SMO 5: Precast Prestressed Concrete Girders

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1 Design Requirements & Guidelines

Prestressed precast concrete (ppc) girders are economic, durable, and can be adapted for a variety of geometries including curved and tapered bridges. They are TDOT's girder of choice for spans between 30 ft. and 140 ft. and spans as long as 155 ft. have been constructed in Tennessee.

Bridge designers tend to use the term girder and beam interchangeably though technically, girder is probably the more desirable term, following the building industry where girders are the primary support members and beams frame into them.

Articles, sections, equations, and table references are from the 6th edition of AASHTO LRFD Bridge Design Specifications 2012 unless otherwise noted.

1.1 Design Specifications

We are required to use AASHTO LRFD Bridge Design Specifications for safety and federal funding. It is, therefore, important to adhere to AASHTO as closely as possible.

SMO is the office policy of the Division of Structures, created to share the Division's experience and to help give direction where AASHTO is unclear or silent.

1.2 Analysis Method

We use the Approximate Method of analysis (4.6.2). This is a line girder analysis with distribution factors computed using equations or the lever rule.

1.3 Filler Depth & Beam Seat Elevations

Before designing the girder, check the filler (aka haunch) depth necessary to avoid conflicts between the top of girder and the bottom of slab and to take into consideration the extra load a filler can potentially add to a girder. The absolute minimum filler at the edge of all girders needs to be 1" thick. At center line (CL) girder the minimum filler depth should be 1 ½" thick for box beams and AASHTO i-girders; 2" thick is recommended for bulb-tees because they are more prone to camber growth. The 1 inch thickness along the edge of the top flange is to accommodate a 1 inch thick bituminous fiberboard used to support deck panels.
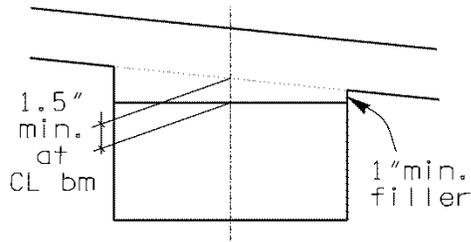
Figure 1 Filler Depth

Calculating filler depth is easy check for a straight bridge , on a straight grade, with a constant cross slope. For bulb tees with a 2% deck cross slope, filler depth at CL girder = 0.02(42"/2)+1" = 1.42", therefore, 2" would be recommended. For a 8% cross slope, filler depth = 0.08x21"+1" = 2.68", therefore, round up to 3" at CL girder.

Note that if the filler depth is so deep the stirrups protruding from the top flange cannot project above the bottom mat of slab reinforcement or do not project above the top of a prestressed deck panel, the filler will require reinforcement (K bars) to engage the slab steel with the stirrups.   If filler depth varies a lot between spans or even within a span, K-bar vertical leg lengths will need to vary accordingly and be called out in the plans.

Figure 2 K-Bars

Calculating filler depth can be a complex exercise for geometrically complex bridges. Often the exterior girders control but not always, so the designer may need to check filler depth for all girders at 10th or 20th points along CL of girder. The factors affecting filler depth are cross slope, vertical curves (sag and crest), horizontal curves (SE transitions), location of the finished grade (FG) line on the deck, skew, and top flange width.

For a curved bridge, ppc girders are chords between substructures supporting a curved deck. The offsets from CL girder to edge of slab vary: at mid-span, the exterior girder with the minimum offset to the edge of slab may encroach into the bottom slab, and on the other side of the deck, the exterior girder with the maximum offset may have excessive fillers.

Filler depths need to be checked for decks in sag and crest curves. Without sufficient filler depth in a sag curve, the deck can pinch out the filler or even start pinching out itself

over the top of the girder. A crest curve can thicken the filler and increase loading in the middle of the span; a sag curve can increase load toward the supports.

The use of Geomath, the TSC program, or your own spread sheet will help in identifying filler problems.

1.4 Design Loads

HL93 loading is the design live load, 3.6.1.2 - Design Vehicular Live Load:

- single span bridges use design lane, design truck, design tandem,
- multi-span bridges use design lane, design truck, design tandem, double truck, and double tandem.

Double truck and double tandem places two trucks or two tandem trucks in a lane, one ahead of the other, on adjacent spans to maximize negative moments at CL pier, 3.6.1.3 – Application of Design Vehicular Live Loads.

Non-Composite simply-supported spans exist prior to curing of the deck. This condition includes:

- Dead load (DL) of the girder
- Dead load of the uncured slab, filler, intermediate diaphragms, and forms
  Subsequent loads are applied to composite spans made continuous with support (aka continuity) diaphragms (4.6.2.1.6, composite design section) and include:

- Live Load, LL (short term modular ratio, $n = E_p / E_c = 28{,}500$ ksi $/ 1820\sqrt{(f'_c \text{ ksi})}$, 5.7.1 & 5.4.2.4)
- Dead Load of parapets, sidewalks, median barriers, and any other DL component, DC (long term modular ratio, 2n, 5.7.1)
- Dead Load of wearing surface and utilities, DW = 35 psf (long term modular ratio, 2n)

1.5 Concrete Compressive Strength

The following defines our standard practice regarding design of prestressed girders:

- $f'_{ci} = 4$ ksi, minimum initial release, meaning girders can be detensioned and removed from the forms

- f $'_c$ = 5 ksi, minimum 28-day strength

- f $'_{c\,slab}$ = 3 ksi for design, (specify 4 ksi on the plans for durability and margin of error for strength)

- Use increments of 100 psi to increase or decrease f $'_{ci}$ and f $'_c$ during design

- Actual strengths are determined by design and should be kept as low as possible.

- f $'_c$ ≥ 9.5 ksi, should be used sparingly and approved by your CE Manager 1 or above before being incorporated into the design of prestressed girders.

1.6 Strand Stressing Limits

For prestressed concrete girders we only specify low-relaxation (low-lax) strands released at 75 percent of ultimate, per Table 5.9.3-1.

Strands must all be pulled to the same force for any given girder. Partial stressing of strands is prohibited, except when two strands are added to the top of a girder as an aid to the fabricator to support the reinforcement cage and to help check camber growth. **Add a note** to the girder sheet that these two strands are to be pulled to 5 kips each.

If the number of 6/10-inch low-lax strands per girder is greater than 44, the engineer must check with fabricators to make sure they can tension them.   Local fabricators include CPI - Construction Products, Inc., Jackson, 731-668-7305 and Ross Prestressed Concrete, Inc., Knoxville, 865-524-1485.

1.7 Deflection Upward

Use f $'_{c\,slab}$ = 3 ksi for calculating girder deflections.   All girders must have upward deflection under total dead load, DC + DW. Since wearing surface may not be applied any time soon and we specify f $'_{c\,slab}$ = 4 ksi, it might be wise to minimize upward deflection because prestressed girders tend to increase their camber over time.

If upward deflection of the girder is just up from zero and two partially stressed strands are added, those strands should be considered in the design of the girder to make sure the girder will not deflect downward.

No multiplier factors for camber/deflection are allowed for prestressed girders by the Specifications. Use a factor of 1.0 in the Conspan Program.

1.8 Strand Size and Type

According to AASHTO LRFD C5.4.4.1, low-lax strands are now considered the standard. We commonly use 1/2-inch and 6/10-inch diameter low-lax strands in girders but nothing bigger.

 The 6/10-inch diameter strand allows the designer to effectively utilize girders with f 'c ≥ 8,000 psi but also to reduce the number of strands required when f 'c < 8,000 psi.

For prestressed members, the strands must all be the same size for any given member, except the two strands added to the top of the girder to support the stirrups as mentioned previously above, which can be a 3/8-inch diameter.

Table 1 Low-Lax Strand Properties

Nominal diameter, in.	1/2	6/10
A_{ps} = cross sectional area, in^2	0.153	0.217
Weight, plf	0.525	0.740
0.75 fpu Aps, kips Initial Pull	31.0	44.0
$f_{pu} A_{ps}$, kips	41.3	58.6
All have a specified ultimate tensile strength f_{pu} = 270 ksi		
See Table 5.9.3-1 – Stress limits for Prestressing Tendons.		

1.9 Strand Development

For the development of prestressing strands see 5.11.4.

1.10 Tension Rebars

Sufficient reinforcement shall be placed in the top of prestressed girders to make use of the increased tensile limiting stress (5.9.4.1.2 & Table 5.9.4.1.2-1). In AASHTO Figure C5.9.4.1.2-1, "x" can be solved for using the release stresses in the top and bottom of the girder in a ratio. Solve for A_s which will be a maximum at the point where transfer occurs, that is, near the ends of the girder:

$$f_{ci\ top} / x = f_{ci\ bot} / (h_{girder} - x)$$

$$x = h_{girder}\ f_{ci\ top} / (f_{ci\ top} + f_{ci\ bot})$$

$$A_s f_s = T = x\ b_{top}\ f_{ci\ top} / 2$$

$$A_s = x\ b_{top}\ f_{ci\ top} / (2f_s)$$

where $f_s = 0.5f_y \leq 30$ ksi
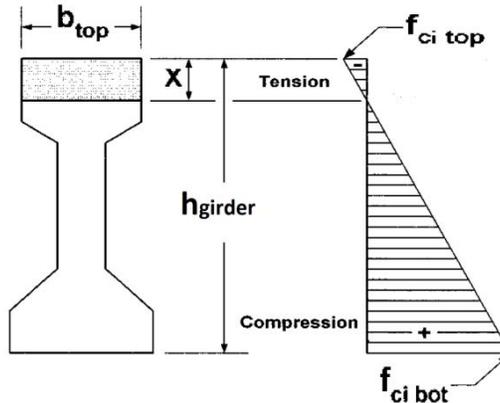
Figure 3 Tension Bars in Top Flange, AASHTO Figure C5.9.4.1.2-1

Tension reinforcement should be symmetric about center line girder and a minimum of two #4 bars for AASHTO I-girders and four #4 bars for Bulb-Tees on 1'-0" centers.

1.11 Girder Length Adjustments

Girder lengths are typically projected horizontal lengths and may not be long enough to give sufficient bearing. Roadway grade, elastic shortening, shrinkage, and thermal contraction can all cause the girder to miss proposed centerline bearing, sometimes by inches. The designer should adjust girder length for roadway grade and **add a note** to the beam sheet to alert the fabricator that girder lengths have been increased to account for roadway grade. It is assumed the fabricator will adjust for elastic shortening and shrinkage, where these are significant enough to apply.

Thermal expansion and contraction could be an issue for long girders. Using a 70° range (90°F-20°F) and 6.0 x 10⁻⁶ ft/°F, a girder might change in length by 0.00504 inches/ft (3.12.2 & 5.4.2.2). A 150 foot bulb-tee fabricated on an 90° day and placed on a 20° day could shrink ¾". One end of that girder could get ¾" less bearing. Two ways to deal with this issue might be to lengthen the girder ¾" or widen its bearing pad.

With bridges on steep grades, another consideration is whether to have the fabricator cast the ends of the girder square to the girder or plumb. The ends of the girder may need to be plumb in order to properly meet the endwall or support diaphragm.

1.12 Diaphragms

Permanent support or end diaphragms are always required between all girders at substructures and intermediate diaphragms will be required as given below.

1.12.1 Intermediate Diaphragms

Use intermediate diaphragms for the following conditions:

for bulb-tee spans

1. up to 40 ft., none are required
2. 40-80 ft., place a diaphragm at mid-span of the beam
3. over 80 ft., place diaphragms at third points along the beam

for AASHTO I-girder spans

1. over 80 ft., place a diaphragm at mid-span of the beam
2. of curved bridges with a radius of 800 ft. or less, place a diaphragm at mid-span.

Intermediate diaphragms shall be cast-in-place concrete or galvanized steel (cross frames for bulb-tees and, less routinely, channels for i-beams). Intermediate diaphragms for bulb-tees and i-beams shall be placed normal to the girders for skews less than 75º measured from CL survey. See Standard drawings STD-14-1, STD-14-2, and STD-14-3 for further details and notes.

If intermediate diaphragms are required, designing girders with concrete intermediate diaphragms gives the contractor the option to have concrete or steel intermediate cross frames. If steel intermediate diaphragms are needed to make the girder design work or are required for some other reason, the plans will need a note restricting the contractor to steel intermediate diaphragms.

1.12.2 Support Diaphragms (see 5.14.1.4.10-Continuity Diaphragms)

Diaphragms at supports keep girders properly spaced and connect girder ends from span to span, forming continuous girder lines. To reduce maintenance problems, increase longevity, and make for better overall performance, we make our bridges jointless with support diaphragms. It's an attempt to create continuity with individual, simply supported, ppc girders and is accomplished by slab reinforcement and connecting bent strands in the bottom row of the girder ends with rebar hoops and encasing them in the diaphragm. Note

that strands above row 1 should be cut within 1" to 3" of the end of the girder to give enough clearance for the LS400 bars to be seated on the bent strands.
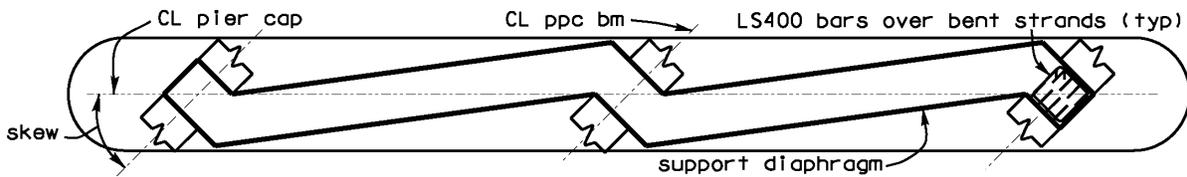
Figure 4 Plan View *of* Support Diaphragm

Ideally, we would only want to embed fully bonded strands in the diaphragms to avoid the possibility that camber growth might induce stress into debonded strands which should otherwise have relief of stress. Pragmatically, however, we have not experienced problems embedding debonded strands and not bending the same strand at each end of girder allows fabricators with fixed bed lengths to save almost 3 feet of space between consecutive girders in a production line of girders, thereby maximizing the space available for girders.

The following strategies are in place to allow end rotation of the girder prior to continuity, to avoid calculating restraint moments, to help reduce cracking in diaphragms/endwalls, and to enhance continuity:

1. 90 days after detensioning is the earliest time a girder can receive a full depth end diaphragm/endwall. If the girders are placed on supports before this time, only the bottom 15 inches of the diaphragm shall be poured (see SMO11 note F-11). The remainder of the diaphragm is poured when the slab over the support is poured.

2. It has been determined that the majority of the creep and shrinkage of prestressed girders occurs in the first 90 days after the strands have been detensioned (5.14.1.4.4-Age of Girder When Continuity Is Established). **However**, for projects with compressed construction schedules, the 90-day requirement may be waived, if the contractor submits for approval to the Structures Division a girder design stamped by a PE, that (1) includes restraint moments or (2) treats a multi-span bridge as a series of simple spans without continuity, and with no additional cost to the contract.

3. When pouring the slab for a multi-span bridge, the middle section in adjacent spans shall be poured to within at least 15 feet of the supports prior to or concurrently with the slab over those supports (see SMO50 plans note).

4. Pouring the bottom 15 inches of the end diaphragm right after the girders are erected provides bracing to the ends of the girders. This will also prevent the beams from walking and the elastomeric bearing pads from being rolled up at the bents and will not restrict the rotation of the girder.

5. The Contractor shall provide temporary bracing until the diaphragms are poured and cured.

Plans note: The support diaphragms at the bents shall be formed and the bottom 15 inches poured as soon as possible after the girders have been set. The remainder of the diaphragm shall be poured concurrently with the deck slab and shall be made after a minimum of 90 days from the date of girder casting. For projects with compressed construction schedules, the 90-day requirement may be waived, if the contractor submits an alternate girder design stamped by a PE to the Division of Structures for approval. The alternate design shall include restraint moments or treat the girders as a series of simple spans, and the design shall not add additional cost to the contract. All diaphragm concrete shall be included in the quantity for item 604-03.09, Class "D" Concrete (Bridge Deck).

Division policy, until further notice, assumes ppc girders are continuous once slab and support diaphragms are integral with the girders. This is a debated assumption because some diaphragms have cracked and the girders appear to have detached themselves from the diaphragm, casting doubt on the assumption of continuity. Consequently, some designers have designed composite girders as simply supported with minimal slab reinforcement instead of using the composite section to resist negative moment. Since our bridges have performed satisfactorily, please continue to assume continuity.

1.13 Transforming Strands

Article 5.9.1.4 allows the transforming of bonded and developed strands with the modular ratio. This increases the moment of inertia of a girder and may be useful in design.

Transforming strands is not recommended at this time because rating software can not transform. Another problem is that unless girder design software allows selective transforming, transforming will only yield accurate results where all strands are bonded and developed, for example, at mid girder. At mid girder, transforming may help if there is a

tension problem or downward deflection there. Transforming will not yield accurate results in regions where bond is broken or where strands are not yet developed. A possible design strategy is:

1. 1st run - do not transform strands and see if the girder will design, which will be a conservative design.
2. 2nd run - transform strands only if there are unsolvable problems in the area where strands are fully bonded.

1.14 Bond Breaks

When bond breaks (aka debonding, shielding, or wrapping strands) are required by design, the following requirements (5.11.4.3) shall be met:

1. do not break bond on vertically or horizontally adjacent strands or exterior strands in the bottom row;
2. for the rows above the bottom row, try to avoid placing bond breaks on the exterior strands or at least place the shortest bond breaks there;
3. for Bulb-Tee and I-girders, a maximum of 25% of the total number of strands may be debonded and 40% of a single horizontal row;
4. for box beams, bond breaks are not to exceed 40% of the total number of strands,
5. office policy is to maintain these limits which can only be exceeded with permission from the CE Manager 1 or above and requires shear in the bond break range to be designed for in accordance with Section 5.8, not accounting for the prestress and designing only as a mildly reinforced concrete section.

1.15 Draping Strands

Draping (aka harping, deflecting, or raising) strands permits the designer to extend the span limits on ppc girders. Each fabricator is different, but most prefer to avoid draping because of safety and cost issues.

Try to avoid draping strands but if it is essential, maximize the number of bond breaks and minimize the number of draped strands.

- Manufacturers of hold-down devices (aka strand restraining devices) like Dayton Superior and Meadow Burke (daytonsuperior.com, meadowburke.com) specify a

maximum safe working load per strand (7.5 kips, but to be conservative, use 5.0 k for ½" dia. strands & 6.0 k for $^6/_{10}$", Ross Prestress) and a max. safe working load per device (use 40.0 k for ½" & 48.0 k for $^6/_{10}$"). The designer must check both the upward force of a single strand and also the group increased by the friction loss factor, f, and compare these to the manufacturer's safe working loads. The friction loss caused by the hold down devices is 5% according to manufacturer's but placed somewhere between 5% and 15% according to beam fabricator Ross Prestress. If the safe working loads need to be exceeded, check with manufacturers and/or fabricators.
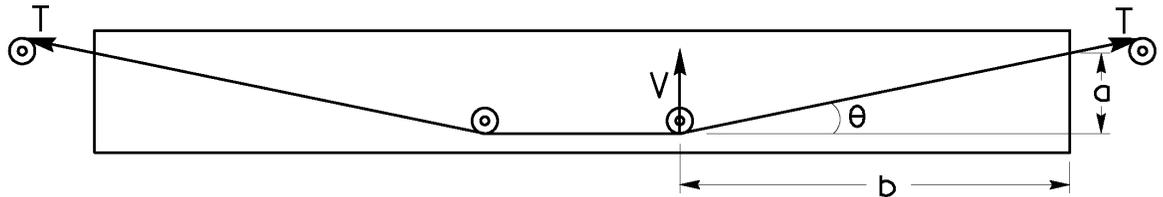
Figure 5 Upward Force on Hold Down Device

- V= upward force = $(1+f)T\sin\theta \approx (1+f)T(a/b)$, where T = initial pull on strand.

- Hold down points are typically at 0.4L from each end of the girder where L is the precast girder length. They could be just about anywhere as long as the upward component of the strand does not exceed the above limits. Moving the hold down points toward the ends of the girder can gain capacity in the girder, but increases the upward pull on the hold-down devices which should be checked.

- Bulkhead capacity varies according to fabricator, but use a total strand force of 2,000 kips as the approximate maximum load a bulkhead can restrain. This may need to be decreased due to eccentricity of the strands.

Draping has a beneficial effect on shear capacity at the ends of the girder. Thus, if a fabricator proposes a straight strand substitute, make sure the stirrup spacing is still adequate. If stirrup spacing is not adequate, do not approve shop drawings until the fabricator has checked shear capacity and revised as needed.

1.16 Detensioning

The sequence for transfer of stress or the cutting strands shall be in accordance with Article 615.14 of TDOT's "Standard Specifications for Road and Bridge Construction" and shall be shown on the approved shop drawings. The designer should check this. At no time

shall more than 1/6th of the total pre-stressing force be eccentric about the vertical plane of girder.

1.17 Economy and Other Girder Design Tips

Contractors buy their girders from fabricators and contractors pass on that expense to the State. It is in the State's interest to keep fabrication costs down.

1. AASHTO I-Beam Types I, II, and III are the most economic prestressed concrete girders, followed by Bulb-Tees (BT-54, BT-63, and particularly BT-72), and lastly box girders. Box girders are still less expensive than steel girders, but box girder fabrication is more labor intensive and tolerances more difficult to maintain compared to other prestressed girders.

2. If vertical clearance is not an issue a BT-72 may be cheaper than a BT-63 or BT-54. There may be a significant savings in the number of strands required with a BT-72 and not much difference in concrete cost since only a 6" thick web changes height. Note that in recent years the price of strands has increased 50% while in the same period concrete has increased only 20%.

3. Eliminating substructures and going to longer spans is not always the most economic structure, especially for sites where it is hard to ship long girders. For remote bridge sites, it is strongly recommended that the local fabricator be consulted regarding the feasibility of transporting girders.

4. Generally it's a good idea to keep the concrete compressive strengths, f 'ci & f 'c, as low as possible. A low f 'ci speeds up detensioning of strands and removal of the girder from the bed, which speeds up girder production.

5. There is significant jump in the cost of the girders when the release strengths f 'ci > 6000 psi and the final concrete strengths f 'c > 8000 psi. The cost increase is due to such things as the cost of extra cement, admixtures, extended steam curing, and longer curing time before detensioning can occur. Under certain conditions with f 'ci < 6000 psi, the fabricator can strip his forms and detension the next day, thereby optimizing fabrication time.

6. Labor is one of the major costs in girder fabrication. One can reduce labor costs by reducing the number of strands and top longitudinal bars, increasing stirrup spacing, and using single stirrups instead of doubles.

7. BT-72's with high strength concrete have been pushed to 155 feet; however, pushing bulb-tees to their limit may require modifications to the standard shear reinforcement and/or web thickness to deal with high shear at the ends of the girder. In these cases, end shear must be carefully checked. Draping strands may also be required.

8. If possible, set span lengths and girder spacing so the optimized interior and exterior girder design for each span has the same strand pattern and concrete strengths. This allows the fabricator to thread the same strand pattern through multiple girders in a production line. Getting the exact same girder length is not as critical to the fabricator as the same strand pattern and concrete strength.

9. Use the same live load distribution factors, that is, the same LLDF's for moment & shear for both interior and exterior girders in a span, if possible. If that's not possible, use different LLDF's for interior and exterior girders as long as you keep the same strand pattern and initial/final concrete compressive strengths (f 'ci & f 'c) for both interior and exterior girders, clearly labeling LLDF's for interior and exterior girders on the plans.

10. The cost of strands per ton is about the same regardless whether it is 1/2", 1/2" special, 9/16" or 6/10" diameter. Since strands have to be manually threaded through the forms and it takes the same number of men to pull 6/10-inch strands as it does 1/2-inch strands, fewer strands mean less labor and more economy. Where 6/10-inch strands can be used to reduce the number of strands per girder to realize the cost savings, they should be used.

11. Strive for minimal upward deflection of the girder, adding strands until it is up and monitoring upward deflection during the rest of the design. If adding strands causes a compression problem in the bottom of the girder at mid-span, increase concrete strength until that problem disappears. Then check for tension and compression problems elsewhere, using bond breaks or strand relocation in the web to solve problems. Draping is your last resort.

12. Check the Strength 1 ultimate moment provided is greater than the moment required.

13. Design the interior girder first to make sure it's upward deflected, then check the design with the exterior girder. You may need to iterate a little to get a design for both.

14. Keep the deck thickness at 8.25" to minimize the girder's largest dead load with such strategies as adding more transverse reinforcement to the slab, and/or reducing slab span by using girders with wider top flanges, or by adding a girder line to reduce slab span, but check the economy of these decisions, especially adding girder lines. Usually reducing the number of girder lines is the more effective cost-cutting strategy.

1.18 PDF Design Notes

After design is finished a PDF of all design inputs, outputs, sketches, and notes should be made and filed on the P: drive under STRUC_DS in the TDOT-Structures-Design-Calculations folder.

2 Detail Requirements

2.1 Design Data on the Girder/Beam Detail Sheet

In addition to the "F" series girder notes from SMO11, supply the detailers with the following so they can complete the Typical Detail girder sheets in section 5 of this guideline:

___cross section showing strand pattern with bond breaks and reinforcement in the top flange

___ f $'_{ci}$ & f $'_c$

___elevation view showing stirrup spacing and deflection points for any draped strands

___live load distribution factors for moment and shear (LLDF's)

___Composite dead loads DC & DW

___strand note with size, type, area, jacking force

2.2 Strand Note

___Strand size = nominal diameter & uncoated 7-wire strands

___Strand type, usually Low-Relaxation (low-lax), grade 270 ksi (f_{pu}) , see Table 5.4.4.1-1

___Strand cross sectional area, in^2

___Initial jacking force, for Low-Lax use 0.75 A_{strand} f_{pu} , see Table 1 and Table 5.9.3-1

Sample strand note   Prestressing strands: ½" diameter, uncoated 7-wire strands, ASTM grade 270 ksi, Low-Lax, A = 0.153 in^2, pulled to 31.0 kips per strand unless otherwise noted.

2.3 Dead Load Correction Curve on a Superstructure Sheet

___Show the dead load correction curve at quarter span points.

___Use the full concrete compressive strength of slab to calculate deflections, f '$_c$ = 3 ksi.

___ Use DL_{slab} + DC + DW but not DL_{girder} to calculate dead load correction.

___ Use the appropriate "D" series notes from SMO11.

If interior and exterior girders have the same design but have different dead load deflections, just use the interior girder deflections for the dead load correction curve.

3 Repair of Damaged Girders

Any damage to girders during fabrication, shipment, or erection shall be repaired according to the current edition of the "Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products". When damage occurs, a detailed repair procedure shall be submitted to TDOT Materials and Tests Division, and Division of Structures for approval as soon as possible. The repair procedure submittal shall include detailed pictures of the damaged area, specifications of the repair material, and which standard repair procedure will be used. The reviewing Divisions need adequate time to review the procedure before approval to proceed can be given.

4 Rating Girders

Conspan has the ability to rate the prestressed bridges by AASHTO LRFD Bridge Design Specifications. Designers **shall** rate all new prestressed bridge designs with this program.

Additional information regarding the use of Conspan rating program:

1. The Designer must enter the stirrup and negative moment steel information on the beam tab to get the rating results.

2. Use **HL-93 Design Load** under Design Load Rating. Use **H15** and **HS20** under Legal Load Rating. We don't usually have a permit load.

3. Check **all** under Rating Options.

4. For the factors on the right side of the rating sheets, if the designer clicks on the blue headings, an explanation screen will come up. The system factor depends on the number of beams and beam spacing. Use a condition factor of **1**, since the bridge is new. Use the projected ADTT to calculate the Strength I factor for the legal live load. Use **0.33** for the Dynamic Load Factor. Use **1.5** load factor for DW since it is not a measured thickness. Use the default factors for the Design Live Loads.

5. If beams are similar (i.e. do not vary much across the span) then it would be sufficient enough to provide a rating for one (1) typical interior beam and one (1) typical exterior beam. On the other hand, if the bridge beams vary a lot (i.e. because the bridge is sharply skewed, flared, curved, etc.) then it might be necessary to provide a rating for each beam. It is preferred to just have a rating for a typical interior and typical exterior beam in most cases.

6. After the rating output file is created, convert it to an Adobe PDF file using Acrobat. A folder has been created on the STRUC_DS drive named "Conspan Rating Files". The files should be stored in this folder. The name of the file should be the Bridge ID number. Also, save the Conspan input file using the Bridge ID number as the file name. Send a brief e-mail to Terry Leatherwood telling him that new files have been stored, along with the file names and bridge name.

The following is a sample screen shot of the Conspan Rating Program.
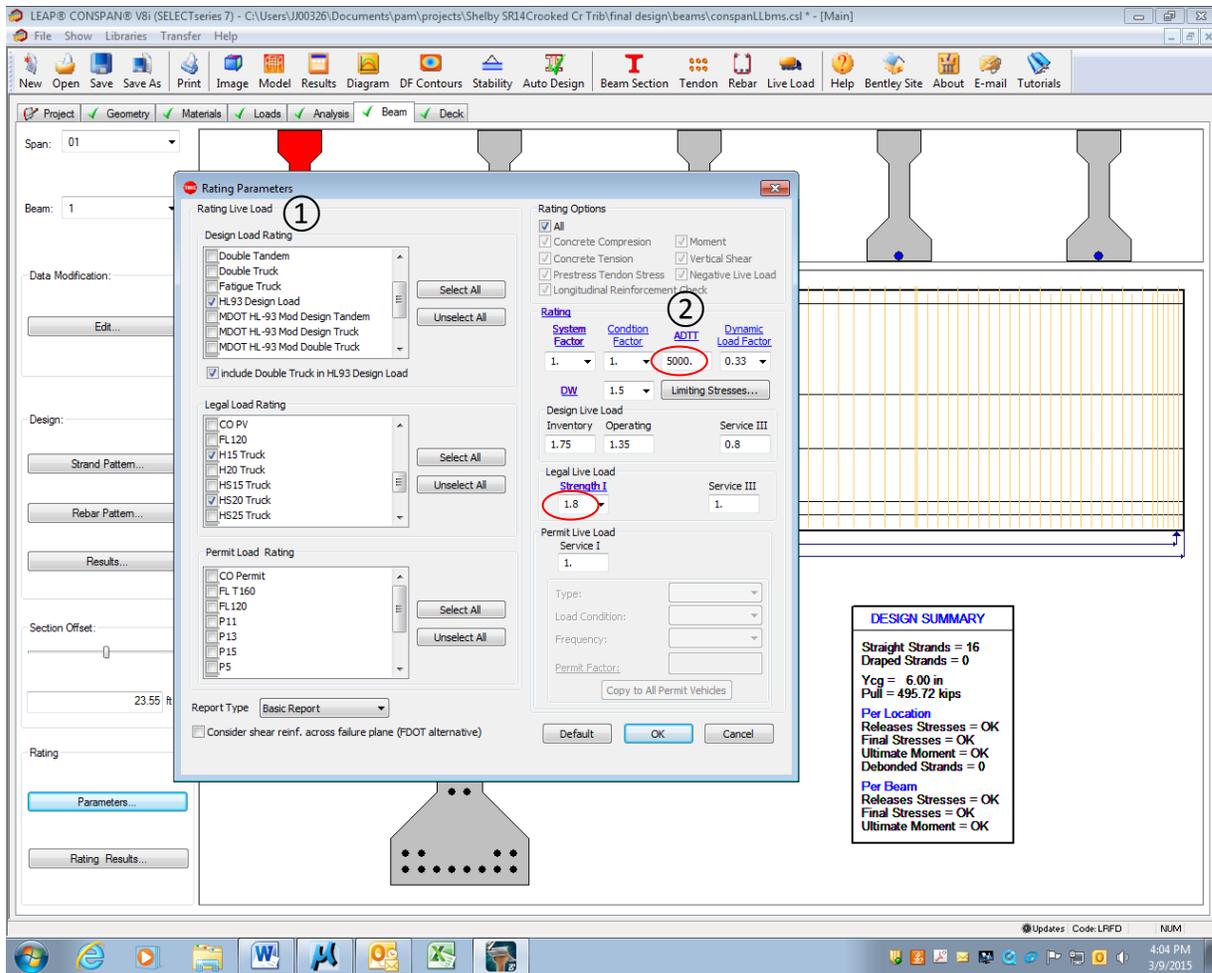
Figure 6 CONSPAN Rating Parameters

①Use design load HL93 and legal load H15 and HS20.

②Use 5000 for ADTT so the legal live load factor is 1.8.

5   Girder Properties

Figures 5.1 through 5.3 provide a summary of girder properties. The Virginia Bulb-Tee Girder is an alternate beam type which may be used in low vertical clearance conditions.

54"-72" Bulb-Tee Section Properties

54"-72" Bulb-Tee Section

Girder Type	Beam Height (in.)	Web Height (in.)	Section Area (sq in.)	Center of Gravity (in.)	Moment of Inertia (in⁴)	Weight (lb/ft)	
BT-54	54"	36"	659	27.63	268,077	686	
BT-63	63"	45"	713	32.12	392,638	743	
BT-72	72"	54"	767	36.60	543,894	799	
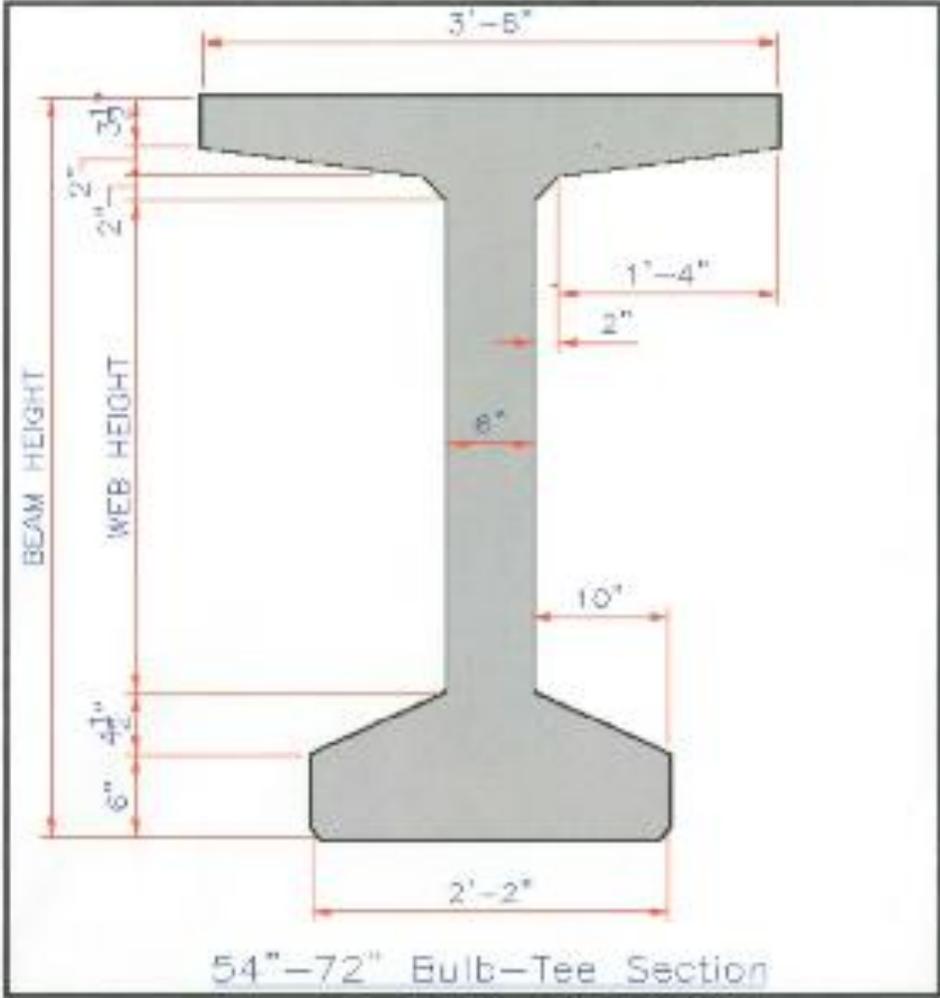
Bulb-Tee Girder Section Properties

Figure 7 Bulb-Tee Properties

AASHTO Girder Section Properties

AASHTO GIRDERS
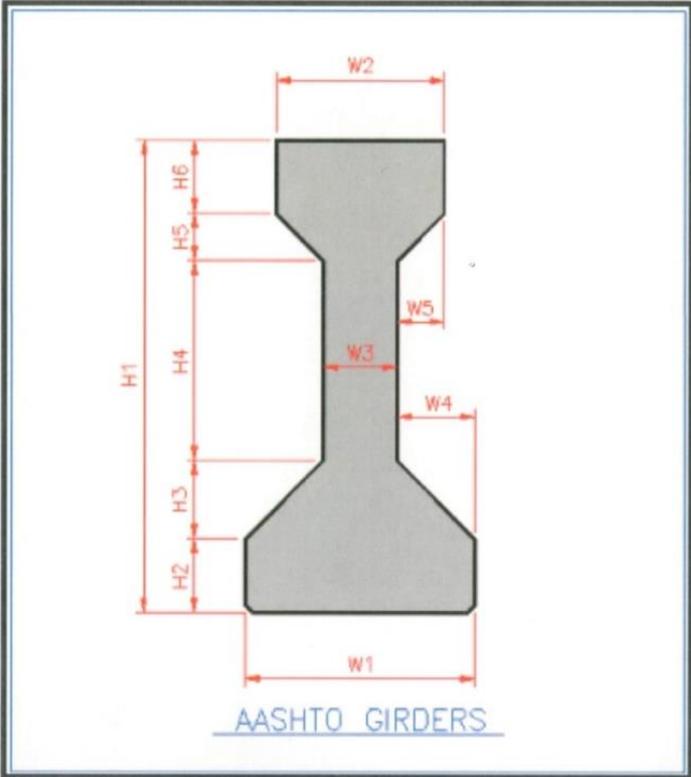
Girder Dimensions										
Type	H1	H2	H3	H4	H5	H6	W1	W2	W3	W4	W5
I	28"	5"	5"	11"	3"	4"	16"	12"	6"	5"	3"
II	36"	6"	6"	15"	3"	6"	18"	12"	6"	6"	3"
III	45"	7"	7.5"	19"	4.5"	7"	22"	16"	7"	7.5"	4.5"
IV	54"	8"	9"	23"	6"	8"	26"	20"	8"	9"	6"

Girder Section Properties					
Beam Type	Section Area (sq inch)	Center of Gravity (inches)	Moment of Inertia (inch^4)	Weight (lb/ft)	
I	276	12.59	22,750	287	
II	369	15.83	50,980	384	
III	560	20.27	125,390	583	
IV	789	24.73	260,730	822	

Figure 8   I-Girder Properties

Figure 5.2   I-Girder Properties

6 CONSPAN Input Parameters

Figures 6.1 through 6.14 are screen shots of typical required CONSPAN input fields. Discuss any deviations from these settings with your supervisor.
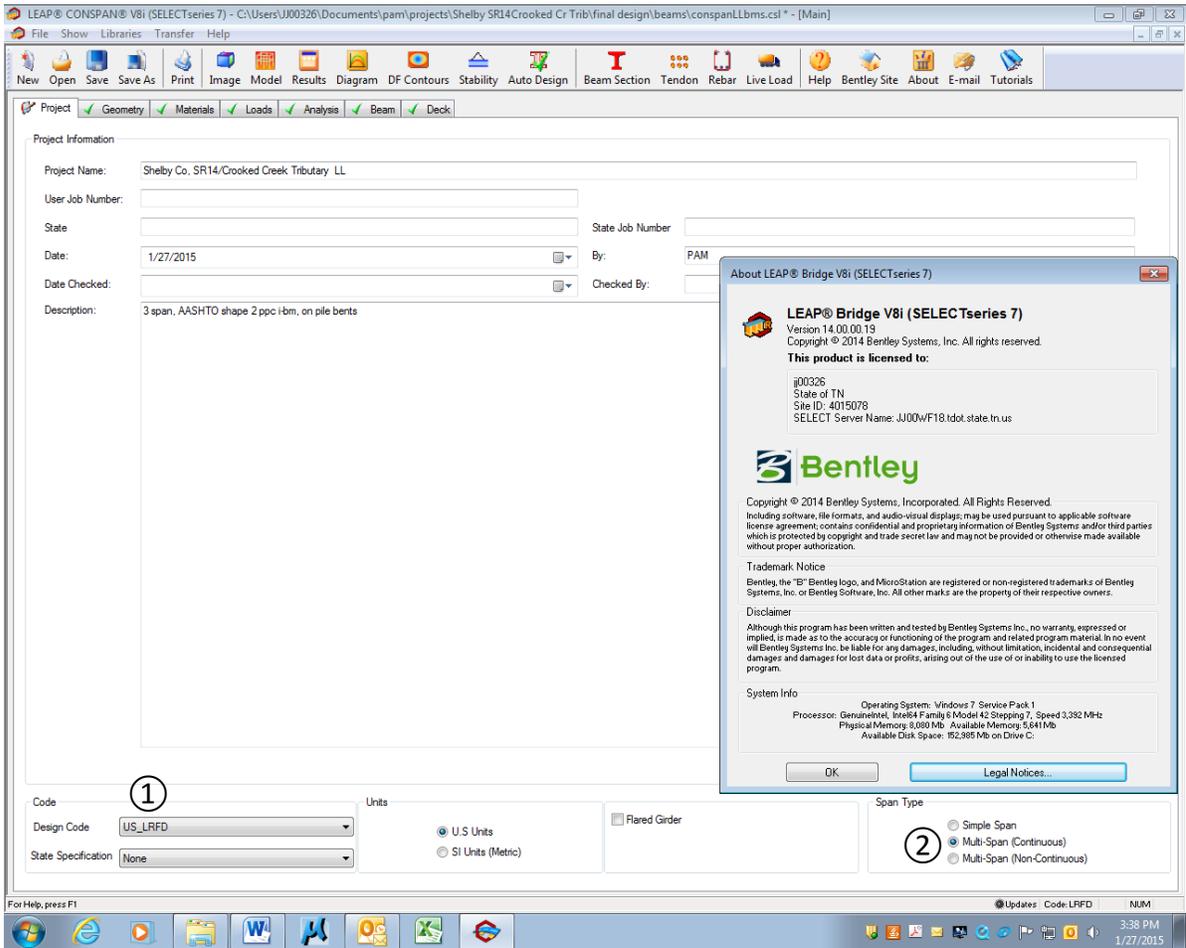
Figure 9 CONSPAN Project

① Always design LRFD.

②Continuity is a valid assumption for a TDOT multi-span bridge. Refer to UT continuity study of TDOT bridges.

①Conspan quirk for version 14.00.00.19 as of 1/22/2015, the parapets will not show up in the deck drawing unless you tinker with the curb data. For example, changing left curb width from 1.21 to 1.210 makes the parapets appear and remain even when deleting the zero.
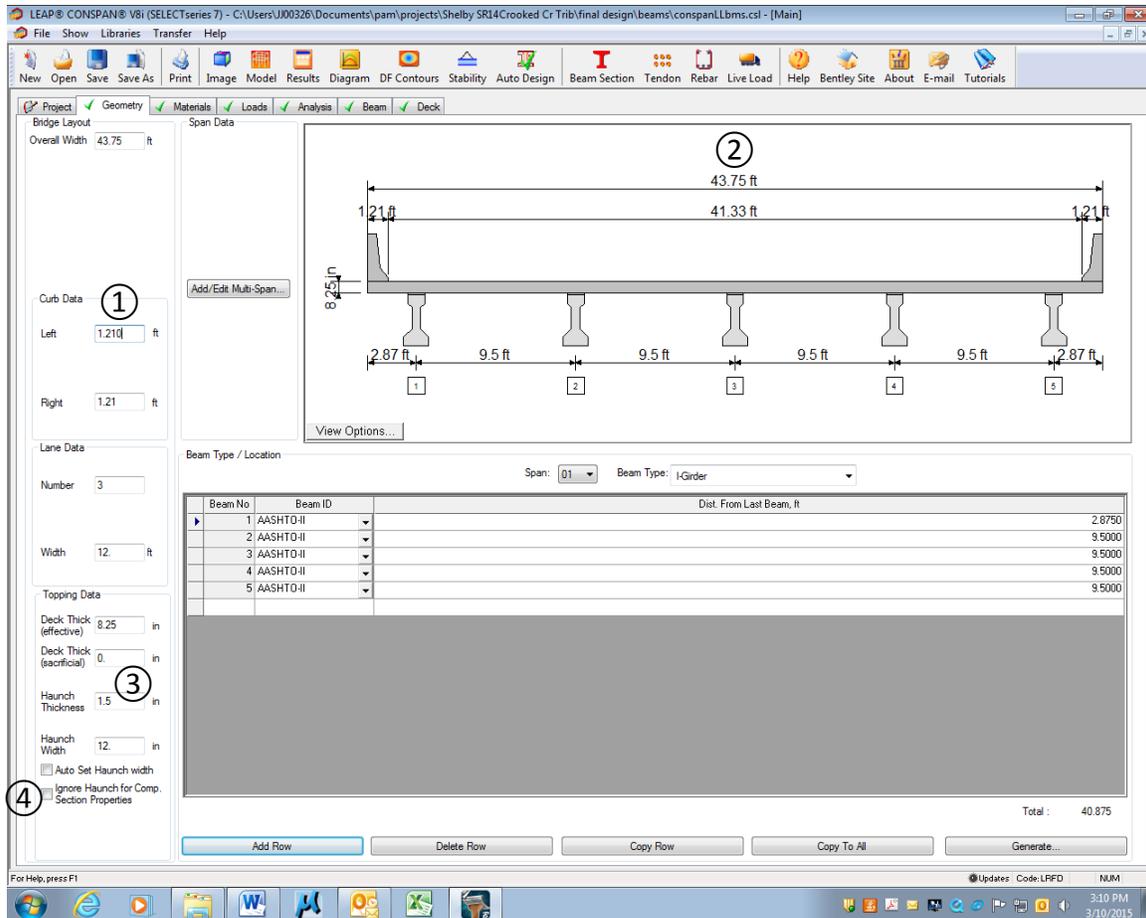
Figure 10 CONSPAN Geometry

② Our maximum deck width available to traffic is measured between the top inside corners of the parapets. Your Conspan width between "curbs or barriers" can be the distance from base of parapet to base of parapet as long as it does not reduce the maximum # of 12' traffic lanes the deck can carry.

③ Include a "haunch" (filler) depth. Some designers input a nominal filler depth like 1.5" for i-girders and input any remaining filler depth as a precast DC line load.

④ Bulb-tees in particular can gain a lot of camber over time. By mid-span the filler may become negligible. Use your better judgment, but if your bulb-tee is already highly cambered, it might be best to ignore the filler for composite bulb-tee section properties.

Span #	Pier to Pier, ft	Precast Length, ft	Bearing to Bearing, ft	Pier CL Precast, ft	Release Span, ft	Skew Angle Start, deg	Skew Angle End, deg	Post-tensioned	
1	50.7100	47.0900	46.3400	2.870000	47.0900	45.00	45.00		no
2	48.5900	47.0900	46.5900	0.750000	47.0900	45.00	45.00		no
3	50.7100	47.0900	46.3400	0.750000	47.0900	45.00	45.00		no
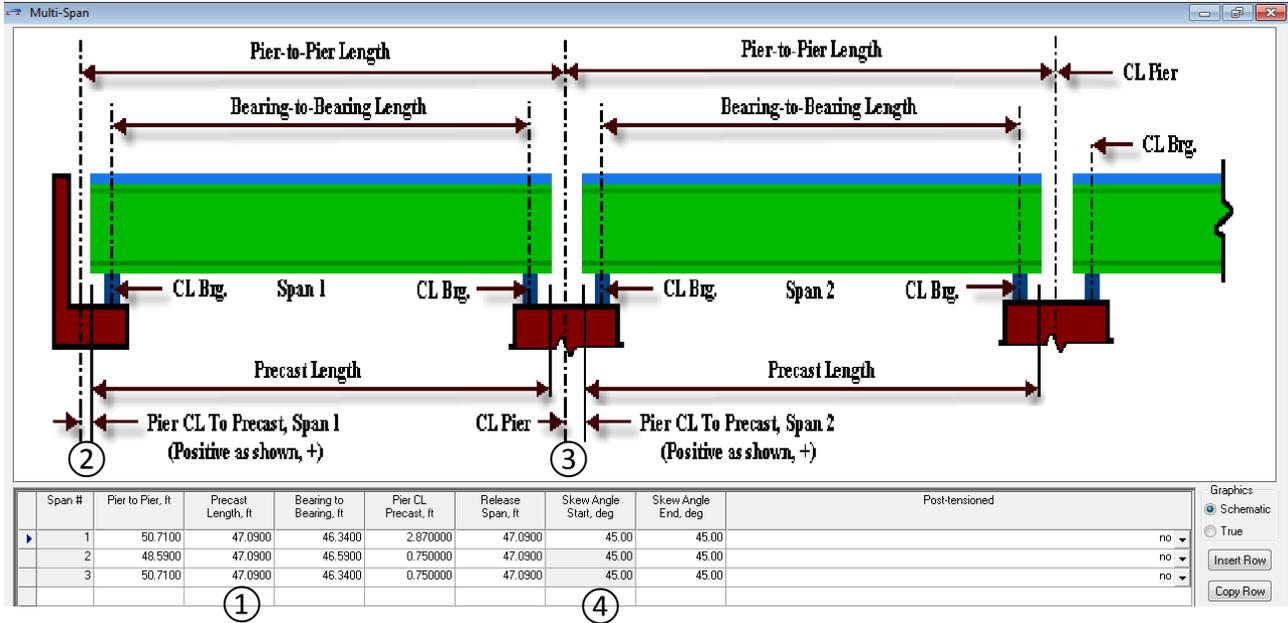
Figure 11 CONSPAN Multi-Span Sketch

Add span data by going to "Geometry" then "Add/Edit Multi-span".

① Girder length on bridge plans will be rounded up to the nearest inch.

② For skewed bridges with AASHTO i-girders and bulb-tees, Pier CL to Precast distance at the abutment is measured from the endwall/roadway bracket junction, TDOT 's begin bridge, to the beginning of girder, which for a 1.5' endwall thickness and flange width "b", distance = 1.5'/sinϴ + 0.5b/tanϴ. For AASHTO i-girders "b" is the bottom flange width. For bulb-tees it may be the top or bottom flange width depending on the skew angle ϴ. The top flange of a bulb-tee may need to be clipped to keep the abutment beam width from increasing too much but the clip never gets any closer than the corner of the bottom flange.

Box beams can be fabricated with skewed ends at β ≥ 60°. For β from 60° to 90°, begin bridge to precast is 1.5'/sinϴ. For skews ϴ < 60° and box width "b", use 1.5'/sin ϴ + 0.5b/tanϴ - 0.5b/tan 60°.

③ I-girders and bulb-tees have their ends squared off so there are no skewed dimensions along CL girder at the piers.

④ Skew angle in Conspan is defined as the angle from CL substructure to a line perpendicular to CL survey.

Figure 12 Plan View of Skewed Box Beam & Endwall
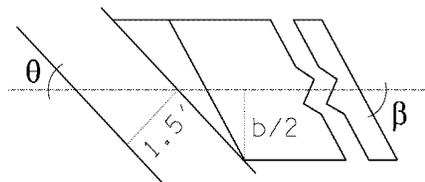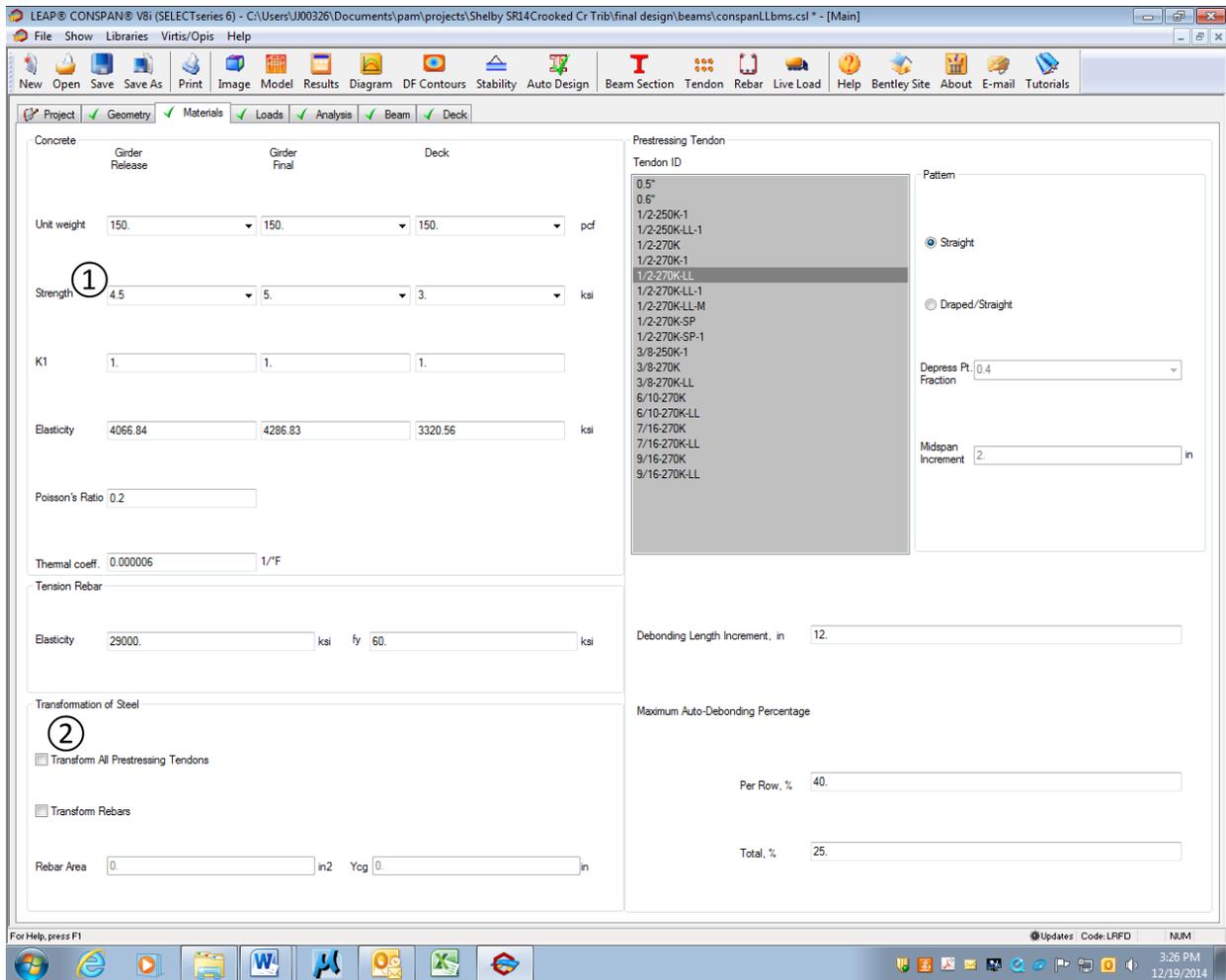
Figure 13 CONSPAN Materials

① Input f '$_{ci\ girder}$ ≥ 4 ksi, f '$_{c\ girder}$ ≥ 5 ksi, f '$_{c\ slab}$ = 3 ksi (4 ksi with approval when girder is difficult to design).

② Design your girder without transformation of strands or "rebars". As of version 14.00.00.19 December 2014, Conspan does not report changes in section properties or stresses when transforming strands or rebars and only girder deflections change; the girder with transformed strands or rebars does not deflect upward as much which seems correct, but let the designer beware that we cannot tell if Conspan transforms correctly. Note also that Conspan transforms all strands which means the results are accurate only in areas where bond is <u>not</u> broken.
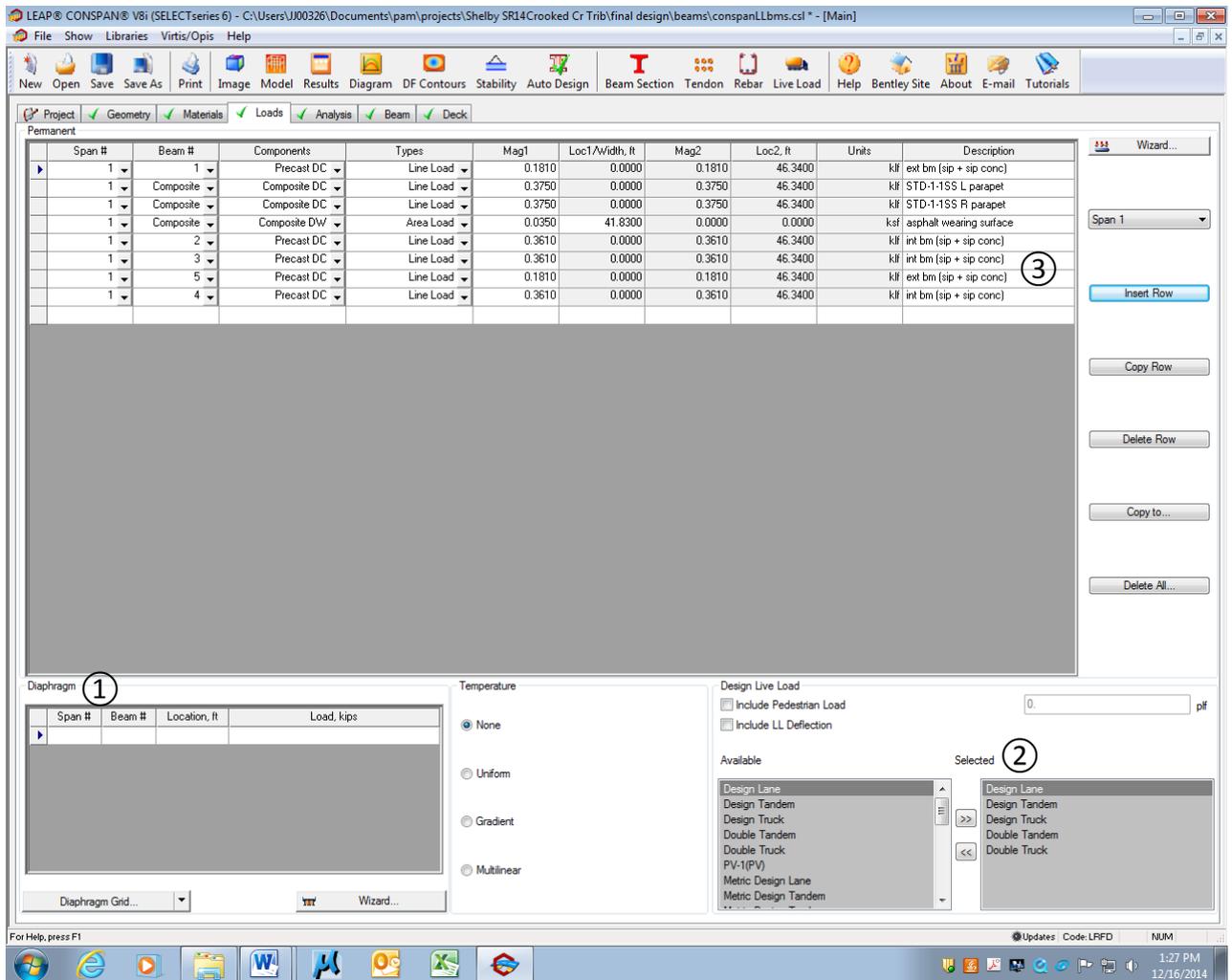
Figure 14 CONSPAN Loads

① If diaphragms are required, assume the heavier concrete diaphragm. If you have problems designing the girder, you can use the lighter steel cross frame for bulb-tees or steel channel for I-beams, but you will need to restrict the contractor to those lighter steel diaphragms with a note on the plans.

② Select design lane, truck, tandem, double truck, and double tandem (STRAHNET routes require double tandem).

③ Assume SIP steel forms and their trough concrete add a load equivalent to 1" added deck thickness.
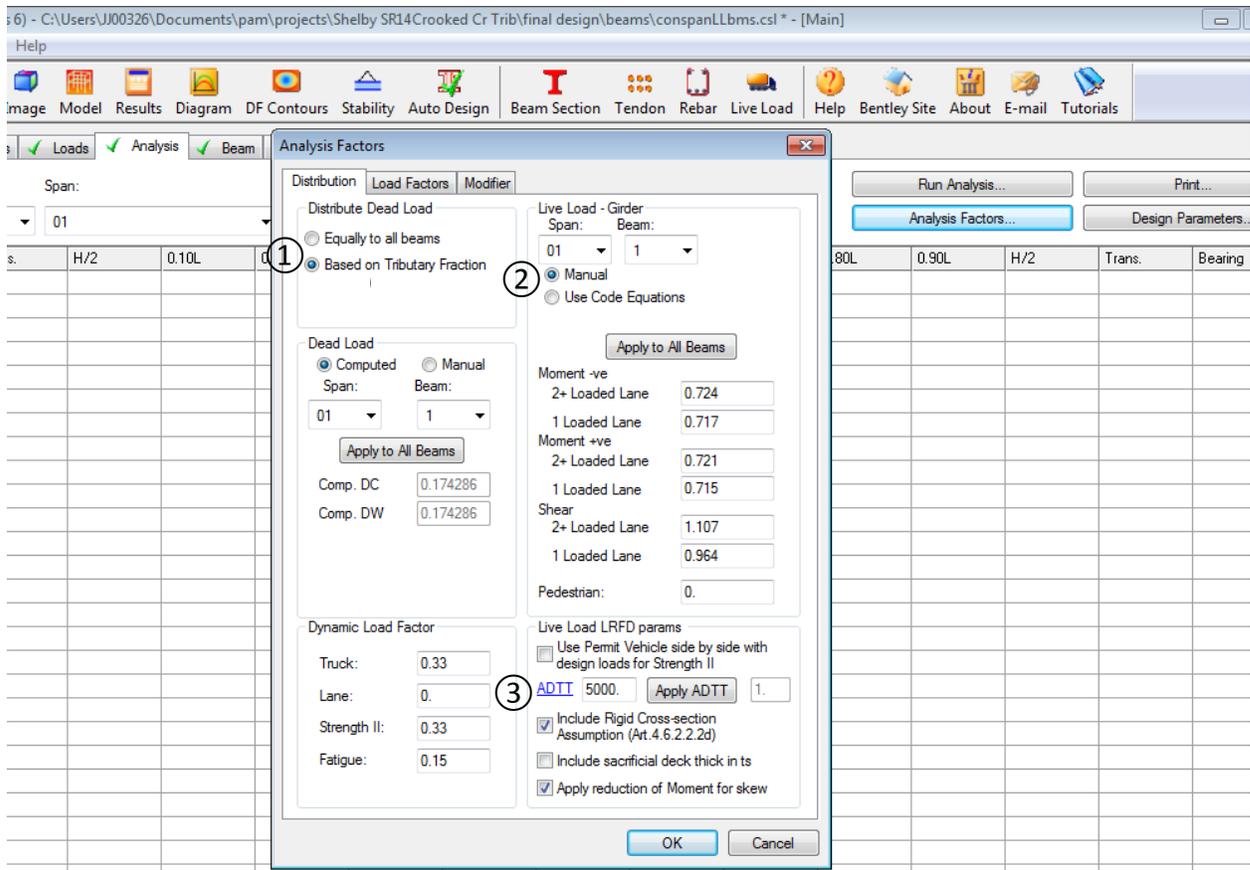
Figure 15 CONSPAN Analysis>Analysis Factors>Distribution

①Using the tributary width decreases the DL on the exterior bm which usually carries more LL than the interior bm, thereby helping to balance loads between interior and exterior bms and helping achieve the same design for both bms.

②"Use Code Equations" to generate live load distribution factors (LLDFs), then input the largest of these into "Manual" and see if the interior and exterior girders can be designed with the same LLDFs, ideally one LLDF for the moments and one LLDF for the shears. If the girder won't design without increasing f'c , start using more LLDFs: 3 LLDFs for each girder (1 for M-, 1 for M+, & 1 for V), up to 6 LLDFs for each (2 for M-, 2 for M+, & 2 for V).

③Use 5000 ADTT which applies 100% of the specified force effect and yields an infinite fatigue life. Don't allow the faded out value next to "Apply ADTT" to be less than 1.0 so the multi-presence factor is not reduced.

① No changes required for "Load Factors" and "Modifier"; use default values.

② Strength II Limit State is for permit vehicles which we normally don't use.
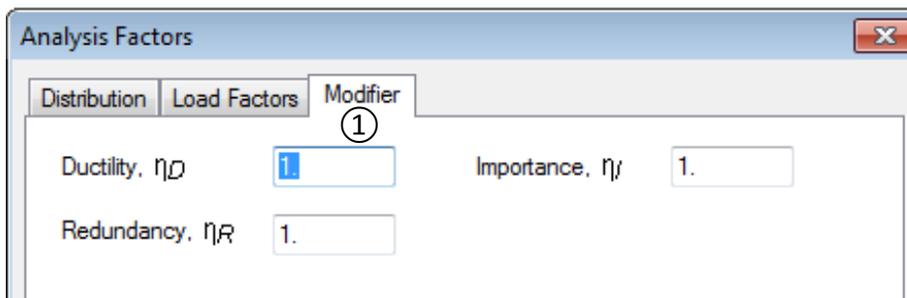
Figure 16 CONSPAN Analysis>Analysis Factors>Load Factors & Modifier
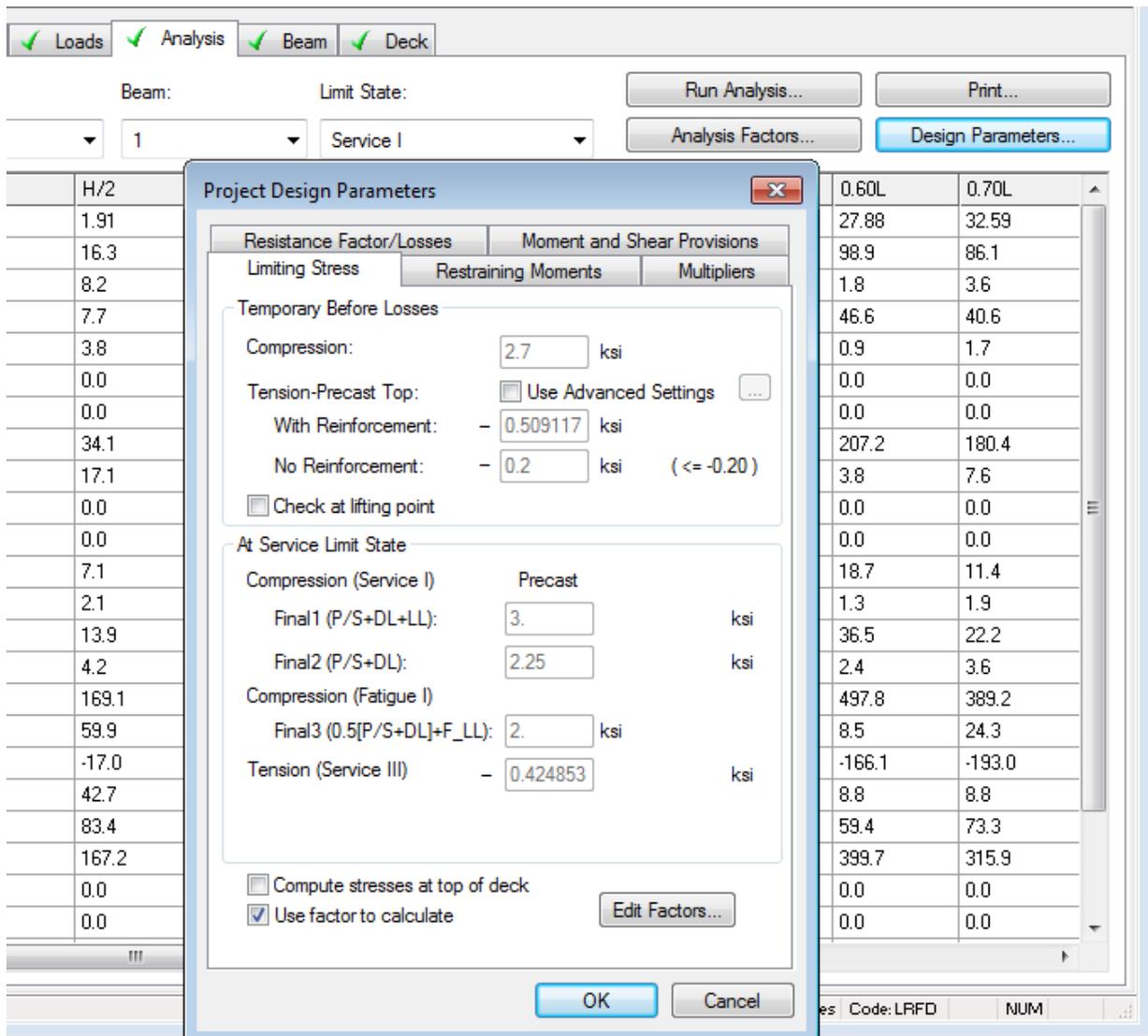
Figure 17 CONSPAN Analysis>Project Design Parameters>Limiting Stress

Use default values for "Limiting Stress" tab.
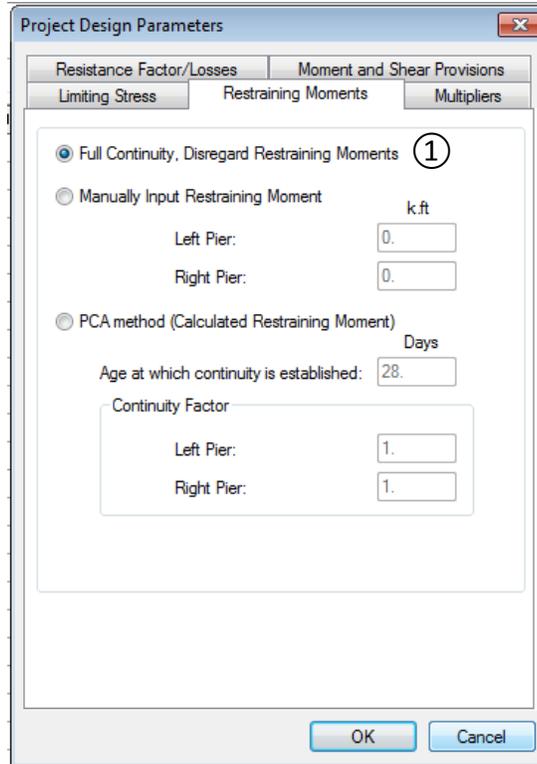
① Designing for full continuity and no restraining moments requires designer to note in the plans that girders must be at least 90 days old before deck and diaphragms can be placed (5.14.1.4.4-Age of Girder When Continuity Is Established).

Figure 18 CONSPAN Anaylsis>Project Design Parameters>Restraining Moments
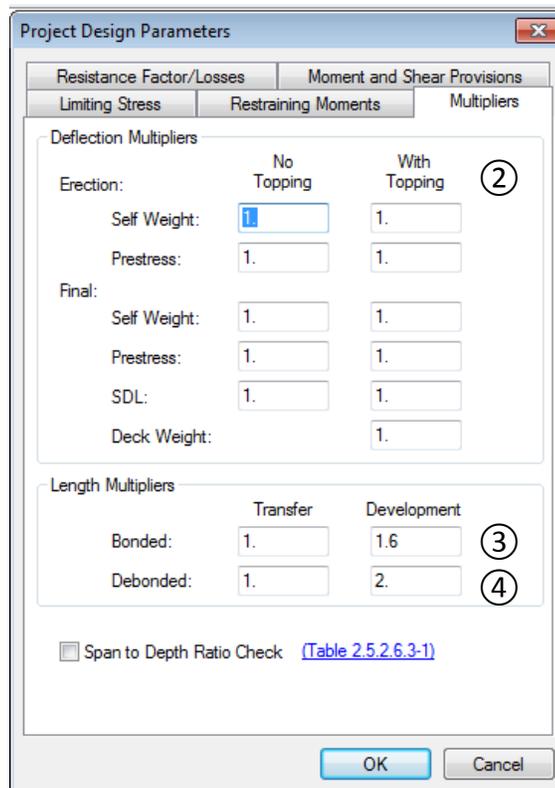
② Office policy.

③ 5.11.4.2

④ 5.11.4.3

Figure 19 CONSPAN Analysis>Project Design Parameters>Multipliers
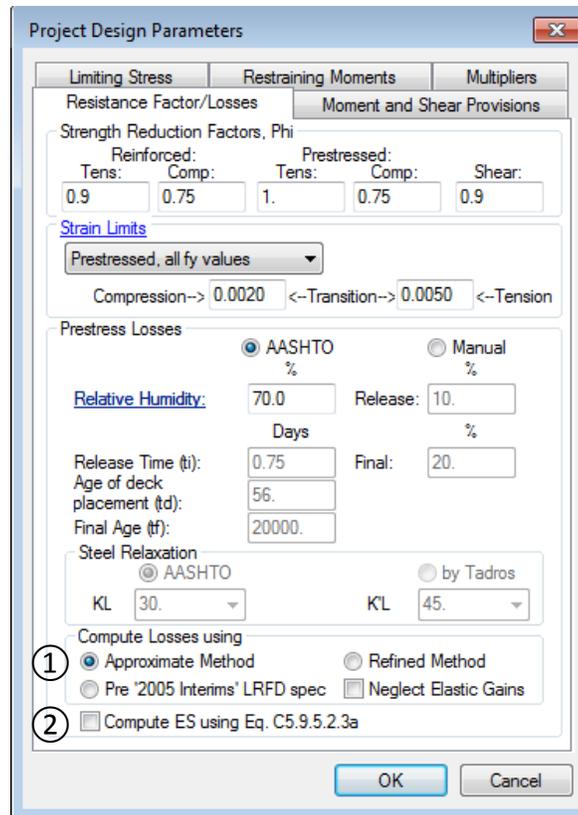
① 5.9.5.3 Approximate Estimate of Time-Dependent Losses is for standard precast, pretensioned members subject to normal loading and environmental conditions.

② Leave blank. Stay with ES computed by Spec. rather than Commentary.

Figure 20 CONSPAN Analysis>Proj Des Params>Resistance Factors/Losses
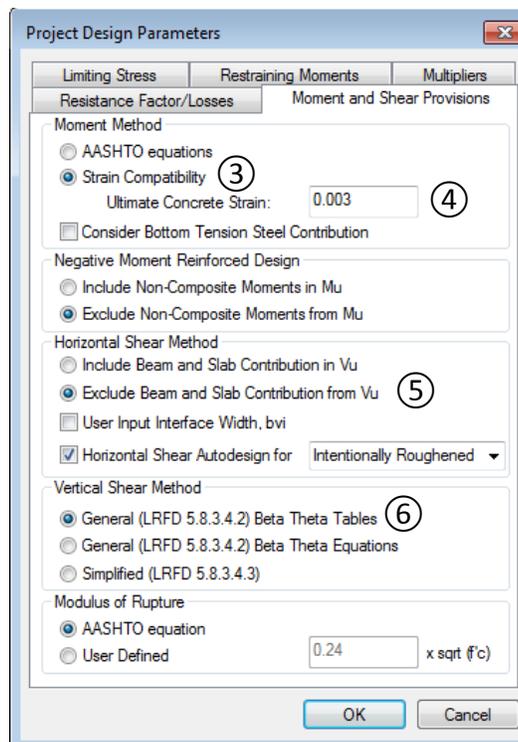
③ More theoretically correct.

④ Use ultimate concrete compressive strain of 0.003, 5.7.2.1.

⑤ Exclude! The slab in Conspan is a non-composite load on the beam and therefore does not contribute to Vu.

⑥ Appendix B5 permits tables but some designers prefer the beta theta equations.

Figure 21 CONSPAN Analysis>Proj Des Params>Moments & Shear Provisions
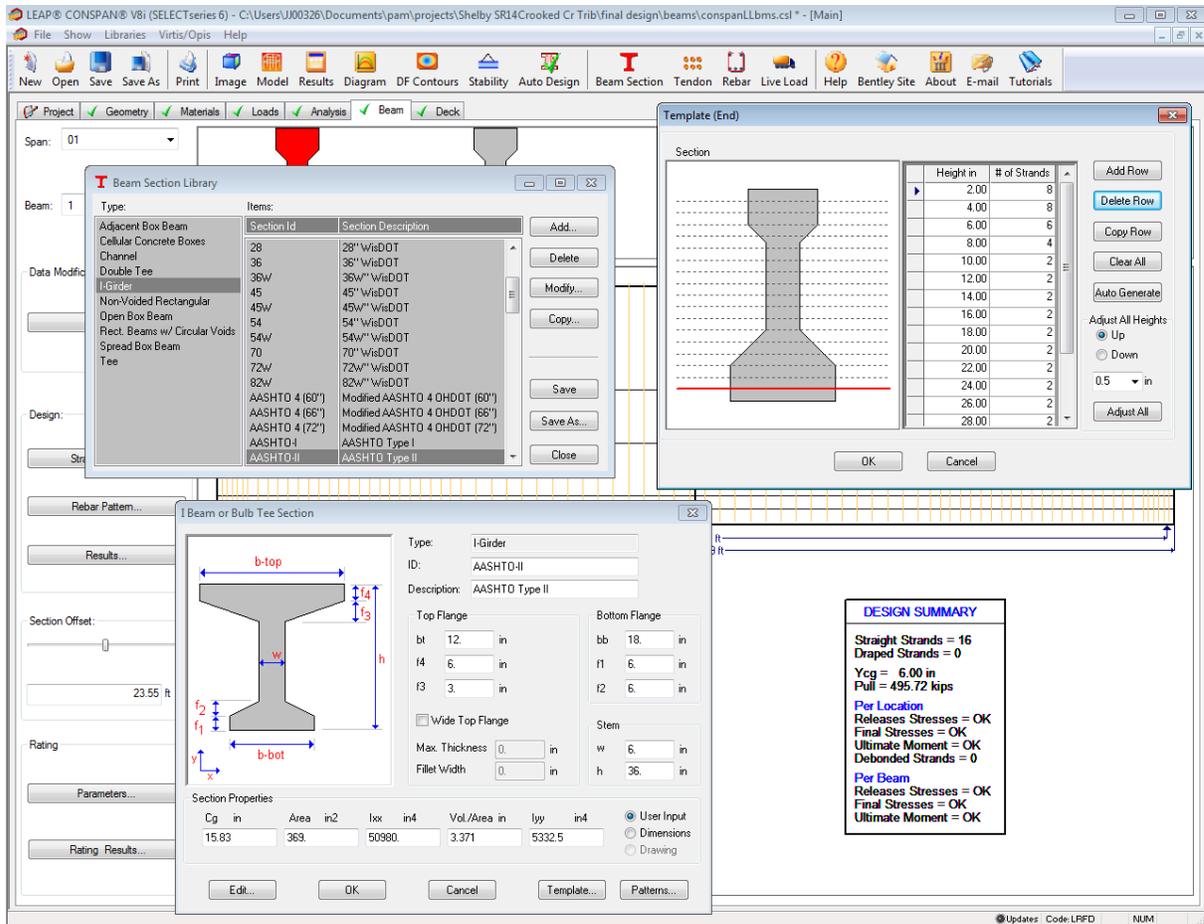
Figure 22 CONSPAN Beam

After running Analysis, check that strands rows are vertically spaced 2 inches on center, beginning 2 inches from the bottom of girder.

Click on Beam Section > Type > Section id > Modify > Template.
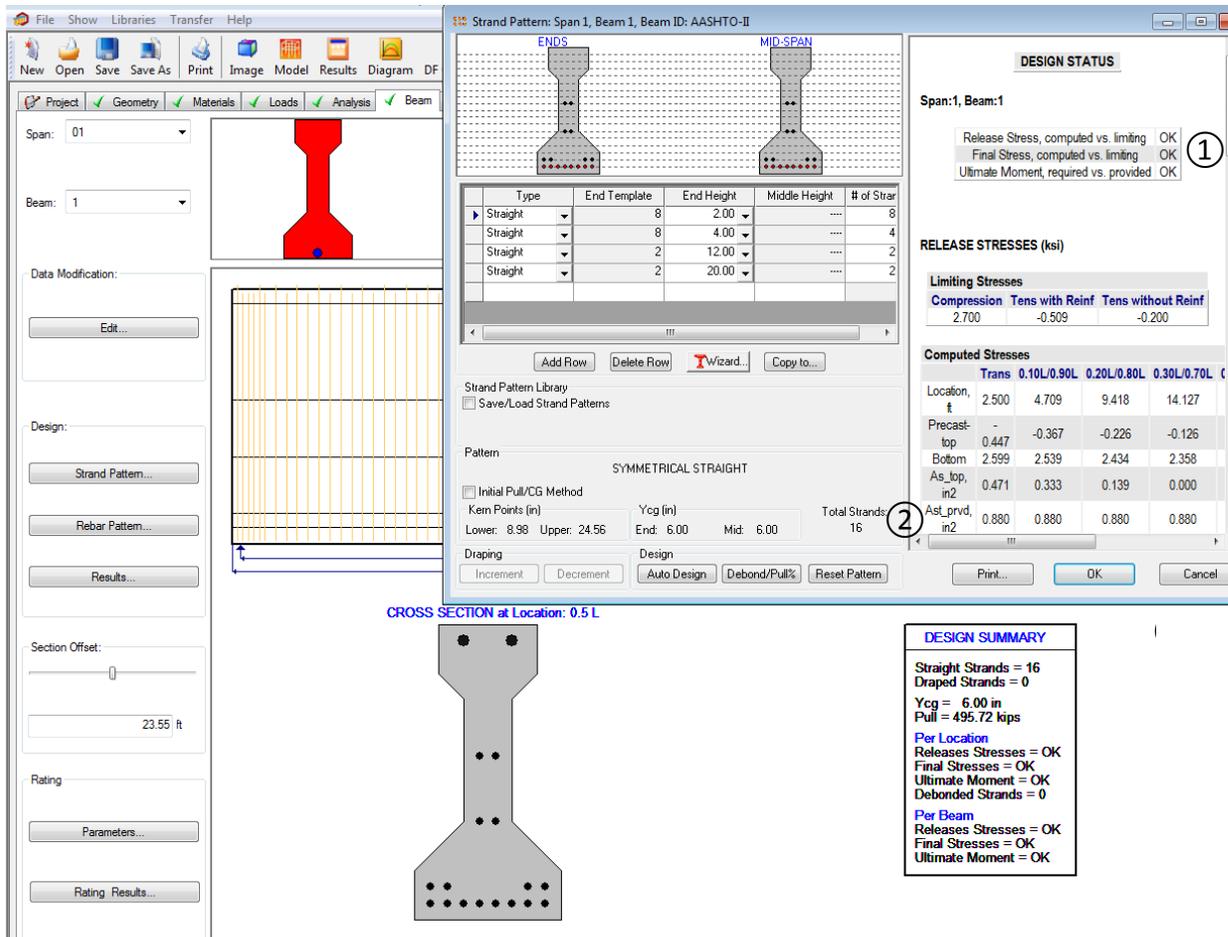
Figure 23 CONSPAN Beam>Strand Pattern

Start with Auto Design and fine tune with Debond/Pull%. Use "Design Status" to check adequacy of ① release and final stresses, ultimate moment, ② longitudinal steel in the top flange, and upward deflection of the girder at the very end of "Design Status".

Moving strands up or down the web can help reduce the amount of debonding. This design did not require bond breaks or draping of strands.
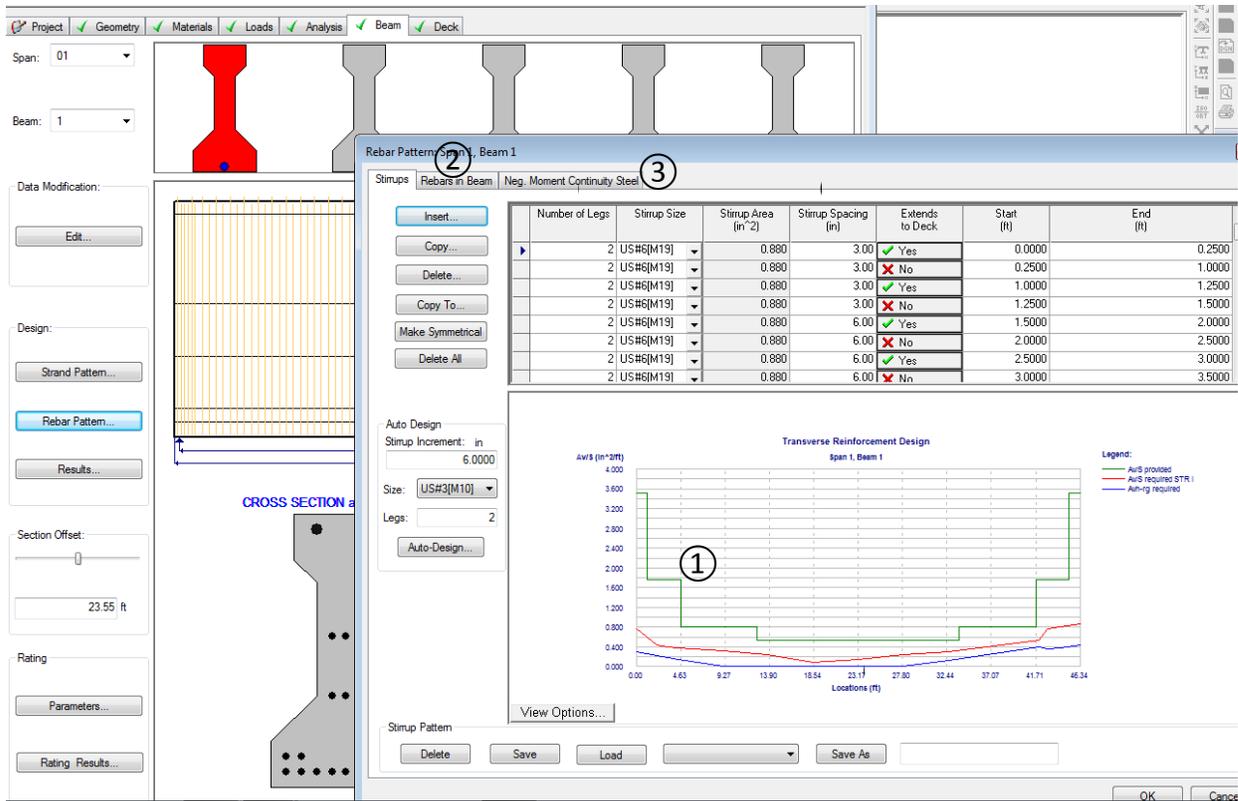
Figure 24 CONSPAN Beam>Rebar Pattern>Stirrups

① Stirrup reinforcement provided (green line) needs to be above the required and symmetrically placed.

② "Rebars in Beam" is where longitudinal reinforcement in the top flange is placed.

③ Use "Neg. Moment Continuity Steel" tab to place longitudinal deck reinforcement per effective flange width (4.6.2.6). This must be entered before the girders are rated.
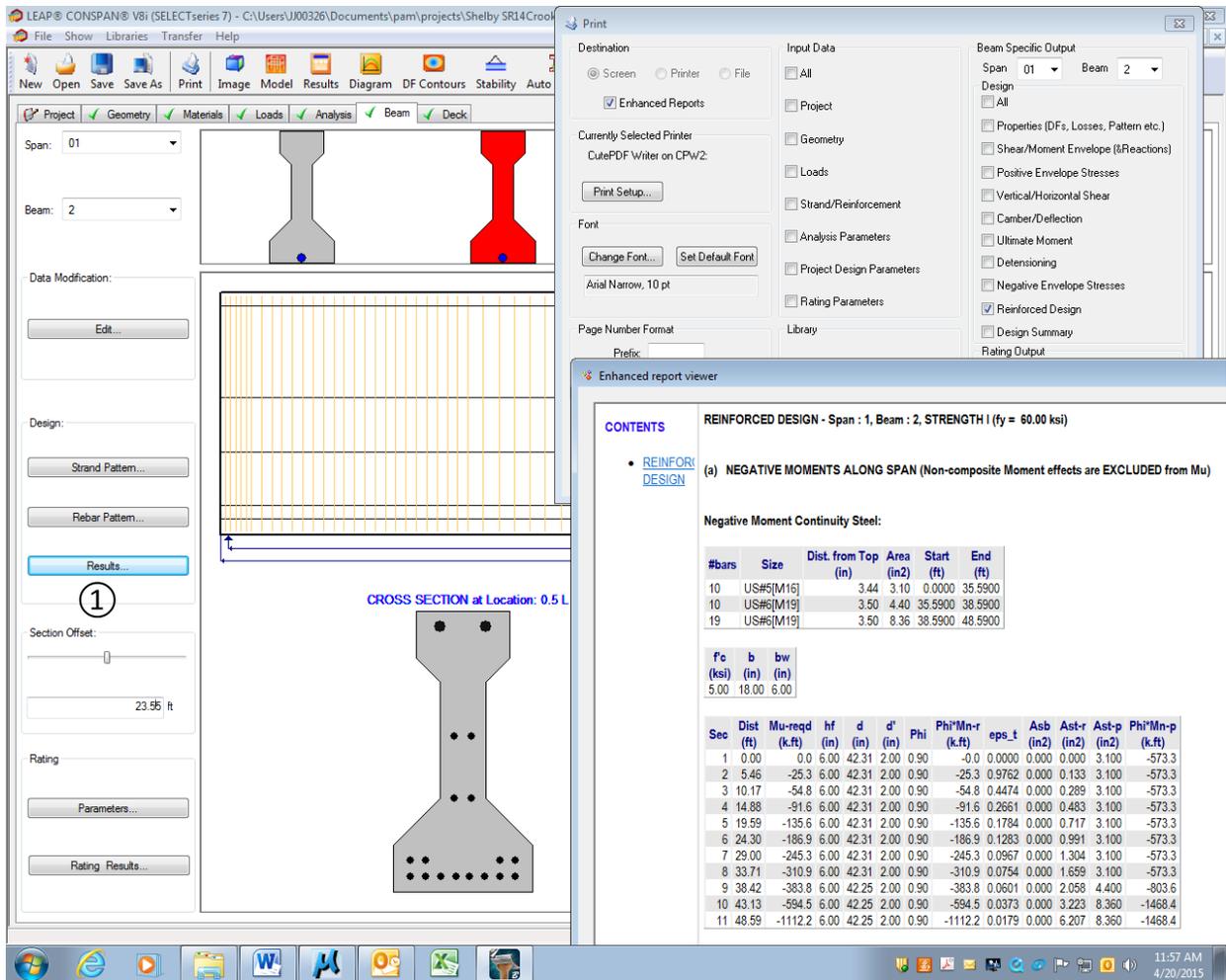
Figure 25 CONSPAN Beam>Results>Reinforced Design

① From the "Beam" tab use the "Results" button to view "Reinforced Design", which gives the analysis of the assumed negative moment continuity steel in top mat, which you keyed into the "Negative Moment Continuity Steel" tab in the "Rebar Pattern" screen.
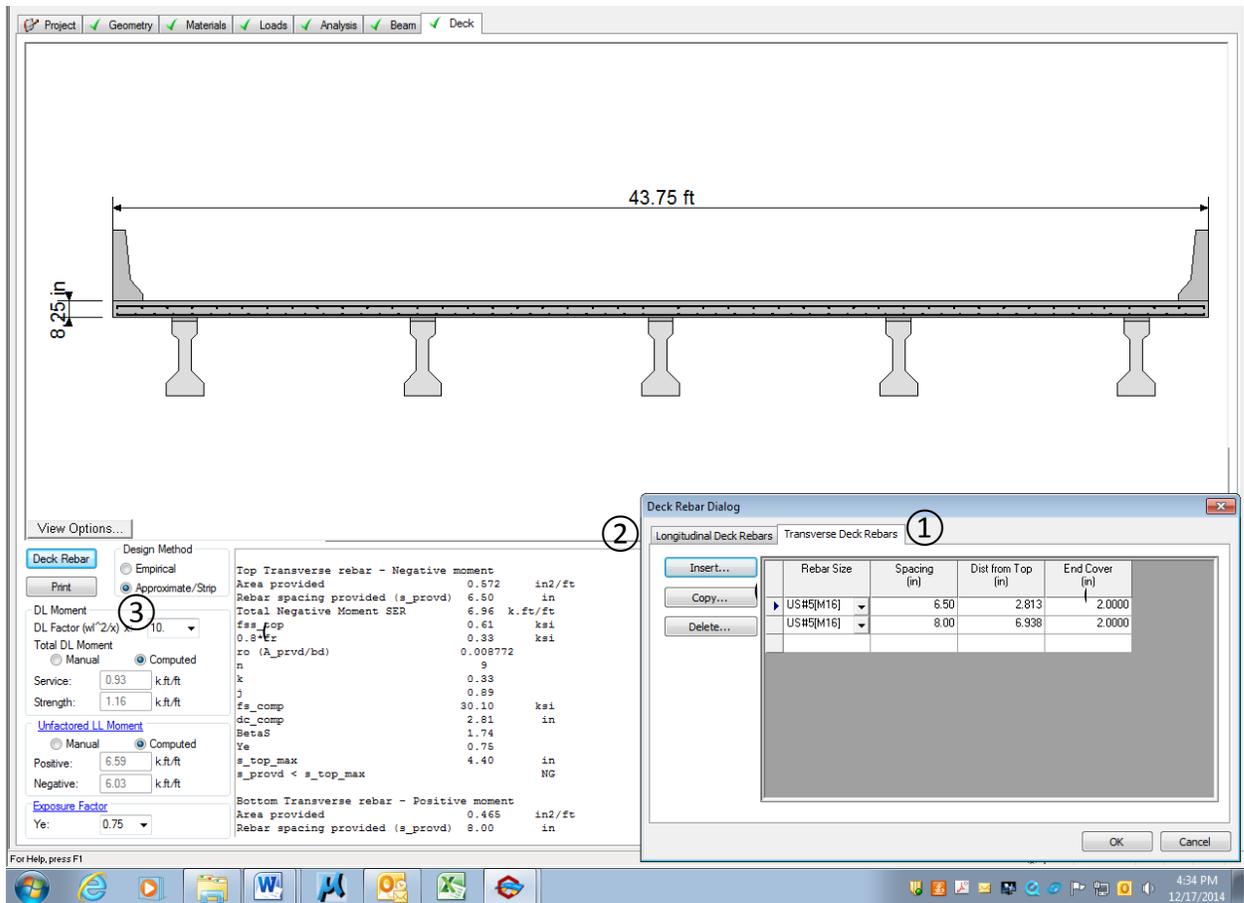
Figure 26 CONSPAN Deck

① **Conspan does <u>not</u> consider the cantilever in its slab design, so the designer must check the cantilever to see if it controls.** Conspan also doesn't count on compression reinforcement and therefore underestimates the deck's flexural capacity. Make use of compression steel in the slab design and reduce top transverse steel by 30% or more.

② "Longitudinal Deck Rebars" are not the longitudinal negative moment continuity bars. Here only top longitudinal shrinkage and temperature steel and bottom longitudinal distribution steel are calculated.

③ Use the Approximate/Strip method (4.6.2). The Empirical method requires a fully cast-in-place slab (9.7.2.4), and since we allow the option of precast, prestressed stay-in-place deck panels, we cannot guarantee a fully cast-in-place slab.

7 Typical Details for Bulb-Tees, I-Girders, and Box Beams

Attached are "inserted" pictures (jpegs, tiffs, or pngs) made from pdfs. For better resolution girder details, use Adobe Acrobat to make pdf of pages 1-32 and combine with girder pdfs.

Figure 27 Bulb-Tee Details

Figure 28 I-Girder Details

Figure 29 Box Beam Details

NOTES AND DETAILS FOR GIRDER DRAWING IN CONTRACT PLANS

ℓ INTERMEDIATE DIAPHRAGM (SEE STD-14-1 FOR DETAILS)
AT MIDPOINT FOR SPANS > 40 FT.
AT 1/3 POINTS FOR SPANS > 80 FT.

PRECAST LENGTH Ⓑ

ℓ INSERT FOR TEMPORARY BRACING NEAR MID-SPAN

SYMMETRY ABOUT MIDPOINT

STRAND GROUP I

3" MIN. TO TOP STRAND

NO. STRANDS AND HEIGHT DETERMINED BY DESIGN.

STRAND GROUP II

1" MINIMUM, 3" MAXIMUM FOR STRANDS NOT BENT

HOLD DOWN POINT DETERMINED BY DESIGN

PROJECT 8½" AND BEND EVERY OTHER FULLY BONDED STRAND IN BOTTOM ROW (SEE STD-14-1)

ℓ INSERTS FOR TEMPORARY CROSS BRACING (SEE STD-14-1)

ELEVATION
(SHOWING STRAND DEFLECTION ARRANGEMENT)

BARS A400

5 SPA. @ 6" = 2'-6"   5 SPA. @ 6" = 2'-6"

SINGLE PROJECTING BARS H--- MAX. SPACING 24"
SINGLE NON-PROJECTING BARS H--- (WHEN REQUIRED)

4 spa. @ 3" = 1'-0"

DOUBLE PROJECTING BARS H600

DOUBLE NON-PROJECTING BARS H601

MIN. SPLICE (TYP.)

BARS A--- S/W
BARS A---

DOUBLE PROJECTING BARS H600

DOUBLE NON-PROJECTING BARS H601

1½" RADIUS FILLET (TYP. EACH END OF GIRDER)

BARS HA300
14 SPA. @ 6" = 7'-0" BT-54
16 SPA. @ 6" = 8'-0" BT-63
18 SPA. @ 6" = 9'-0" BT-72

ELEVATION
(SHOWING STIRRUP BAR ARRANGEMENT)

Ⓐ FABRICATOR SHALL ALTERNATE ORIENTATION OF SINGLE STIRRUPS.

Ⓑ PRECAST LENGTH ADJUSTED FOR ROADWAY GRADE.

STRAND GROUP I (MAX. OF 10 STRANDS)

STRAND GROUP II

¾" CHAMFER (TYP.)

2" 11 SPA. @ 1'-10 2"
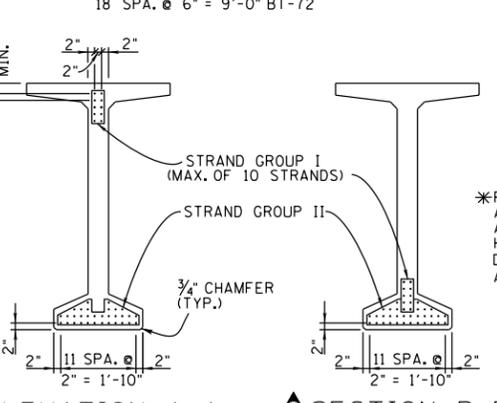
ELEVATION A-A

SECTION B-B

SPACING DET. BY DESIGN

BARS A400

EVEN NO. OF BARS A--- (TYP.)
BARS A--- (TYP.) SIZE DETERMINED BY DESIGN.

2¼" CL. (TYP.)

ALTERNATING BARS H600 OR BARS H---

SEE DETAIL "X"

BARS HA300

ALTERNATING BARS H601 OR BARS H---

1" CL. (BARS (TYP.) HA300)

SHOWING PROJECTING BARS

SHOWING NON-PROJECTING BARS

✱ PROJECTION TO BE ABOVE DECK PANEL AREA. BARS H500 & H600 ARE TO BE DIMENSIONED ACCORDINGLY.

SECTION C-C

•• 4'-9" BT-54
•• 5'-6" BT-63
•• 6'-3" BT-72
BARS H600 OR H---

4'-2" BT-54
4'-11" BT-63
5'-8" BT-72
BARS H601 OR H---

•• DIMENSION SHOWN BASED ON 5" BAR PROJECTION

LENGTH
BARS A400

ALTERNATING SIDES BARS H500, H501 DOUBLE BARS H600 & H601

PREST. STRANDS BOTTOM ROW

2'-0"
BARS HA300 (TWO PIECE NOT ALLOWED)

PREST. STRANDS BOTTOM ROW

DETAIL "X"

ESTIMATED QUANTITIES
(PER BEAM)

NO. REQ'D	CLASS "A" CONCRETE C.Y.	REINFORCING STEEL LB.	PRESTRESSING STRANDS (LOW RELAXATION) LB.

BILL OF STEEL
(PER BEAM)

BAR	SIZE	NO. REQ'D	LENGTH
A400	4		3'-2"
A500	5		5'-6"
A---			
H---			
H600	6	16	
H601	6	24	
HA300	3		4'-7"

DESIGNED BY	PAM	DATE 1/2016
DRAWN BY	PAM	DATE 1/2016
SUPERVISED BY	TEH	DATE 1/2016
CHECKED BY		DATE

NOTES & DETAILS FOR DESIGNERS/DETAILERS

ADHERE TO AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS AS CLOSELY AS POSSIBLE.
● DESIGNING A MULTI-SPAN BRIDGE WITH PPC GIRDERS ► 5.14.1.4:
  ● FOR SPANS DESIGNED CONTINUOUS, DESIGNER MUST SATISFY 5.14.1.4 AND
  ● PROVIDE A POSITIVE MOMENT CONNECTION TO SATISFY φMn > 1.2Mcr (5.14.1.4.4) WHICH MAY REQUIRE ANCHORING ADDITIONAL FULLY BONDED STRANDS IN THE CONTINUITY DIAPHRAGM THAN JUST THE BOTTOM ROW.
  ● SEE NCHRP 519, 2004, "CONNECTION OF SIMPLE SPAN PRECAST CONCRETE GIRDERS FOR CONTINUITY".
● CALCULATE TENSION REINFORCEMENT IN THE TOP OF THE GIRDER ► 5.9.4.1.2
● CHECK TRANSVERSE AND GIRDER-SLAB INTERFACE SHEAR ► 5.8.2, 5.8.3, 5.8.4
● BOND BREAKS
  ● NO MORE THAN 25% OF ALL STRANDS SHALL HAVE BOND BREAKS AND NO MORE THAN 40% OF ALL STRANDS IN ANY SINGLE ROW SHALL HAVE BOND BREAKS ► 5.11.4.3.
  ● BOND SHALL NOT BE BROKEN ON HORIZONTALLY OR VERTICALLY ADJACENT STRANDS.
  ● BOND SHALL NOT BE BROKEN ON THE BOTTOM ROW.
  ● BOND SHALL NOT BE BROKEN ON CORNER STRANDS OF THE BOTTOM ROW.
  ● IF BOND BREAKS CANNOT BE AVOIDED ON OTHER EXTERIOR STRANDS, PLACE ONLY THE SHORTEST BREAKS THERE.
  ● BOND SHALL BE BROKEN SYMMETRICALLY ABOUT CL GIRDER.
● DRAPED (AKA HARPED, RAISED, OR DEFLECTED) STRANDS
  ● DEBONDING IS PREFERRED OVER DRAPING STRANDS BUT DRAPING OR A COMBINATION OF BOTH DRAPING AND DEBONDING IS ALLOWED.
  ● HOLD DOWN POINTS FOR DRAPED STRANDS MAY BE ADJUSTED BY THE DESIGNER WITH THE FOLLOWING LIMITATIONS:
    ● VERTICAL FORCE ON A HOLD DOWN DEVICE SHALL NOT EXCEED 50 KIPS AND NO MORE THAN 10 STRANDS SHALL BE DRAPED PER DEVICE.
    ● VERTICAL FORCE FOR A SINGLE STRAND SHALL NOT EXCEED 5 KIPS.
● REMEMBER TO CHECK SHOP DRAWINGS FOR LOCATIONS OF HOLES OR INSERTS TO ATTACH INTERMEDIATE DIAPHRAGMS.
● NOTE ON PLANS WHETHER OR NOT YOU INCREASED GIRDER LENGTH TO COMPENSATE FOR ROADWAY GRADE.
● RANGE FOR BARS HA300 = 1.5 X BM HT ► 5.10.10.2.

PROJECT NO.	YEAR	SHEET NO.
	2016	

REVISIONS

NO.	DATE	BY	BRIEF DESCRIPTION
1	1-4-93	MAH	REMOVED TEMP. BRACING DETAIL, ADDED INTERMEDIATE DIAPHRAGM DETAIL AND NOTES. REVISED PROJ. BARS & CLEARANCES
2	7-15-93	MAH	MOVED TYPICAL DETAILS TO STD-14-1 ADDED PLAN SHOWING LOCATION OF INTERMEDIATE DIAPHRAGMS & NOTES
3	11-1-93	MAH	REVISED BEAM DESIGN DATA
4	12-19-94	MAH	REVISED STRAND PROJ., BARS C500, A500, & BARS HA300 & SPACING OF BARS H601
5	7-31-00	CMH	DELETED BARS C500, REVISED BILL OF STEEL, SECTION R1 - R1 & ELEVATIONS
6	5-1-08	JHW	REVISED NUMBER OF BARS H600 & H601
7	3-12-15	PAM	REVISED NOTES & DETAILS
8	1-28-16	PAM	REVISED DIAPHRAGM STEEL & NOTES
9	7-28-16	PAM	REVISED BARS HA300 & NOTES
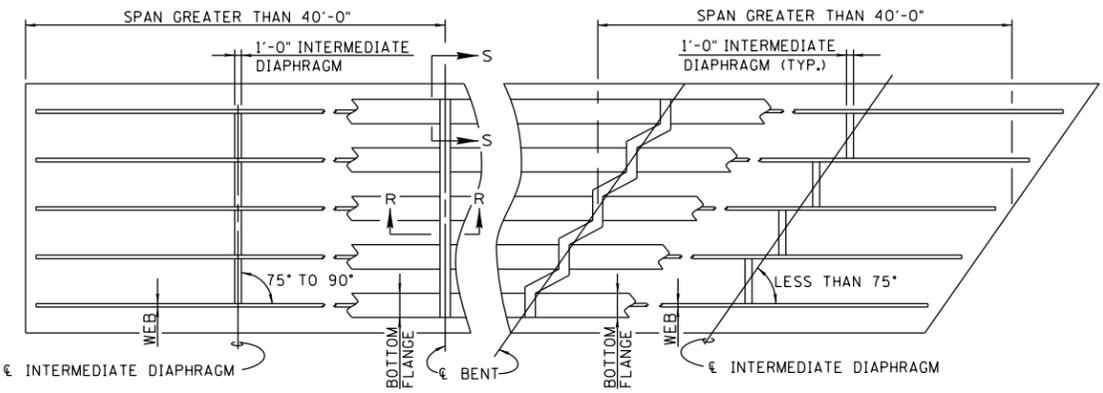
SPAN GREATER THAN 40'-0"   SPAN GREATER THAN 40'-0"

1'-0" INTERMEDIATE DIAPHRAGM   1'-0" INTERMEDIATE DIAPHRAGM (TYP.)

75° TO 90°   LESS THAN 75°

WEB   WEB

BOTTOM FLANGE   ℓ BENT   BOTTOM FLANGE

ℓ INTERMEDIATE DIAPHRAGM   ℓ INTERMEDIATE DIAPHRAGM

PLAN SHOWING LOCATION OF INTERMEDIATE DIAPHRAGMS

INTERMEDIATE DIAPHRAGMS ARE REQUIRED: (1) AT MID-SPAN FOR SPANS FROM 40 FT. TO 80 FT., (2) AT 1/3 POINTS FOR SPANS OVER 80 FT.

PRESTRESSED GIRDER DESIGN DATA PER GIRDER

LIVE LOAD DISTRIBUTION FACTORS IN LANES:

INTERIOR GIRDER LLDF	EXTERIOR GIRDER LLDF
MOMENT _____	MOMENT _____
SHEAR _____	SHEAR _____

COMPOSITE DEAD LOADS:
DC = _____ LB/FT
DW = _____ LB/FT

COMPOSITE DESIGN SLAB STRENGTH: f'c = 3000 PSI.

DOWNWARD DEFLECTION UNDER TOTAL DL IS NOT ALLOWED.

PRESTRESSING STRAND DATA

LOW-LAX, ASTM GRADE 270 KSI, UNCOATED 7-WIRE STRANDS
DIAMETER = _____ INCH
AREA = _____ SQ. IN.
INITIAL JACKING FORCE = _____ KIPS.

GIRDER NOTES:
● SEE STD-14-1 FOR BULB-TEE GIRDER STANDARD DETAILS AND NOTES.

● THE GIRDER CONCRETE SHALL ATTAIN A COMPRESSIVE STRENGTH OF NOT LESS THAN _____ PSI AT 28 DAYS AND STRESS TRANSFER SHALL NOT BE MADE UNTIL CONCRETE TEST SPECIMENS HAVE REACHED A COMPRESSIVE STRENGTH OF AT LEAST _____ PSI.

● SEE GENERAL NOTES FOR FINISHING CONCRETE SURFACES.

● TOP FLANGE TO BE CLEAN, FREE OF LAITANCE, AND INTENTIONALLY ROUGHENED TO AN AMPLITUDE OF ¼".

● ALL GIRDERS TO BE _BT-54, BT-63, OR BT-72._

4000 psi

BARS A5--

PRESTRESSING STRANDS

BARS LS400 @ 1'-0" MAX.

BARS LS401

1" BITUMINOUS FIBERBOARD ANCHOR BOLT SEE SEISMIC STANDARD

SECTION R - R

PRESTRESSING STRANDS

BARS LS401

PRESTRESSING STRANDS

SECTION R1 - R1

14"

14"

OUTSIDE EDGE OF SUPPORT DIAPHRAGM FOLLOWS OUTSIDE EDGE OF BOTTOM FLANGE

BARS LS401
(LIST IN BILL OF STEEL UNDER SUPERSTRUCTURE)

BEAM TYPE	CONCRETE C.Y. PER/FT.	TEXTURE FINISH S.Y. PER/FT	TEXTURE FINISH S.Y. PER/FT
BT-54	0.1695	1.6374	0.9391
BT-63	0.1834	1.8041	1.0224
BT-72	0.1973	1.9707	1.1057

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION

STANDARD DETAILS FOR PRESTRESSED BULB-TEE GIRDERS 2016

SECTION S-S

7 WIRE-STRAND WEIGHT
⁵⁄₁₀" DIA. = 0.740 LB./FT.
½" DIA. = 0.525 LB./FT.

CORRECT _____ Wayne J. Seger
ENGINEER OF STRUCTURES

SM05 BULB-TEES

NOTES AND DETAILS FOR GIRDER DRAWING IN CONTRACT PLANS

℄ INTERMEDIATE DIAPHRAGM (SEE STD-14-2 FOR DETAILS)
AT MIDPOINT FOR SPANS > 80 FT.
AT MIDPOINT FOR CURVED BRIDGES WITH RADIUS < 800 FT.

PRECAST LENGTH

ELEVATION C-C

SYMMETRY ABOUT MIDPOINT

STRAND GROUP I

NO. STRANDS AND HEIGHT DETERMINED BY DESIGN.

3" MIN. TO TOP STRAND

STRAND GROUP II

1" MINIMUM, 3" MAXIMUM FOR STRANDS NOT BENT

HOLD DOWN POINT DETERMINED BY DESIGN

PROJECT 8½" AND BEND EVERY OTHER FULLY BONDED STRAND IN BOTTOM ROW (SEE STD-14-2)

ELEVATION
(SHOWING STRAND DEFLECTION ARRANGEMENT)

PRECAST LENGTH (B)

(A)

3"
5 SPA. @ 6" = 2'-6"
4 SPA. @ 3" = 1'-0"
NO. 6 BARS
BARS A- ⅝"
BARS A-
MIN. SPLICE
BARS A- ⅝"
BARS A-
5 SPA. @ 6" = 2'-6"
3"
NO. 6 BARS
4 SPA. @ 3" = 1'-0"

4½"

BARS HA300
7 SPA. @ 6" = 3'-6" TYPE I
9 SPA. @ 6" = 4'-6" TYPE II
12 SPA. @ 6" = 6'-0" TYPE III
14 SPA. @ 6" = 7'-0" TYPE IV

℄ INTERMEDIATE DIAPHRAGM SEE STD-14-2 FOR DETAILS

1½" RADIUS FILLET (TYP. EACH END OF GIRDER)

ELEVATION
(SHOWING STIRRUP ARRANGEMENT)

BARS A-
BARS A-
PRESTRESSING STRANDS
NON-PROJECTING BARS H601
PROJECTING BARS H600
1"
1"
1"
1"
5" PROJ.
2"
3" MIN.
2"
BARS HA300
STRANDS @ 2" SPACING
2"
STRAND GROUP I
STRAND GROUP II
STRANDS @ 2" SPACING
2" SPA. c.c.
¾" CHAMFER (TYP.)
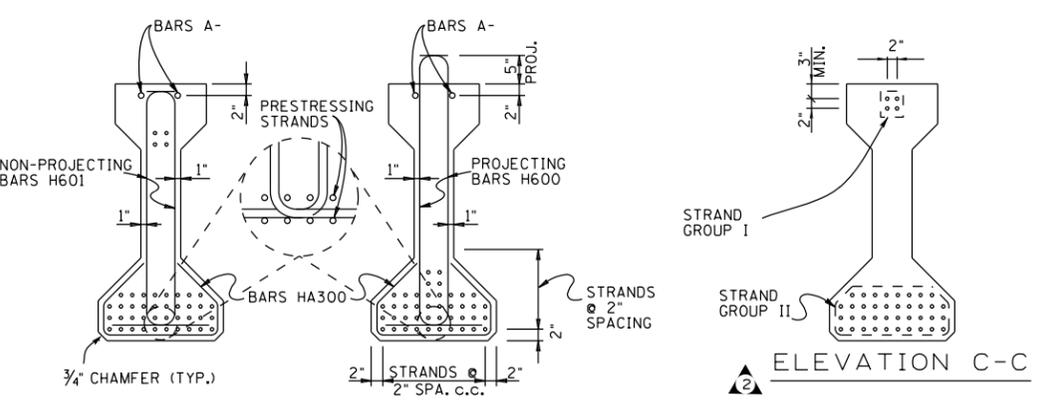
SECTION A-A
(TYPE III I-BEAM SHOWN)

SECTION B-B
(TYPE III I-BEAM SHOWN)

ELEVATION C-C

(A) 8" MIN. SPACING FOR PROJECTING BARS
18" MAX. SPACING FOR STIRRUPS

FABRICATOR SHALL ALTERNATE ORIENTATION OF SINGLE STIRRUPS, IF USED.

(B) PRECAST LENGTH ADJUSTED FOR ROADWAY GRADE.

PRESTRESSED GIRDER DESIGN DATA PER GIRDER

LIVE LOAD DISTRIBUTION FACTORS IN LANES:
INTERIOR GIRDER LLDF   EXTERIOR GIRDER LLDF
MOMENT _____   MOMENT _____
SHEAR _____   SHEAR _____

COMPOSITE DEAD LOADS:
DC = _____ LB/FT
DW = _____ LB/FT

COMPOSITE DESIGN SLAB STRENGTH: f'c = 3000 PSI.

DOWNWARD DEFLECTION UNDER TOTAL DL IS NOT ALLOWED.

PRESTRESSING STRAND DATA

LOW-LAX, ASTM GRADE 270 KSI, UNCOATED 7-WIRE STRANDS
DIAMETER = _____ INCH
AREA = _____ IN²
INITIAL JACKING FORCE = _____ KIPS.

GIRDER NOTES:

● SEE STD-14-2 FOR PRESTRESSED I-GIRDER STANDARD DETAILS AND NOTES.

● THE GIRDER CONCRETE SHALL ATTAIN A COMPRESSIVE STRENGTH OF NOT LESS THAN _____ PSI AT 28 DAYS AND STRESS TRANSFER SHALL NOT BE MADE UNTIL CONCRETE TEST SPECIMENS HAVE REACHED A COMPRESSIVE STRENGTH OF AT LEAST _____ PSI.

● SEE GENERAL NOTES FOR FINISHING CONCRETE SURFACES.

● TOP FLANGE TO BE CLEAN, FREE OF LAITANCE, AND INTENTIONALLY ROUGHENED TO AN AMPLITUDE OF ¼".

● ALL GIRDERS ARE AASHTO - PCI STANDARD TYPE I...IV.

ESTIMATED QUANTITIES
(PER BEAM)

NO. REQ'D.	CLASS 'A' CONCRETE C.Y.	STEEL BAR REINFORCING LB.	PRESTRESSING STRANDS (LOW RELAXATION) LB.

BILL OF STEEL
(PER BEAM)

BAR	SIZE	NO. REQ'D.	LENGTH
A500	5		5'-6"
A-			
H500	5		
H501	5		
H600	6		
H601	6		
HA300	3		

DESIGNED BY __PAM__   DATE 1-2016
DRAWN BY __PAM__   DATE 1-2016
SUPERVISED BY __TEH__   DATE 1-2016
CHECKED BY _____   DATE _____

NOTES & DETAILS FOR DESIGNERS & DETAILERS

ADHERE TO AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS AS CLOSELY AS POSSIBLE.
● DESIGNING A MULTI-SPAN BRIDGE WITH PPC GIRDERS ► 5.14.1.4:
  ● FOR SPANS DESIGNED CONTINUOUS FOR LOADING, THE DESIGNER MUST SATISFY 5.14.1.4, AND
  ● PROVIDE A POSITIVE MOMENT CONNECTION TO SATISFY $\phi M_n \geq 1.2 M_{cr}$ (5.14.1.4.4) WHICH MAY REQUIRE ANCHORING ADDITIONAL FULLY BONDED STRANDS IN THE CONTINUITY DIAPHRAGM THAN JUST THE BOTTOM ROW.
  ● SEE NCHRP 519, 2004, "CONNECTION OF SIMPLE SPAN PRECAST CONCRETE GIRDERS FOR CONTINUITY".
● CALCULATE TENSION REINFORCEMENT IN THE TOP OF THE GIRDER ► 5.9.4.1.2
● CHECK TRANSVERSE AND GIRDER-SLAB INTERFACE SHEAR ► 5.8.2, 5.8.3, 5.8.4
● BOND BREAKS
  ● NO MORE THAN 25% OF ALL STRANDS SHALL HAVE BOND BREAKS AND NO MORE THAN 40% OF ALL STRANDS IN ANY SINGLE ROW SHALL HAVE BOND BREAKS ► 5.11.4.3.
  ● BOND SHALL NOT BE BROKEN ON HORIZONTALLY OR VERTICALLY ADJACENT STRANDS.
  ● BOND SHALL NOT BE BROKEN ON CORNER STRANDS OF THE BOTTOM ROW.
  ● IF BOND BREAKS CANNOT BE AVOIDED ON OTHER EXTERIOR STRANDS, PLACE ONLY THE SHORTEST BREAKS THERE.
  ● BOND SHALL BE BROKEN SYMMETRICALLY ABOUT CL GIRDER.
● DRAPED (AKA HARPED, RAISED, OR DEFLECTED) STRANDS
  ● DEBONDING IS PREFERRED OVER DRAPING STRANDS BUT DRAPING OR A COMBINATION OF BOTH DRAPING AND DEBONDING IS ALLOWED.
  ● HOLD DOWN POINTS FOR DRAPED STRANDS MAY BE ADJUSTED BY THE DESIGNER WITH THE FOLLOWING LIMITATIONS:
    ● VERTICAL FORCE ON A HOLD DOWN DEVICE SHALL NOT EXCEED 50 KIPS AND NO MORE THAN 10 STRANDS SHALL BE DRAPED PER DEVICE.
    ● VERTICAL FORCE FOR A SINGLE STRAND SHALL NOT EXCEED 5 KIPS.
● REMEMBER TO CHECK SHOP DRAWINGS FOR LOCATIONS OF HOLES OR INSERTS TO ATTACH INTERMEDIATE DIAPHRAGMS.
● NOTE ON PLANS WHETHER OR NOT YOU INCREASED GIRDER LENGTH TO COMPENSATE FOR ROADWAY GRADE.
● RANGE FOR BARS HA300 = 1.5 X BM HT ► 5.10.10.2.

PROJECT NO.	YEAR	SHEET NO.
	2016	

REVISIONS

NO.	DATE	BY	BRIEF DESCRIPTION
1	7-31-00	CMH	DELETED BARS C500. REVISED BILL OF STEEL. SECTION R1 - R1 & ELEVATION
2	3-12-15	PAM	MODIFIED & ADDED NOTES.
3	1-28-16	PAM	REVISED DIAPHRAGM STEEL & NOTES
4	7-28-16	PAM	REVISED BARS HA300 & NOTES
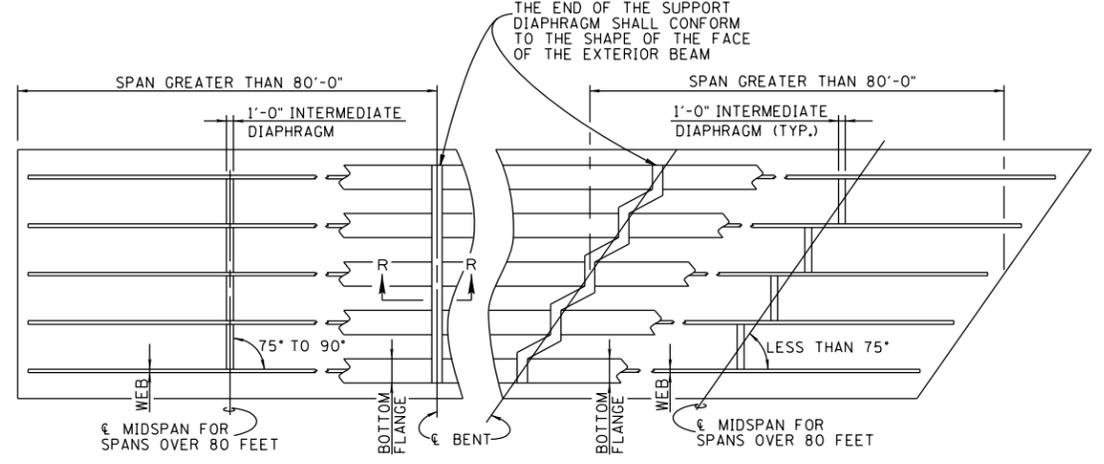
THE END OF THE SUPPORT DIAPHRAGM SHALL CONFORM TO THE SHAPE OF THE FACE OF THE EXTERIOR BEAM

SPAN GREATER THAN 80'-0"
1'-0" INTERMEDIATE DIAPHRAGM
SPAN GREATER THAN 80'-0"
1'-0" INTERMEDIATE DIAPHRAGM (TYP.)

R
R
75° TO 90°
LESS THAN 75°
℄ BENT
WEB
BOTTOM FLANGE
BOTTOM FLANGE
WEB
℄ MIDSPAN FOR SPANS OVER 80 FEET
℄ MIDSPAN FOR SPANS OVER 80 FEET

PLAN SHOWING LOCATION OF INTERMEDIATE DIAPHRAGMS
INTERMEDIATE DIAPHRAGMS ARE REQUIRED FOR SPANS OVER 80 FEET AND BRIDGES ON HORIZONTAL CURVE WITH RADIUS LESS THAN 800 FEET.

4000 psi
1'-6"
BARS A5-
3" TO MAX.
BARS LS400 @ 1'-0" MAX.
R1
R1
BARS LS401
1" BITUMINOUS FIBERBOARD
ANCHOR BOLT SEE SEISMIC STANDARD

SECTION R - R

1'-6"
PRESTRESSING STRANDS
BARS LS401
PRESTRESSING STRANDS
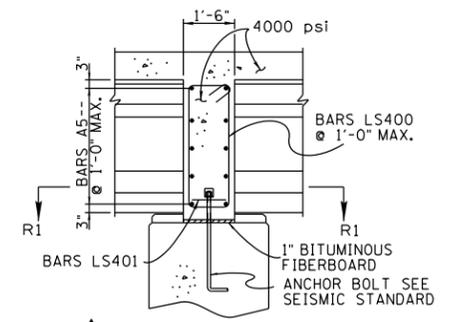
SECTION R1 - R1
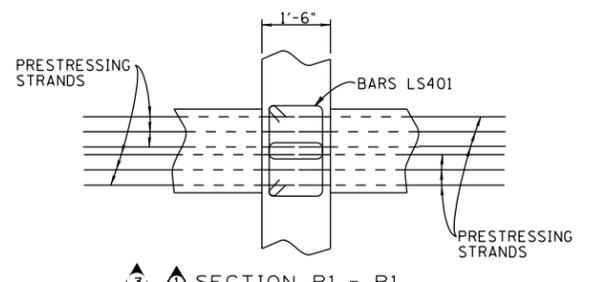
BEAM TYPE	CONCRETE C.Y. PER FOOT	TEXTURE FINISH S.Y. PER FOOT	TEXTURE FINISH S.Y. PER FOOT
I	0.07098	0.7281	0.4381
II	0.0949	0.9024	0.5345
III	0.14403	1.1291	0.6664
IV	0.20293	1.3558	0.79893

14"
45°
14"

BARS LS401
(LIST IN BILL OF STEEL UNDER SUPERSTRUCTURE)

7 WIRE-STRAND WEIGHT
⁶⁄₁₀" DIA. = 0.740 LB./FT.
½" DIA. = 0.525 LB./FT.

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION

STANDARD DETAILS FOR PRESTRESSED I-GIRDERS 2016

CORRECT _Wayne J. Seger_
ENGINEER OF STRUCTURES

SM05 I-GIRDERS

NOTES AND DETAILS FOR
BEAM DRAWING IN CONTRACT PLANS

PROJECTING: DBL BARS ZA- ⅝ H-
NON-PROJECTING: BARS H- ⅝ H-

PROJECTING: DBL BARS ZA- ⅝ H-
NON-PROJECTING: BARS H- ⅝ H-
(FLARED STIRRUPS)

PRECAST LENGTH Ⓐ

PROJECTING DBL BARS ZA- ⅝ H- , (NON-PROJECTING BARS H- ⅝ H- AS NEEDED)

Ⓐ PRECAST LENGTH ADJUSTED FOR ROADWAY GRADE.

BARS C- ⅝ BARS C-

SYMMETRY ABOUT MIDPOINT

2 SPA. @ _" c.c.

PROJECTING BAR SPACING
8" MINIMUM, 12" MAXIMUM

2 SPA. @ _" c.c.

4½"

2"

¾" END CHAMFER (TYP.)

8"

1" MINIMUM, 2" MAXIMUM
FOR STRANDS NOT BENT

PRESTRESSING STRANDS

1" DIA. DRAIN
(EACH END OF
EACH VOID)

Ⓔ END OF BEAM

5½"

PROJECT 8½" AND BEND EVERY OTHER
FULLY BONDED STRAND IN BOTTOM ROW
(SEE STD-14-3)

BARS H300
4 SPA. @ 6"=2'-0"

ELEVATION
ALONG Ⓔ BEAM

90°

Ⓔ BEAM

INTERIOR DIAPHRAGM

VOID Ⓒ

STIRRUPS PARALLEL TO END OF BEAM
(75°- 90° SKEWS)
FLARED STIRRUPS
(60° - 75° SKEWS)

CLIP CORNER 2" WHEN
SKEW IS LESS THAN 75°

60° MIN.

2" MIN. SPA.

1'-6"   25'-0" MAXIMUM   9"   25'-0" MAXIMUM   9"   25'-0" MAXIMUM   1'-6"

PRECAST LENGTH

PLAN

DOUBLE BARS ZA- (OR
H- FOR NON-PROJECTING
STIRRUPS)

BARS C- ⅝ C-

1" CL. MIN.

BARS H300 AT
END OF BEAMS
1¼" CL.

BARS H-
2¼" CL.

STRANDS @ 2" c.c. SPA.
SEE INDIVIDUAL DESIGN

REINFORCEMENT

DOUBLE BARS ZA- (OR
H- FOR NON-PROJECTING
STIRRUPS)

BARS C- ⅝ C-

1" CL. MIN.

1" CL. MIN.

BARS A300

BARS H300 AT
END OF BEAMS
1¼" CL.

18" MAX., 18" MAX.

BARS H-
2¼" CL.

STRANDS @ 2" c.c. SPA.
SEE INDIVIDUAL DESIGN

BOX BEAM DIMENSIONS

¾" CHAMFER
(TYP.)

(TYP.)

ESTIMATED QUANTITIES
(PER BEAM)

NO. REQ'D.	CLASS 'A' CONCRETE C.Y.	STEEL BAR REINFORCING LB.	PRESTRESSING STRANDS (LOW RELAXATION) LB.

DESIGNED BY	PAM/MH	DATE	1-2016
DRAWN BY	PAM/KF	DATE	1-2016
SUPERVISED BY	TEH	DATE	1-2016
CHECKED BY		DATE	

BILL OF STEEL
(PER BEAM)

BAR	SIZE	NO. REQ'D.	LENGTH
A300	3		
A500	5		5'-6"
A501	5		
A502	5		
C-			
G400	4		2'-4"
H300	3	10	
H400	4		
ZA400	4		

PRESTRESSED BEAM DESIGN DATA PER BEAM

LIVE LOAD DISTRIBUTION FACTORS IN LANES:
INTERIOR BEAM LLDF   EXTERIOR BEAM LLDF
MOMENT _____   MOMENT _____
SHEAR _____   SHEAR _____

COMPOSITE DEAD LOADS:
DC = _____ LB/FT
DW = _____ LB/FT

COMPOSITE DESIGN SLAB STRENGTH: f'c = 3000 PSI.

DOWNWARD DEFLECTION UNDER TOTAL DL IS NOT ALLOWED.

PRESTRESSING STRAND DATA

LOW-LAX, ASTM GRADE 270 KSI, UNCOATED 7-WIRE STRANDS
DIAMETER = _____ INCH
AREA = _____ IN²
INITIAL JACKING FORCE = _____ KIPS.

BEAM NOTES:

● SEE STD-14-3 FOR PRESTRESSED BOX BEAM STANDARD
DETAILS AND NOTES.

● THE BEAM CONCRETE SHALL ATTAIN A
COMPRESSIVE STRENGTH OF NOT LESS THAN
_____ PSI AT 28 DAYS AND STRESS TRANSFER
SHALL NOT BE MADE UNTIL CONCRETE TEST SPECIMENS
HAVE REACHED A COMPRESSIVE STRENGTH OF AT LEAST
_____ PSI.

● SEE GENERAL NOTES FOR FINISHING CONCRETE SURFACES.

● TOP FLANGE TO BE CLEAN, FREE OF LAITANCE, AND
INTENTIONALLY ROUGHENED TO AN AMPLITUDE OF ¼".

● ALL BOX BEAMS ARE _____" H X _____" W.


NOTES & DETAILS FOR DESIGNERS & DETAILERS

ADHERE TO AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS AS CLOSELY AS POSSIBLE.
● DESIGNING A MULTI-SPAN BRIDGE WITH PPC GIRDERS ► 5.14.1.4:
  ● FOR SPANS DESIGNED CONTINUOUS FOR LOADING, THE DESIGNER MUST SATISFY 5.14.1.4, AND
  ● PROVIDE A POSITIVE MOMENT CONNECTION TO SATISFY φMn ≥ 1.2Mcr (5.14.1.4.4) WHICH MAY
  REQUIRE ANCHORING ADDITIONAL FULLY BONDED STRANDS IN THE CONTINUITY DIAPHRAGM THAN JUST
 THE BOTTOM ROW.
  ● SEE NCHRP 519, 2004, "CONNECTION OF SIMPLE SPAN PRECAST CONCRETE GIRDERS FOR CONTINUITY".
● CALCULATE TENSION REINFORCEMENT IN THE TOP OF THE GIRDER ► 5.9.4.1.2
● CHECK TRANSVERSE AND BEAM-SLAB INTERFACE SHEAR ► 5.8.2, 5.8.3, 5.8.4
● BOND BREAKS
  ● NO MORE THAN 25% OF ALL STRANDS SHALL HAVE BOND BREAKS AND NO MORE THAN
  40% OF ALL STRANDS IN ANY SINGLE ROW SHALL HAVE BOND BREAKS ► 5.11.4.3.
  ● BOND SHALL NOT BE BROKEN ON HORIZONTALLY OR VERTICALLY ADJACENT STRANDS.
  ● BOND SHALL NOT BE BROKEN ON CORNER STRANDS OF THE BOTTOM ROW NOR IN THE WEB.
  ● IF BOND BREAKS CANNOT BE AVOIDED ON OTHER EXTERIOR STRANDS, PLACE ONLY
  THE SHORTEST BREAKS THERE.
  ● BOND SHALL BE BROKEN SYMMETRICALLY ABOUT CL GIRDER.
● STIRRUPS - SIZE AND SPACING OF PROJECTING & NON-PROJECTING BARS DETERMINED BY DESIGN.
  ● WHEN ADDITIONAL INTERFACE SHEAR STEEL IS REQUIRED BEYOND DOUBLE ZA- BARS AT 8" SPACING,
  USE SUPPLEMENTAL REINFORCING PER DETAIL "A".
  ● NON-PROJECTING BARS HAVE 2 H- BARS, ONE TOP & ONE BOTTOM.
  ● BARS H300 ARE ALWAYS PARALLEL TO END OF BEAM.

Ⓐ ● NOTE ON PLANS WHETHER OR NOT YOU INCREASED GIRDER LENGTH TO COMPENSATE FOR ROADWAY GRADE.
Ⓑ ● FLANGE THICKNESSES MAY BE INCREASED BY ½" INCREMENTS AS REQUIRED BY DESIGN.
  ● WEB & FLANGE THICKNESSES MUST CONFORM TO 5.14.1.2.2. OFFICE POLICY IS TO USE 3½" MIN. TOP FLANGE.
Ⓒ ● DETAIL SINGLE OR DUAL VOIDS AS APPROPRIATE.

DIMENSIONS OF SECTIONS

36" OR 48"

DEPTH VARIES FROM 27" IN 3" INCREMENTS

VARIES WITH DEPTH

≥ 3½"

≥ 5"

VOID SIZE AND LOCATION
BY INDIVIDUAL DESIGN

DEPTH VARIES
12" MIN, 24" MAX.

9"
OR 12"

18" OR 24"

9"
OR 12"

PLAN SHOWING SUPPORT DIAPHRAGMS

60° TO 90°

LESS THAN 60°

Ⓔ BENT

DETAIL 'A'

BARS G-

BARS ZA-

5" PROJ.

SECTION R1 - R1
(USE THREE BARS LS401 FOR 48" BOX BEAM)

36" BOX

PRESTRESSING STRANDS

BARS LS401

1'-6"

BARS LS401

BARS LS401
(LIST IN BILL OF STEEL
UNDER SUPERSTRUCTURE)

14"

14"

45°

BARS LS401

14"

14"

45°

SKEW TO MATCH
SKEW OF BEAM

BARS LS401
(LIST IN BILL OF STEEL
UNDER SUPERSTRUCTURE)

PRESTRESSING STRANDS

BARS LS401

36" BOX

SECTION R1 - R1
(USE THREE BARS LS401 FOR 48" BOX BEAM)

SECTION R - R

CLASS "D" CONCRETE

BARS A5-

BARS LS400
@ 1'-0" MAX.

BARS LS401

1" BITUMINOUS
FIBERBOARD
ANCHOR BOLT SEE
SEISMIC STANDARD

R1   R1   R   R

7 WIRE-STRAND WEIGHT
⁹⁄₁₆" DIA. = 0.740 LB./FT.
½" DIA. = 0.525 LB./FT.

CORRECT

ENGINEER OF STRUCTURES

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION

STANDARD DETAILS
FOR
PRESTRESSED
BOX BEAMS
2016

SM05 BOX BEAMS