EFFECTS OF COMPRESSIVE STRESS LIMITS ON PRESTRESSED CONCRETE GIRDER DESIGN

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ABSTRACT

Current AASHTO and PennDOT LRFD Specifications specify compressive stress limits for prestressed concrete bridge girders under service loads, and mandate their use in design. Recent research conducted at the University of Nebraska suggests checking these compressive stress limits are unnecessary.

This paper extends the University of Nebraska research to bridge beams in Pennsylvania where the effects of eliminating the compressive stress limits are closely examined. Comparisons are made between two designs. The first design adopts the compressive stress limits as specified in the current AASHTO and PennDOT LRFD Specifications. The second design eliminates these compressive stress limits in the intermediate and final stages, and uses a slightly increased and justified stress limit in the initial stage.

Results indicate that eliminating the required compressive stress limits, coupled with a slight increase in compressive limits at the initial stage, can result in shallower beam depths and slightly longer span lengths for our national bridges.

Keywords: Design Methodologies, History-Historical, LRFD Specifications, High Strength Concrete, Case Study

INTRODUCTION

The conventional method to designing prestressed, pretensioned concrete beams involves the combination of Working Stress Design (WSD) and Ultimate Strength Design (USD) principles. With reference to bridge beams in Pennsylvania, the AASHTO LRFD[1] and PennDOT Design Manual, Part 4 specifications[2] (DM4) are the primary sources to design prestressed members including specifying the allowable compressive stress limits. Accordingly, for WSD, engineers check that stresses under service loads (at various loading stages) do not exceed allowable stresses and use USD to prevent possible collapse under factored loads. For most bridges in Pennsylvania, the design of a prestressed beam is controlled by WSD criteria. Typical bridge sections used in Pennsylvania are shown in Figure 1.

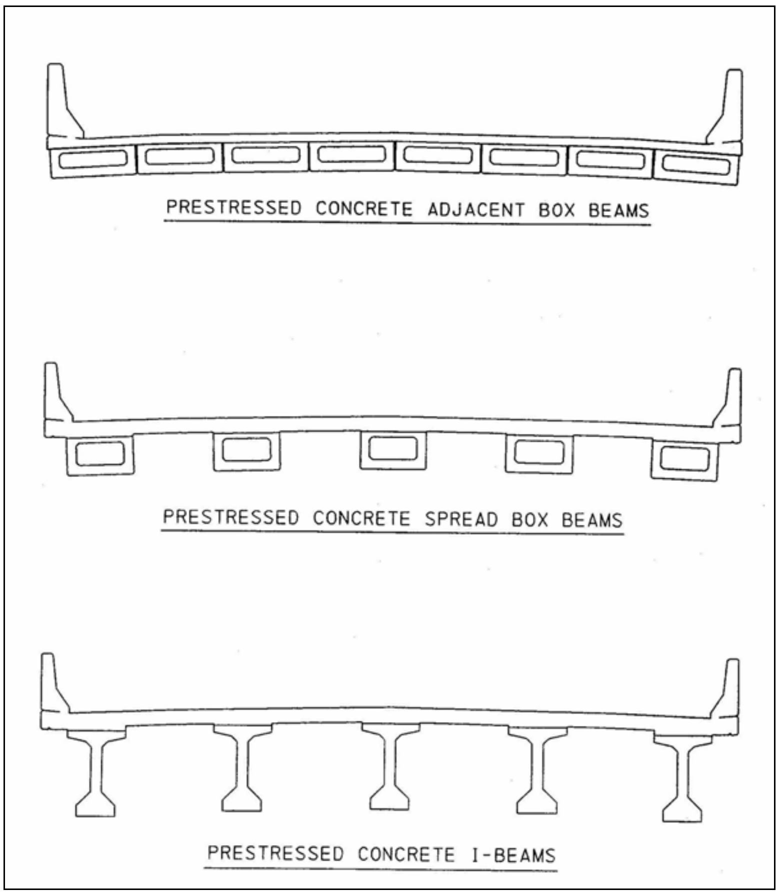

Figure 1 – Typical PennDOT Bridge Sections

The compressive stress limits were essentially adopted from conventional reinforced concrete design. Noppakunwijai et al.[3] determined that in 1953, the "AASHTO Standard Specifications for Highway Bridges" specified a 0.4 f'_c compressive stress limit for conventional concrete subject to dead, live, and impact loads. The Bureau of Public Roads (later renamed as the Federal Highway Administration) conservatively adopted this same limit for prestressed concrete. This limit was later revised to 0.45 f'_c in 1958. Noppakunwijai et al.[3] also noted the 0.45 f'_c limit was removed for conventional concrete design at the introduction of strength limit design, but conservatively remained for prestressed concrete design.

The compressive stress limit was unchanged in prestressed concrete until 1995 when the limit was increased to 0.6 f'_c under effective prestress, dead load and live load. This was primarily based on work by Huo et al[4], which mentions that the intent of the 0.6 f'_c is transitional with the goal of eliminating this and all compressive stress limits. In fact, a review of the Nebraska Department of Road's (NDOR) Bridge Operations, Policy and Procedures (BOPP) Manual[5] indicates the NDOR have already eliminated checking these compressive stress limits at the initial, intermediate and final stages primarily based on the work in references 3 and 9.

Noppakinwijai et al.[3] and Huo et al.[4] basically state that using WSD to compute concrete stresses in a prestressed beam using a linear elastic analysis does not correspond to actual member behavior, which is non-linear and time-dependent. In a composite prestressed beam, continuous stress redistribution occurs due to the differential creep and shrinkage of the two concrete components (deck and beam). These time dependent effects provide relief to the beam that reduces the compressive stresses. The current compressive limits are also not an indicator of when concrete will crush in compression[3], which is believed by some to be the main purpose of the 0.6 f'_c limit (at final stage), or control serviceability and crack control. USD prevents crushing under factored loads and the current AASHTO LRFD has provisions to control serviceability, excessive deflections and cracking under service loads.

The main concern with eliminating the compressive stress limits appears to be the effects of long term creep in continuous structures, especially under prestress plus dead load (the AASHTO 0.45 f'_c limit). At the 2003 PCI Convention, members of the LRFD subcommittee on Bridges[6] discussed the issue with members stating "Concern that high permanent stresses due to prestress and dead load could lead to creep failure, particularly in negative moment regions, over continuity piers."

With continued research, the concerns about long term creep will be addressed. However, the primary objective of the noted research above is to propose the elimination of "arbitrary compressive stress limits[3]" and maximize the "full potential of prestressed concrete design[3]." To maximize this potential, USD should be given more emphasis in designing prestressed concrete structures, especially in the initial stage. USD was first introduced more than forty years ago and has gained acceptance as a primary design principle. For prestressed concrete, it is basically a check to prevent collapse under factored loads. It rarely controls the design because of the continued use of WSD.

OBJECTIVES

The primary objective of this paper is to submit the results of a study that investigated the effects of eliminating compressive stress limits on composite, prestressed concrete bridge beams that are commonly used in Pennsylvania. To do this, a parametric study was conducted for twelve bridges with various span lengths and beam spacing. The bridges were analyzed using the AASHTO LRFD specifications[1] as amended by PennDOT Design Manual, Part 4 (DM4)[2] using two methods; the first being the conventional method of using working stress design and ensuring the actual service limit stresses are less than the allowable compressive stress limits (Method 1). The second “proposed” method (Method 2) re-analyzed these bridges by eliminating the compressive stress limits imposed by AASHTO and the PennDOT DM4. In addition, Method 2 used an increased, but justified initial compressive stress limit compared to the conventional limit in Method 1. A comparison is made between the two methods for such parameters as beam sizes, number of required strands, and initial and final concrete stresses. The analyses are performed using the PennDOT PSLRFD program[7].

Recommendations are provided for future consideration of compressive stress limits.

The twelve bridges that were analyzed are as follows:

Table 1 – Twelve Bridges in Parametric Study

Bridge Number	Beam Type	Beam Size	Number of Spans	Span Length(s)- feet
1	Adjacent Box Beam	48x21	1	63'-0"
2	Adjacent Box Beam	48x39	1	110'-0"
3	Adjacent Box Beam	48x60	1	140'-0"
4	Spread Box Beam	48x24	1	55'-0"
5	Spread Box Beam	48x45	1	97'-0"
6	Spread Box Beam	48x66	1	125'-0"
7	PA I Beam	26x36	3	2@66-4", 1@64'-7"
8	AASHTO I Beam	28x66	5	5 @ 121' ±
9	AASHTO I Beam	28x90	1	143'-0"
10	PCEF Bulb Tee Beam	33x39.25	1	86'-0"±
11	PCEF Bulb Tee Beam	33x61.25	1	111'-0"±
12	PCEF Bulb Tee Beam	33x79.25	1	127'-0"±

METHODOLOGY AND ANALYSIS PROCEDURE

The twelve bridges were analyzed using two methods. Method 1 is the conventional analysis and Method 2 is the proposed method with elimination of the compressive stress limits.

The current AASHTO LRFD[1] allowable compressive stress limits are shown in Figure 2. It should be noted that PennDOT's DM4[2] amends the AASHTO compressive stress limit for the sum of the effective prestress plus permanent dead loads from 0.45 f'_c to 0.40 f'_c.

Location	Stress Limit
• In other than segmentally constructed bridges due to the sum of effective prestress and permanent loads	0.45 f'_c (KSI)
• In segmentally constructed bridges due to the sum of effective prestress and permanent loads	0.45 f'_c (KSI)
• In other than segmentally constructed bridges due to live load and one-half the sum of effective prestress and permanent loads	0.40 f'_c (KSI)
• Due to the sum of effective prestress, permanent loads, and transient loads and during shipping and handling	0.60 $\varphi_w f'_c$ (KSI)

Figure 2 – AASHTO LRFD Allowable Compressive Stress Limits[1]

METHOD 1 - CONVENTIONAL ANALYSIS

The current approach for computing stresses in the extreme fibers typically involves a linear elastic analysis at the initial, intermediate, and final stages (WSD).

Initial Loading Immediately After Prestress Transfer – P/S + Self Weight:

The initial stage is where the prestressing force is transferred to the precast beam before the deck is cast. Usually this stage takes place at the beam fabrication facility. The compressive and tensile stresses immediately after prestress transfer are computed using the following linear stress relationships between the top and bottom fibers[8].

$$f_t = \frac{-P_i}{A}\left[1 - \frac{ec_t}{r^2}\right] - \frac{M_D}{S_t} \qquad (1)$$

$$f_b = \frac{-P_i}{A}\left[1 + \frac{ec_b}{r^2}\right] + \frac{M_D}{S_b} \qquad (2)$$

Where:

f_t , f_b = Concrete stresses at top and bottom extreme fibers, respectively
P_i = Initial prestressing force (at transfer)
M_D = Moment due to self-weight
e = Eccentricity of tendons from the concrete section center of gravity (cgc)
A = Cross-sectional area of the precast beam

c_b , c_t = Distances from the c.g. to the extreme top and bottom fibers, respectively, for the precast section alone
r = Radius of gyration
S_t , S_b = Section moduli of the precast section alone, referencing the extreme top and bottom fibers of the precast section, respectively

Intermediate Stage - P/S +∑DL:

The intermediate stage accounts for the effective prestress force on the beam plus all sustained dead loads after the concrete slab has hardened. The stresses in the extreme fibers are computed as follows[8]:

$$f_t = \frac{-P_i}{A}\left[1 - \frac{ec_t}{r^2}\right] - \frac{M_D + M_{SD}}{S_t} - \frac{M_{CSD}}{S_{Ct}} \quad (3)$$

$$f_b = \frac{-P_i}{A}\left[1 + \frac{ec_b}{r^2}\right] + \frac{M_D + M_{SD}}{S_b} + \frac{M_{CSD}}{S_{Cb}} \quad (4)$$

Where:

P_e = Effective prestressing force after all losses
M_{SD} = Moment due to superimposed dead load (cast-in-place slab and diaphragms) applied prior to composite action between the girders and slab
M_{CSD} = Moment due to superimposed dead load (rail/barrier weight) applied after composite action between the girders and slab
S_{Ct}, S_{Cb} = Section moduli of the composite section, referencing the extreme top and bottom fibers of the precast section, respectively

Final Stage - P/S + ∑DL + LL:

The final loading stage is where the effective prestressing force, all dead loads plus live loads (including impact) are acting on the composite section after the concrete deck has hardened. The stresses in the extreme fibers are[8]:

$$f_t = \frac{-P_i}{A}\left[1 - \frac{ec_t}{r^2}\right] - \frac{M_D + M_{SD}}{S_t} - \frac{M_{CSD} + M_{L+I}}{S_{Ct}} \quad (5)$$

$$f_b = \frac{-P_i}{A}\left[1 + \frac{ec_b}{r^2}\right] + \frac{M_D + M_{SD}}{S_b} + \frac{M_{CSD} + M_{L+I}}{S_{Cb}} \quad (6)$$

Where M_{L+I} is the moment due to live and impact load. It should be noted that PennDOT uses the transformed section properties for live load, which includes transforming the area of the prestressing steel.

The above equations are used to determine the stresses along sections of a prestressed beam such as the $1/10^{th}$ points. The critical sections where the maximum stresses occur can then be identified for each loading stage.

METHOD 2 – PROPOSED ANALYSIS

The proposed analysis re-analyzed the twelve bridges by neglecting the compressive stress limits at the intermediate and final stages. At the initial stage, strength design or a more simplified, but justified approach can be applied where the initial compressive stress limit is slightly increased based on an empirical equation developed by Noppakinwijai et al.[9] These two principles are discussed below.

STRENGTH DESIGN METHOD AT PRESTRESS TRANSFER

The strength design approach at prestress transfer is based on research by Noppakinwijai et al.[9] and is currently specified for use in the BOPP manual[5] by the Nebraska Department of Roads (NDOR).

Strength Design at prestress transfer would involve analyzing the prestressed beam similar to a conventional reinforced concrete column. It would involve trial and error of the unknown variables and development of interaction diagrams to develop the required release strengths. By using this method, it is apparent this approach would be a significant change from the more familiar WSD methods. However, Noppakinwijai et al.[9] have developed an alternative and simplified method called the "*Approximate Equivalent Allowable Stress Method*," which uses the same analysis principles as the conventional method (Method 1), but also accounts for the added benefits of the strength design approach. This method is easier to apply and considered a transitory approach to using strength design principles. For this study, the "*Approximate Equivalent Allowable Stress Method*" was used as discussed in the next section.

APPROXIMATE EQUIVALENT ALLOWABLE STRESS FOR PRETENSIONED MEMBERS

The approximate equivalent stress method for pretensioned members can be used to develop release strengths comparable to the strength design method, but are much easier to apply. The objective is to use the familiar allowable stress design procedures but use an alternative limit to 0.6 f'_{ci}. The alternative limit would be an approximate result as if the more rigorous strength design method was used.

The empirical method computes a value K such that the compressive stress limit is K f'_{ci}. K allows an increase above 0.6 f'_{ci} as shown in the following equation[9]:

$$\left(0.6 + \frac{y_b}{5h}\right) f'_{ci} \leq 0.75 f'_{ci} \tag{7}$$

For this study, this approximate method was utilized due to its simplicity and easier conformity with the PSLRFD program[7]. The K values for the twelve bridges in this study are tabulated in Table 2.

Table 2 – K Values for Approximate Equivalent Stress Method

Bridge Number	Beam Type	Beam Size	Number of Spans	yb in	K[1] Value
1	Adjacent Box Beam	48x21	1	10.41	0.70
2	Adjacent Box Beam	48x39	1	19.29	0.70
3	Adjacent Box Beam	48x60	1	29.7	0.70
4	Spread Box Beam	48x24	1	10.77	0.69
5	Spread Box Beam	48x45	1	20.34	0.69
6	Spread Box Beam	48x66	1	30.29	0.69
7	PA I Beam	26x36	3	16.36	0.69
8	AASHTO I Beam	28x66	5	33.43	0.70
9	AASHTO I Beam	28x90	1	45.26	0.70
10	PCEF Bulb Tee Beam	33x39.25	1	18.5	0.69
11	PCEF Bulb Tee Beam	33x61.25	1	29.51	0.70
12	PCEF Bulb Tee Beam	33x79.25	1	36.93	0.69

(1) $K = .6 + y_b/5h$

The K values in Table 2 also correspond favorably with recent research by Hale et al.[11] where their concluding recommendations state "compression stresses immediately after release be increased from 0.60 f'_{ci} to 0.70 f'_{ci}." This study was based on casting and testing of four I-shaped bridge girders.

THE PennDOT PSLRFD COMPUTER PROGRAM

This parametric study used the PennDOT PSLRFD computer program[7] to analyze the bridges for Methods 1 and 2. This program is commonly used in Pennsylvania to design and analyze prestressed concrete beams from user inputs of various design parameters (see Chapter 5 in Reference 7). For Method 2, the approximate equivalent stress method is used because of its simplicity to use with PSLRFD. The only input file that needs to be revised between Methods 1 and 2 is the *Concrete Material Allowable Properties* (MCA) command where the allowable stresses are specified.

PSLRFD limits the strand eccentricities such that the prestressing force and external moments are less than the allowable compressive and tensile stresses specified in the MCA command. Therefore, revising the allowable compressive stresses will alter the location of the prestressing strands between Methods 1 and 2. To explain, Method 2 will have a different limit kern area and limiting eccentricities than Method 1 because there is a change in the boundary conditions where the prestressing force can be applied. As discussed below in the Results, this is beneficial. The limit kern area is defined as the area of the section within which an axial compressive force of a given magnitude can be placed without violating any of the allowable stresses (tension or compression) [12].

Therefore, with Method 2, the strand eccentricities will be set mainly using the allowable tensile stresses as the primary boundary condition. In fact, for prestressed bridges in Pennsylvania, the service limit tensile stresses are already the controlling criterion in many designs because of PennDOT's allowable tensile stress limits.

SUMMARY OF ALLOWABLE STRESSES BETWEEN METHODS 1 AND 2

Based on the above discussions, the allowable stresses used in Methods 1 and 2 are summarized as follows:

Table 3 – Summary of Allowable Stresses for Parametric Study

Stage	Method 1		Method 2	
	Compression	Tension	Compression	Tension
Initial*	$0.6\ f'_{ci}$	$3\sqrt{f'_{ci}}$	$K\ f'_{ci}$	$3\sqrt{f'_{ci}}$
Intermediate	$0.4\ f'_c$		Neglect	
Final	$0.6\ f'_{ci}$	$3\sqrt{f'_c}$	Neglect	$3\sqrt{f'_c}$

* Immediately after prestress transfer

The tensile stresses are from the PennDOT Design Manual, Part 4[2]. Also, For PennDOT, f'_{ci} is taken as $0.85\ f'_c$.

PARAMETRIC STUDY RESULTS

The following sections discuss the results for each of the bridge typical sections with the pertinent input data and output data presented. Summary discussions are provided that illustrate the differences between the Methods 1 and 2 analyses. The sign convention is per PSLRD[7] where a negative sign is for compression and positive is for tension.

ADJACENT BOX BEAMS (BRIDGES 1, 2 AND 3)

A typical composite, adjacent box beam bridge is shown in Figure 1. The selected span lengths studied for Bridges 1, 2 and 3 are 63 ft, 110 ft, and 140 ft, respectively. The loading and results are presented in Tables 4 and 5.

As the results show in Table 5, the beam designs are primarily controlled by the allowable tensile stresses at the initial and final stage, especially for the shallower beams (Bridges 1 and 2). In fact, there are no appreciable changes in the compressive stresses between the Methods 1 and 2 analyses because the PennDOT allowable tensile stress limits control the strand eccentricities more than the allowable compressive stress limits. Accordingly, the compressive limits are not a factor in these designs.

However, as the span length increases, the depth of the box sections increase and differences in tensile and compressive stresses become apparent. The results for Bridge 3 support this point. As discussed above, the PSLRFD program[7] limits the strand eccentricity to allow the tensile and compressive stresses to be within the allowable stress levels. Using Method 2, the program is adjusting the eccentricities such that the tensile stresses become the primary boundary condition. Therefore, with the aid of a slight increase in the initial compressive stress limit, the tensile stresses can be lowered using Method 2 which allows for slightly longer span lengths. As a result, the 48/60 box beam in Bridge 3 can actually be designed for a maximum span length of 143 ft compared to 142 ft using the conventional analysis.

A review of the Bridge 3 results shows the following trends for adjacent box beams:

- The PSLRFD program[7] uses the controlling tensile stresses in Method 2 as the primary boundary condition to locate the strand eccentricities benefiting from the slight increase in the initial compressive stress limit. In addition, the resulting compressive stresses in Method 2 are not excessive and not a factor in the design.
- The Method 2 final tensile stresses are actually lower than that computed using Method 1.
- The 48/60 spread box beam (Bridge 3) can be designed for a 143 ft span vs. 142 ft.

Table 4 - Input Data for Bridges 1 to 3 for Adjacent Box Beams

Parameters	BRIDGE 1	BRIDGE 2	BRIDGE 3
SPAN LENGTH (L) - ft	63	110	140
TYPE OF BEAM (S)	48x21	48x39	48x60
GIRDER SPACING -ft	4	4	4
CODE (LRFD)	PennDOT	PennDOT	PennDOT
LOADS			
Live Load	PHL - 93	PHL - 93	PHL-93
Impact Factor	1.33	1.33	1.33
Distribution Factor	0.339	0.295	0.281
Deck and Haunch (kip/ft)	0.327	0.327	0.327
Girder (kip/ft)	0.577	0.782	1.012
DC2 (kip/ft)	0.173	0.173	0.173
FWS (kip/ft)	0.097	0.097	0.097
MATERIALS			
Concrete Strength - CIP Slab	4000 psi	4000 psi	4000 psi
Concrete Strength - Precast Bear	8000 psi	8000 psi	8000 psi
Unit Weight of Beam and Slab	150 pcf	150 pcf	150 pcf
Strand Ultimate Strength	270 ksi	270 ksi	270 ksi
Strand Diamater (in^2)	0.52	0.52	0.52
Total Strand Area Aps (in^2)	5.34	8.68	10.02
PRESTRESS LOSSES	PSLRFD	PSLRFD	PSLRFD
ALLOWABLE COMPRESSIVE STRESSES (KSI)			
Conventional - Initial	-4.08	-4.08	-4.08
Conventional -Intermediate	-3.2	-3.2	-3.2
Conventional - Final	-4.8	-4.8	-4.8
Proposed - Initial	-4.76	-4.76	-4.76
Proposed - Intermediate	N/A	N/A	N/A

Table 5 - Results of Method 1 vs. Method 2 for Adjacent Box Beams

Design Results		Bridge 1		Bridge 2		Bridge 3	
		Method 1	Method 2	Method 1	Method 2	Method 1	Method 2
Concrete Strength Req. (psi)							
Initial* (f'ci): (.85 f'c)		6800	6800	6800	6800	6800	6800
Final (f'c):		8000	8000	8000	8000	8000	8000
Stresses (psi)							
Release	top	0.236	0.236	0.262	0.262	0.265	0.263
	bottom	-3.412	-3.412	-4.100	-4.100	-3.762	-3.752
Interm. Stage	top	-2.110	-2.110	-3.140	-3.140	-3.137	-3.140
	bottom	-2.320	-2.390	-2.985	-2.985	-2.844	-2.848
Final Stage	top	-3.040	-3.040	-4.000	-4.000	-3.824	-3.808
	bottom	0.202	0.202	0.258	0.258	0.259	0.249
Number of Strands		32	32	52	52	60	60
c.g.s. from bottom (in)		3.00	3.00	5.23	5.23	8.47	8.33
Mu (kip-ft)		1714.20	1714.20	4667.10	4667.10	7818.70	7818.70
ΦMn (Kip-ft)		2135.30	2135.30	5753.10	5753.10	9844.90	9872.20
Req. Beam Size		48/21	48/21	48/39	48/39	48/60	48/60
Max Span Length for 48/60						142 ft	143 ft

* Immediately after prestress transfer

SPREAD BOX BEAMS – BRIDGES 4 TO 6

A typical section of a spread box beam bridge is shown in Figure 1. The span lengths analyzed for Bridges 4, 5, and 6 are 55 ft, 97 ft, 125 ft respectively. The loading and results are presented in Tables 6 and 7.

For the spread box beams, the PennDOT initial tensile stress limits control the designs for bridges 4 and 5, but the intermediate and final stage stresses (compressive and tensile) are actually lower for Method 2. In addition, Bridge 6 shows a reduction in the number of required strands. Similar to the adjacent box beams (Bridges 1 to 3), the primary reason for these results is the different limiting eccentricities of the prestressing strands between the Methods 1 and 2 analyses.

It is also possible to further increase span lengths for Bridge 6 where the maximum span length is 127 ft using Method 2, but 126 ft for Method 1. Again, Method 2 is aided by using the tensile stresses as the primary boundary condition and slightly increasing the compressive stress limit in the initial stage.

Table 6 - Input Data for Bridges 4 to 6 Spread Box Beams

Parameters	BRIDGE 4	BRIDGE 5	BRIDGE 6
SPAN LENGTH (L) - ft	55	97	125
TYPE OF BEAM (S)	48x24	48x45	48x66
GIRDER SPACING -ft	9.9479	7.9583	7.9583
CODE (LRFD)	PennDOT	PennDOT	PennDOT
LOADS			
Live Load	PHL - 93	PHL - 93	PHL-93
Impact Factor	1.33	1.33	1.33
Distribution Factor	0.702	0.560	0.552
Deck and Haunch (kip/ft)	0.995	0.796	0.796
Girder (kip/ft)	0.631	0.861	1.091
DC2 (kip/ft)	0.260	0.260	0.260
FWS (kip/ft)	0.262	0.210	0.210
MATERIALS			
Concrete Strength - CIP Slab	4000 psi	4000 psi	4000 psi
Concrete Strength - Precast Beam	8000 psi	8000 psi	8000 psi
Unit Weight of Beam and Slab	150 pcf	150 pcf	150 pcf
Strand Ultimate Strength	270 ksi	270 ksi	270 ksi
Strand Diamater (in^2)	0.52	0.52	0.52
Total Strand Area Aps (in^2)	7.01	9.69	11.69
PRESTRESS LOSSES	PSLRFD	PSLRFD	PSLRFD
ALLOWABLE STRESSES (KSI)			
Conventional - Initial	-4.08	-4.08	-4.08
Conventional -Intermediate	-3.2	-3.2	-3.2
Conventional - Final	-4.8	-4.8	-4.8
Proposed - Initial	-4.76	-4.76	-4.76
Proposed - Intermediate	N/A	N/A	N/A
Proposed - Final	N/A	N/A	N/A

The results for the spread box beam trends can be summarized as follows:

- Method 2 allows for revising the strand eccentricities where the tensile stresses are the primary boundary condition. For the spread box beams, the tensile stresses were greater in the initial stage, but *lower* in the final stage than Method 1.
- Slightly longer span lengths can be achieved with Method 2, especially for the deeper beams with span lengths around 125 ft.
- A reduction in strands can be achieved for longer spans with Method 2.
- The Method 2 compressive stresses are not excessive. In some cases, they are *lower* than using Method 1 (tensile stresses are primary boundary condition).

Table 7 - Results of Method 1 vs. Method 2 Analyses for Spread Box Beams

Design Results		Bridge 4 Method 1	Bridge 4 Method 2	Bridge 5 Method 1	Bridge 5 Method 2	Bridge 6 Method 1	Bridge 6 Method 2
Concrete Strength Req. (psi)							
Initial* (f'ci): (.85 f'c)		6800	6800	6800	6800	6800	6800
Final (f'c):		8000	8000	8000	8000	8000	8000
Stresses (psi)							
Release	top	0.228	0.259	0.252	0.256	0.252	0.262
	bottom	-4.000	-4.070	-4.110	-4.180	-4.070	-3.962
Interm. Stage	top	-2.360	-2.240	-2.874	-2.810	-2.910	-2.810
	bottom	-2.902	-2.580	-2.815	-2.740	-2.920	-2.707
Final Stage	top	-2.730	-2.616	-3.350	-3.270	-3.440	-3.340
	bottom	0.262	0.197	0.267	0.222	0.270	0.267
Number of Strands		42	42	58	58	70	68
c.g.s. from bottom		4.21	3.83	6.98	6.5	10.73	9.68
Mu (kip-ft)		2141.20	2141.20	6349.20	6349.20	10423.00	10423.00
Mn (Kip-ft)		3559.20	3615.70	8160.70	8259.50	13755.90	13717.40
Req. Beam Size		48/24	48/24	48/45	48/45	48/66	48/66
Max Span Length 48/66						126	127

* Immediately after prestress transfer

PA I BEAMS AND AASHTO I BEAMS – BRIDGES 7 TO 9

These bridges consider both continuous and simple spans. Bridges 7 and 8 are continuous for live load and superimposed dead load and Bridge 9 is a simple span. The loading and results are presented in Tables 8 and 9.

Bridge 7 – Continuous PA I Beam Bridge:

There is an appreciable difference in results for this continuous bridge between Methods 1 and 2. To illustrate, Table 7 shows the compressive stresses for Method 2 are much higher than Method 1 because additional strands can be placed in the beam. These stresses are especially higher at the initial stage and the final stage (bottom of beam) over the pier. The additional strands in Method 2 increase the moment capacity of the section and allows for a shallower beam depth. This can be beneficial on many bridge projects. For Bridge 7, the Method 1 analysis designs a beam with a 36 inch beam depth while the Method 2 analysis achieves a shallower depth of 33 inches.

As mentioned above, there are concerns that eliminating the intermediate and final stage compressive limits will result in high compressive forces and possible long term creep failure for continuous bridges, especially for the prestress plus permanent dead load case. For this bridge, that concern does not appear to be evident. To explain, the bottom compressive stress at the pier for the intermediate stage increases from -3.18 ksi or -0.4 f'_c (Method 1) to -3.61 ksi or -0.45 f'_c (Method 2) in the intermediate stage. For the final stage stresses, the compressive stress at the bottom of beam at the pier increased from -4.26 ksi or-0.53 f'_c (Method 1) to -4.75 ksi or -0.59 f'_c (Method 2). These increases at the intermediate and final stages are tolerable and still within the AASHTO LRFD[1] compressive stress limits.

Although the Method 2 analysis has an increase in the number of strands, the section is still under-reinforced. This ensures the yielding of the steel reinforcement before crushing of the concrete due to factored dead and live loads. The compressive stress increase in Method 2 is also limited because the strands cannot exceed the PennDOT allowable tensile limits in the initial stage of $3\sqrt{f'_{ci}}$. This limit is, in effect, a major restriction in the number of strands that can be placed in the beam that helps to prevent excessive compressive stress increases in the intermediate and final stages.

From Table 9, a summary of these results are:

- The compressive stresses appreciably increase for this continuous structure, especially in the bottom of beam over the pier. The increased compressive stresses were tolerable.
- The compression is still held to an acceptable range due to the initial allowable tensile stress boundary condition.
- The increased initial compression limits allow more strands to be placed in the beam that increases moment capacity. This allows for a shallower beam depth using Method 2.

<u>Bridge 8 – Continuous AASHTO I Beam Bridge</u>

Bridge 8 considers a continuous 5 span AASTHO I beam bridge using 28x66 beams for the conventional design. The span lengths are 120.75 ft, 121.75 ft, 121.75 ft, 121.75 ft and 120.75 ft, respectively. The results are similar to Bridge 7 where there is an appreciable increase in the compressive stresses during all loading stages. However, they are not excessive and the final tensile stress is lower using the Method 2 analysis. Similar to Bridge 7, the Method 2 allows more strands to be placed in the beam. This provides benefits during the subsequent stages because the additional strands increase moment capacity and results in a shallower structure depth. The summary for this bridge is similar to Bridge 7 including the concerns for excessive compressive stresses are not evident. The results are shown in Table 9.

<u>Bridge 9 – Simple Span AASHTO I Beam</u>

Bridge 9 considers a simple span 28x90 AASHTO I beam bridge with a 143 ft span length. The Method 2 results in a small increase to the compressive stresses, but they are not excessive because the beam design is controlled by the final tensile stress. Accordingly, there is no change in the number of strands or eccentricity using Method 2. The results appear similar to Bridge 1 and 2 where the compressive stresses are not a factor in the design.

Table 8 Input Data for Bridges 7 to 9

Parameters	BRIDGE 7	BRIDGE 8	BRIDGE 9
SPAN 1 LENGTH (S) - ft	67.28	120.75	143
SPAN 2 LENGTH (S) - ft	66.64	121.75	
SPAN 3 LENGTH (S) - ft	67.52	121.75	
SPAN 4 LENGTH (S) - ft		121.75	
SPAN 5 LENGTH (S) - ft		120.75	
BEAM DEPTH(S) - in	30 to 36	90 TO 96	90 to 96
NO. OF SPANS	3	5	
GIRDER SPACING -ft	8.01	8.1979	9.25
CODE (LRFD)	PennDOT	PennDOT	PennDOT
LOADS			
Live Load	PHL - 93	PHL - 93	PHL-93
Impact Factor	1.33	1.33	1.33
Distribution Factor			
Span 1	0.702	0.774	0.949
Span 2	0.702	0.773	
Span 3	0.702	0.765	
Span 4		0.773	
Span 5		0.774	
Deck and Haunch (kip/ft)	0.810	0.820	0.881
Girder (kip/ft)	0.811	1.080	1.280
DC2 (kip/ft)	0.260	0.260	0.745
FWS (kip/ft)	0.205	0.228	0.000
MATERIALS			
Concrete Strength - CIP Slab	4000 psi	4000 psi	4000 psi
Concrete Strength - Beam	8000 psi	8000 psi	8000 psi
Unit Weight of Beam and Slab	150 pcf	150 pcf	150 pcf
Strand Ultimate Strength	270 ksi	270 ksi	270 ksi
Strand Diamater (in^2)	0.52	0.52	0.52
PRESTRESS LOSSES	PSLRFD	PSLRFD	PSLRFD
ALLOWABLE STRESSES (KSI)			
Conventional - Initial	-4.08	-4.08	-4.08
Conventional -Intermediate	-3.2	-3.2	-3.2
Conventional - Final	-4.8	-4.8	-4.8
Proposed - Initial	-4.76	-4.76	-4.76
Proposed - Intermediate	N/A	N/A	N/A
Proposed - Final	N/A	N/A	N/A

Table 9 Results of Conventional vs. Proposed Analyses for Bridges 7 to 9

Design Results		BRIDGE 7		BRIDGE 8		BRIDGE 9	
		Method 1	Method 2	Method 1	Method 2	Method 1	Method 2
Concrete Strength Req. (psi)							
Initial* (f'ci):		6800	6800	6800	6800	6800	6800
Final (f'c):		8000	8000	8000	8000	8000	8000
Max Stresses (psi)							
Release	top	0.261	0.266	0.264	0.265	-0.054	0.220
	bottom	-4.110	-4.740	-4.129	-4.743	-4.050	-4.388
Interm. Stage	top	-2.400	-3.092	-2.238	-2.398	-2.133	-2.133
	bottom	-3.180	-3.610	-3.235	-3.467	-3.121	-3.386
Final Stage - @ pier	bottom	-4.260	-4.750	-4.226	-4.597		
midspan	bottom	0.272	0.268	0.272	0.268	0.258	0.258
	top	-3.015	-3.687			-2.841	-2.841
Number of Strands - Span 1		36	55	72	77	86	86
Number of Strands - Span 2		42	48	72	78		
Number of Strands - Span 3		44	52	78	78		
Number of Strands - Span 4				72	78		
Number of Strands - Span 5				73	78		
Strand Area (in^2) Span 1		7.35	9.18	12.02	12.86	14.36	14.36
Strand Area (in2) Span 2		8.02	8.02	12.02	13.03		
Strand Area (in2) Span 3		7.35	8.68	13.03	13.03		
Strand Area (in2) Span 4				12.02	13.03		
Strand Area (in2) Span 5				12.19	13.03		
Mu (kip-ft) Span 1		3572.00	3473.50	11309.30	11197.80	18646.80	18646.80
Mu (kip-ft) Span 2		3457.70	3394.40	11387.00	11275.10		
Mu (kip-ft) Span 3		3589.10	3556.30	11275.10	11275.10		
Mu (kip-ft) Span 4				11413.90	11301.90		
Mu (kip-ft) Span 5				11477.10	11365.60		
Mn (Kip-ft) Span 1		5203.10	5482.80	14664.90	14894.60	24938.70	24938.70
Mn (Kip-ft) Span 2		5116.60	5131.40	14761.50	15005.40		
Mn (Kip-ft) Span 3		5218.00	5400.00	15005.40	15005.40		
Mn (kip-ft) Span 4				14746.70	15072.10		
Mn (kip-ft) Span 5				15013.20	15116.60		
Req. Beam Size - Span 1		24/36	24/33	28/66	28/63	28/90	28/90
Req. Beam Size - Span 2		26/33	24/33	28/66	28/63	28/90	28/90
Req. Beam Size - Span 3		24/36	26/33	28/63	28/63	28/90	28/90
Req. Beam Size - Span 4		N/A	N/A	28/66	28/63	28/90	28/90
Req. Beam Size - Span 5		N/A	N/A	28/66	28/63	28/90	28/90

* Immediately after prestress transfer

PCEF BULB TEE BEAMS – BRIDGES 10 TO 12

This study considers beam depths of 39.25 inches, 61.25 inches, and 79.25 inches, respectively, for bridges 10, 11, and 12. Bridges 10 and 11 utilize a straight, debonded strand design, and Bridge 12 uses a draped strand design. The input data and results are located in Tables 10 and 11.

These results show the span lengths can be increased 3 to 4 feet using the Method 2 analysis for bridges 10 and 11. The slight increase in the initial stage compressive limit is a major factor for these results. This is because additional strands can be placed in the beams. Similar to Bridges 7 and 8, the resulting increase in moment capacity allows for increasing the span lengths for Method 2. The final compressive stresses increase, but are not excessive compared to the Method 1 analysis approach. For example, the final stage compressive stresses at the top of beam for Bridge 10 increases from -3.149 ksi or -0.4 f'_c (Method 1) to -3.829 ksi or -0.48 f'_c (Method 2). It should be noted the AASHTO LRFD code limit for the final stage compressive stress is 0.6 f'_c.

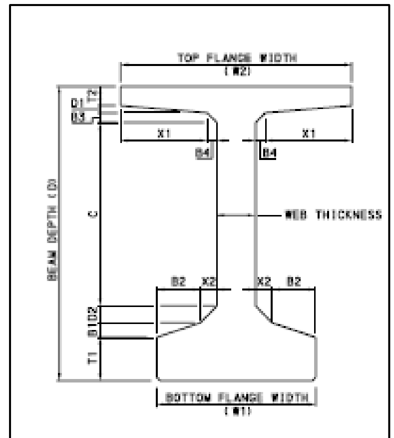

Figure 3 –Bulb Tee Beam[13]

A summary of the Bulb Tee trends are as follows:

- A slight increase in the initial compressive stress limit allows for the placement of additional strands for these sections in Method 2 compared to Method 1
- The additional strands increase moment capacity and allow span lengths to be extended up to four feet using Method 2.
- The intermediate and final compressive stresses using Method 2 are acceptable.

Table 10 Input Data for Bridges 10 to 12

Parameters	BRIDGE 10	BRIDGE 11	BRIDGE 12
TYPE OF BEAM (S)	33x39.25	33x61.25	33x79.25
GIRDER SPACING -ft			
CODE (LRFD)	PennDOT	PennDOT	PennDOT
LOADS			
Live Load	PHL - 93	PHL - 93	PHL-93
Impact Factor	1.33	1.33	1.33
Distribution Factor	0.825	0.976	1.175
Deck and Haunch (kip/ft)	0.669	0.769	1.119
Girder (kip/ft)	0.813	0.944	1.146
DC2 (kip/ft)	0.325	0.325	0.325
FWS (kip/ft)	0.213	0.255	0.319
MATERIALS			
Concrete Strength - CIP Slab	4000 psi	4000 psi	4000 psi
Concrete Strength - Precast Beam	8000 psi	8000 psi	8000 psi
Unit Weight of Beam and Slab	150 pcf	150 pcf	150 pcf
Strand Ultimate Strength	270 ksi	270 ksi	270 ksi
Strand Diamater (in)	0.52	0.52	0.52
PRESTRESS LOSSES	PSLRFD	PSLRFD	PSLRFD
ALLOWABLE STRESSES (KSI)			
Conventional - Initial	-4.08	-4.08	-4.08
Conventional -Intermediate	-3.2	-3.2	-3.2
Conventional - Final	-4.8	-4.8	-4.8
Proposed - Initial	-4.76	-4.76	-4.76
Proposed - Intermediate	N/A	N/A	N/A

Table 11 Results of Method 1 vs. Method 2 Analyses for Bridges 10 to 12

Design Results		Bridge 10		Bridge 11		Bridge 12	
		Method 1	Method 2	Method 1	Method 2	Method 1	Method 2
Concrete Strength Req. (psi)							
Initial* (f'ci): (.85 f'c)		6800	6800	6800	6800	6800	6800
Final (f'c):		8000	8000	8000	8000	8000	8000
Stresses (psi)							
Release	top	0.259	-0.168	0.261	-0.152	-0.127	0.222
	bottom	-4.127	-4.739	-4.120	-4.735	-4.052	-4.599
Interm. Stage	top	-2.277	-2.898	-2.272	-2.809	-2.370	-2.455
	bottom	-2.723	-3.341	-2.910	-3.429	-3.071	-3.475
Final Stage	top	-3.149	-3.829	-3.160	-3.733	-3.209	-3.304
	bottom	0.266	0.266	0.266	0.269	0.265	0.262
Number of Strands		63	79	72	86	85	88
Total Strand Area (in^2)		10.52	13.19	12.02	14.36	14.19	14.7
c.g.s. from bottom		6.87	8.77	9.78	13.23	9.87	10.48
Mu (kip-ft)		6112.10	6622.00	10990.70	11465.70	16900.10	17111.60
Mn (Kip-ft)		7660.50	7961.70	13658.50	14156.10	21718.40	22037.90
Req. Beam Size							
Max Span Length (ft)		86	90	110	113	126	127

* Immediately after prestress transfer

CONCLUSION AND RECOMMENDATIONS

From this study, it appears Prestressed beams can be designed by using the Method 2 analysis as long as the tensile stresses are limited to $3\sqrt{f'_c}$. By eliminating the arbitrary compressive stress limits and applying research from the University of Nebraska to PennDOT prestressed beams (increasing the compressive stress limits at the initial stage and elimination of these limits at the final stage), slightly longer span lengths and shallower beam depths will result especially in continuous structures and the bulb tee beams.

Some compressive stresses were higher from using Method 2 compared to Method 1. These increases were especially evident for the continuous beams (Bridges 7 and 8) and the Bulb Tees (Bridges 10 to 12). However, as long as the tensile stresses are limited to $3\sqrt{f'_c}$, the stresses were acceptable such that the non-linear effects of possible long term creep failure would appear not to be a concern. This is primarily because the final compressive stresses using Method 2 were still close to the current AASHTO LRFD[1] code limits. There are reasons for this; (1) the PennDOT initial tensile limits prevented an excessive number of additional strands to be placed in the beams, and (2) the slight increase in the initial compressive stress immediately after prestress transfer can be tolerated as shown in current research without subsequent excessive stress increases in the intermediate and final stages.

A main difference with Method 2 compared to Method 1 is that the initial and final tensile limits become the primary boundary condition in Method 2. In essence, the prestressing strands were placed to primarily control tensile stresses. The resulting compressive stress increases were still not excessive.

The main advantages associated with Method 2 are as follows:

- Slightly longer span lengths were achieved, especially with the bulb tee beam bridges (Bridge 10 to 12) and longer span spread box beam bridges.
- Shallower beam depths were achieved for continuous structures.
- The final stage tensile stresses were lower for spread box beam bridges mainly due to the change in the strand eccentricities between Methods 1 and 2.
- Slightly fewer strands were required for the longer span spread box beam bridges using Method 2.

RECOMMENDATIONS

Recommendations for bridges typically used in Pennsylvania would include:

1) Increase the initial allowable compressive stress limit from 0.6 f'_{ci} to 0.7 f'_{ci}. Based on current research and this study, it appears this increase can be tolerated in the design of prestressed beams. The "*Approximate Equivalent Allowable Stress Method*" would be a justified approach to establish the initial compressive stress limit.

2) Eliminate the intermediate and final stage compressive stress limits. They had no effect in the box beam bridges and the initial PennDOT tensile stress limit will still minimize the number of strands that can be placed in the beams so that there will be no appreciable increase in these compressive stresses.

3) If intermediate and final stage compressive stresses are excessive for continuous bridges (ie. they are much greater than the current AASHTO code limits), consider a time-dependent analysis to determine stresses (non-linear analysis). Usually, with this approach the compressive stresses are lower than as shown for a linear elastic analysis. A recommended procedure can be found in References 3 and 14.

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