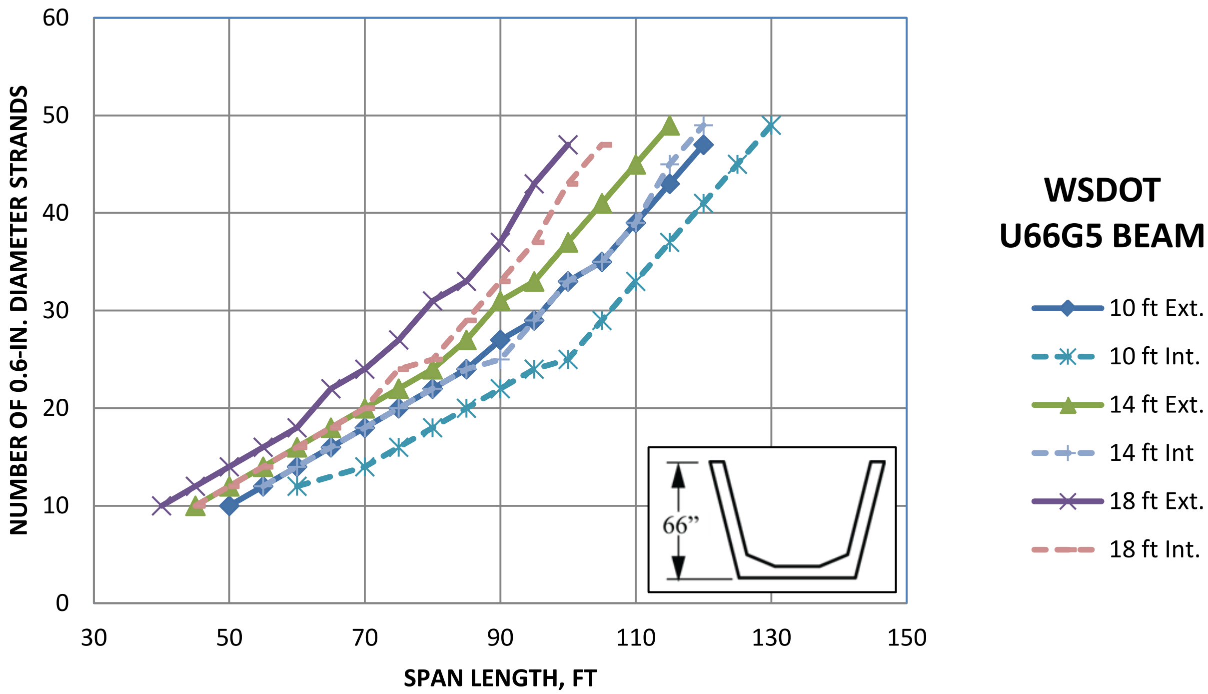
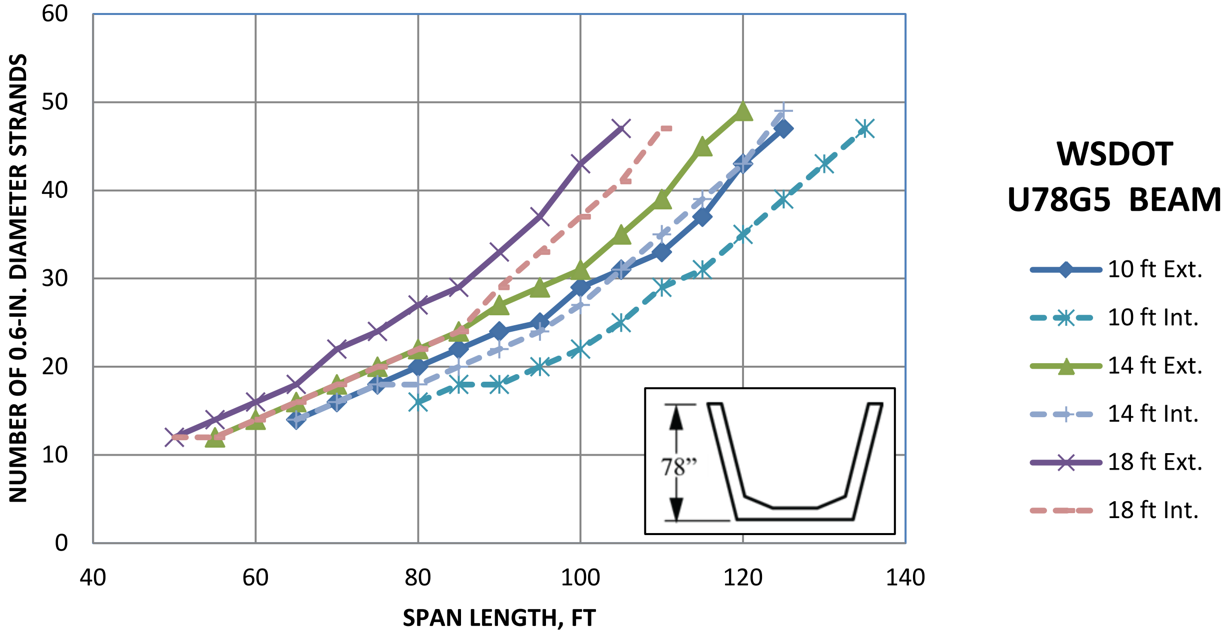


Chart U-5
Washington U78G5 Beams



6.10 PRELIMINARY DESIGN DATA

This section contains input data and results from computer runs to generate the preliminary design charts presented in **Section 6.9.** These table numbers correspond to the chart numbers in **Section 6.9.**

Table BB-1
AASHTO Box Beams 48 in. Wide – Maximum Span (ft) vs. Beam Spacing

Beam \ Spacing	4 ft	6 ft	8 ft	10 ft	12ft
BIV-48	120	105	100	95	90
BIII-48	115	100	95	90	85
BII-48	100	90	85	80	75

*A minimum concrete transfer strength of 3.0 ksi is recommended by PCI MNL-116 section 5.3.17.
++Final camber is net deflection after all losses and noncomposite and composite dead loads are applied.

Table BB-2
AASHTO Adjacent Box Beams 48 in. Wide

Spacing ft	Span ft	Slab Thickness in.	f'_{ci} ksi	No. of Strands	Final Camber in.++	f_b @ $L/2$ ksi	f_t @ $L/2$ ksi	M_u @ $L/2$ ft-kips	M_r @ $L/2$ ft-kips	Control
AASHTO BII Adjacent 48-in.-Wide Exterior Box Beam										
BII	40	6	1.358*	6	0.08	0.059	0.454	817	1,077	Strength
BII	45	6	1.344*	6	-0.02	-0.121	0.610	992	1,077	Strength
BII	50	6	1.813*	8	0.03	-0.053	0.720	1,186	1,414	Strength
BII	55	6	1.800*	8	-0.18	-0.269	0.910	1,393	1,414	Strength
BII	60	6	2.266*	10	-0.18	-0.238	1.051	1,612	1,741	Strength
BII	65	6	2.727*	12	-0.21	-0.229	1.208	1,843	2,058	Strength
BII	70	6	3.185	14	-0.27	-0.240	1.382	2,088	2,365	Strength
BII	75	6	3.178	14	-0.87	-0.517	1.631	2,345	2,365	Stress
BII	80	6	4.091	18	-0.58	-0.326	1.779	2,615	2,951	Stress
BII	85	6	4.540	20	-0.87	-0.399	2.001	2,898	3,231	Stress
BII	90	6	4.986	22	-1.26	-0.493	2.240	3,194	3,502	Stress
BII	95	6	5.612	25	-1.54	-0.517	2.490	3,503	3,873	Stress
BII	100	6	6.409	29	-1.65	-0.479	2.754	3,825	4,327	Stress
AASHTO BIII Adjacent 48-in.-Wide Exterior Box Beam										
BIII	40	6	0.822*	4	-0.02	-0.105	0.414	836	846	Strength
BIII	45	6	1.266*	6	0.04	0.005	0.481	1,015	1,253	Strength
BIII	50	6	1.254*	6	-0.06	-0.158	0.625	1,214	1,253	Strength
BIII	55	6	1.694*	8	-0.02	-0.083	0.720	1,427	1,648	Strength
BIII	60	6	2.130*	10	0.04	-0.025	0.828	1,652	2,033	Strength
BIII	65	6	2.121*	10	-0.21	-0.226	1.009	1,890	2,033	Strength
BIII	70	6	2.554*	12	-0.22	-0.198	1.143	2,142	2,408	Strength
BIII	75	6	2.547*	12	-0.64	-0.424	1.349	2,406	2,408	Stress
BIII	80	6	2.979*	14	-0.75	-0.427	1.508	2,685	2,773	Stress
BIII	85	6	3.407	16	-0.92	-0.447	1.682	2,976	3,128	Stress
BIII	90	6	3.833	18	-1.16	-0.484	1.868	3,281	3,474	Stress
BIII	95	6	4.675	22	-0.88	-0.321	2.015	3,600	4,137	Stress
BIII	100	6	4.885	23	-1.56	-0.502	2.256	3,932	4,298	Stress
BIII	105	6	5.653	27	-1.53	-0.416	2.467	4,277	4,879	Stress
BIII	110	6	6.409	31	-1.54	-0.359	2.693	4,637	5,427	Stress
BIII	115	6	6.789	33	-2.33	-0.503	2.965	5,009	5,690	Stress
AASHTO BIV Adjacent 48-in.-Wide Exterior Box Beam										
BIV	40	6	0.799*	4	0.00	-0.061	0.373	845	905	Strength
BIV	45	6	1.228*	6	0.06	0.059	0.429	1,027	1,340	Strength
BIV	50	6	1.215*	6	-0.02	-0.088	0.561	1,229	1,340	Strength
BIV	55	6	1.639*	8	0.04	0.002	0.643	1,444	1,765	Strength
BIV	60	6	1.626*	8	-0.12	-0.168	0.798	1,672	1,765	Strength
BIV	65	6	2.046*	10	-0.09	-0.104	0.903	1,914	2,179	Strength
BIV	70	6	2.033*	10	-0.37	-0.297	1.080	2,169	2,179	Stress
BIV	75	6	2.447*	12	-0.40	-0.258	1.209	2,437	2,583	Strength
BIV	80	6	2.857*	14	-0.45	-0.233	1.349	2,719	2,977	Strength
BIV	85	6	3.263	16	-0.53	-0.222	1.502	3,015	3,361	Strength
BIV	90	6	3.250	16	-1.17	-0.459	1.724	3,325	3,361	Stress
BIV	95	6	3.651	18	-1.40	-0.473	1.900	3,648	3,735	Stress
BIV	100	6	4.047	20	-1.71	-0.499	2.087	3,985	4,100	Stress
BIV	105	6	4.640	23	-1.77	-0.426	2.259	4,336	4,630	Stress
BIV	110	6	5.001	25	-2.32	-0.497	2.487	4,701	4,954	Stress
BIV	115	6	5.724	29	-2.28	-0.376	2.689	5,079	5,574	Stress
BIV	120	6	6.075	31	-3.06	-0.474	2.942	5,472	5,872	Stress