EVALUATION OF COMMON DESIGN POLICIES FOR PRECAST PRESTRESSED I-GIRDER BRIDGES

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ABSTRACT

The AASHTO LRFD Bridge Design Specifications provide bridge engineers with the minimum design requirements for safe highway bridge structures. However, many bridge owners have adopted more stringent policies for the design of precast-prestressed girder bridges. These policies include some combination of designing with gross or transformed section properties, reduced allowable tensile stress, and simple span moments for superimposed dead and live loads. This study attempts to quantify the sensitivity of these common policies on the design of precast-prestressed bridge girders. The most common disadvantages of more stringent policies include reduction in span capability, reduced girder spacing or additional lines of girders, or an increase in prestressing levels.

Keywords: Design, Girders, Criteria, Sensitivity, Research, LRFD

INTRODUCTION

The AASHTO LRFD Bridge Design Specifications[1] (AASHTO LRFD) provide bridge engineers with the minimum design requirements for safe highway bridge structures. However, many bridge owners have adopted more stringent policies for the design of precast-prestressed girder bridges. These policies consist of some combination of design using gross or transformed section properties, reduced allowable tension stress under service loads, and the full envelope of simple span positive moments and continuous span negative moments for spans made continuous for superimposed dead and live loads.

Bridges designed using more stringent policies will obviously be stouter and more costly when compared to bridges designed only to the minimum requirements of AASHTO LRFD. The most common differences include some combination of a reduction in span length, reduced girder spacing or additional lines of girders, or an increase in prestressing levels. This study attempts to quantify the sensitivity of common policies on the design of precast-prestressed bridge girders. Span capability, girder spacing, and prestressing requirements are computed based on the minimum requirements set forth in AASHTO LRFD. Each of the more stringent policies is then evaluated individually to understand its effect on the design. The combined effect of all the design policies is also investigated.

In the author’s opinion, the real cost savings to be considered when evaluating agency design policies derive from extending spans (reducing the number of piers) and/or reducing the number of girder lines. Generally, the number of prestressing strands in a typical bridge girder does not significantly influence the overall cost of the bridge, unless it exceeds the capacity of the local precasting industry. It should also be noted that higher prestressing levels require increased concrete release and shipping strengths, which could adversely affect production schedules, handling, shipping and ultimately cost.

A key component to evaluating design policies is to first establish local precasting capabilities with respect to maximum jacking capacity, concrete strengths at release and shipping, and handling and shipping weight or length limitations. As long as these limits are not exceeded, the agency is free to design the most economical solution within their established design policies. Material technology and plant capabilities are constantly evolving and improving. Evaluating design policies can provide valuable insight into where improved capabilities or altered design policies can be of most benefit.

SURVEY OF DESIGN POLICIES

A survey of state Departments of Transportation (DOTs) was conducted to gauge the extent to which bridge owners deviate from the minimum requirements set forth in AASHTO LRFD. Bridge owners were asked the following questions:

1. What type of section properties does your state use for the design of precast-prestressed girder bridges (gross, transformed)?

2. What allowable tension stress policy does your state have for the design of precast-prestressed girder bridges (AASHTO LRFD Table 5.9.4.2.2-1, zero tension at service limit state, other)?
3. What continuity policy does your state have for the design of continuous precast-prestressed girder bridges (AASHTO LRFD Section 5.14.1.4 “Bridges Composed of Simple Span Precast Girders Made Continuous”, simple span moments for superimposed dead loads and live load, other)?
4. What prestress loss policy does your state have for design of precast-prestressed girder bridges (AASHTO LRFD Section 5.9.5.3 “Approximate Estimate of Time Dependent Losses”, AASHTO LRFD Section 5.9.5.4 “Refined Estimates of Time Dependent Losses”, other)?

A total of 38 state DOTs responded to the survey, and their responses to questions 1 through 3 are summarized in Figs. 1 through 3. The responses to question 4 are not applicable to this study because the influence of prestress loss methods are not examined as explained later.

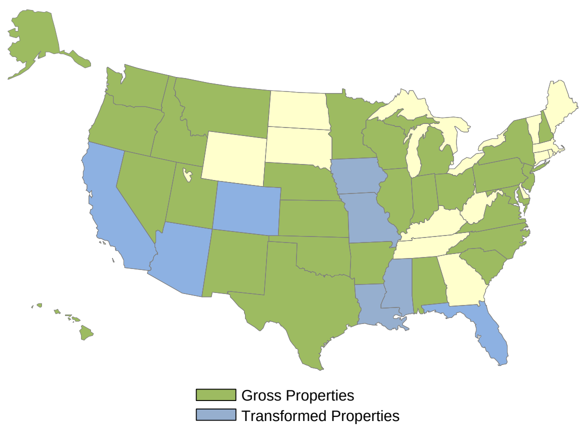

Figure 1 Survey results for Section Properties Design Policy

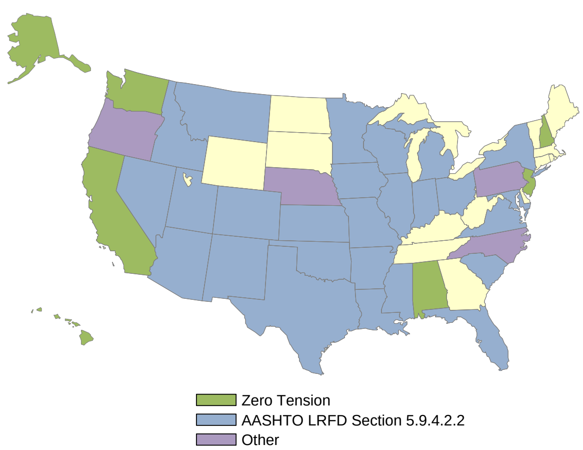

Figure 2 Survey results for Allowable Tension Design Policy

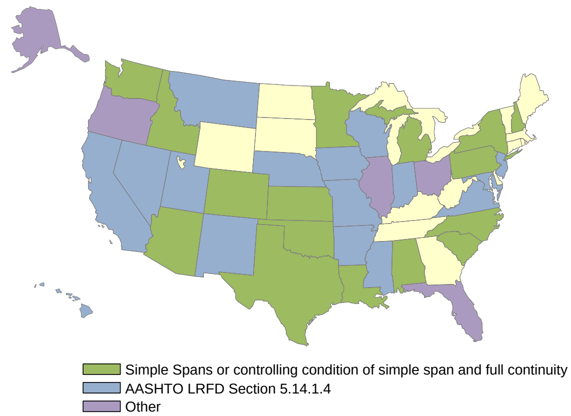

Figure 3 Survey results for Continuity Design Policy

RESEARCH APPROACH

The bridge sections used in this study are a slab-on-girder system composed of a cast-in-place deck on precast wide flange (WF) I-girder elements. The effect of the design policies considered in this study are determined by analyzing a typical interior girder for various bridge configurations consisting of six Washington State Department of Transportation (WSDOT) WF-Series precast girders with baseline girder spacing of 6 feet and 12 feet. The bridge deck is assumed to be 7.5 inches and 9.5 inches thick, for girder spacing of 6 feet and 12 feet, respectively. The haunch build up is assumed to be 3 inches thick. Typical bridge sections are shown in Figs. 4 and 5.

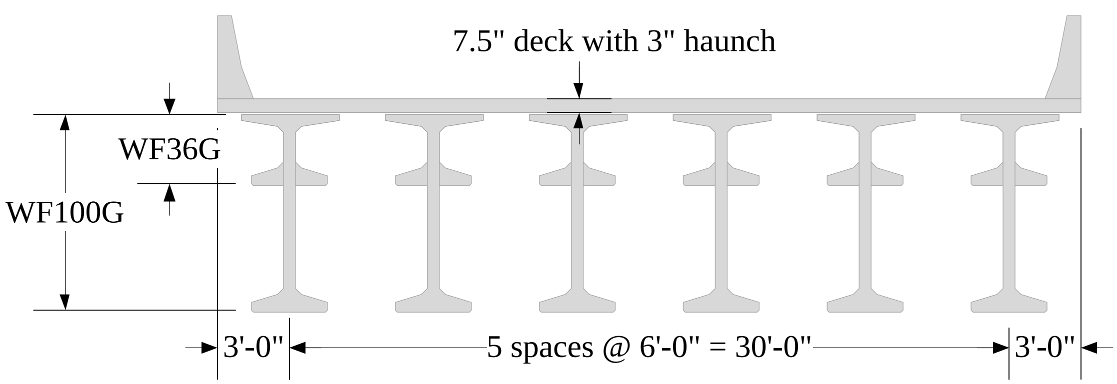

Figure 4 Bridge Section with 6 ft girder spacing.

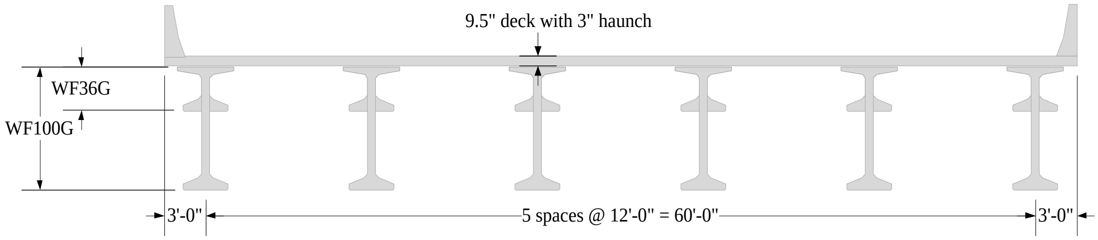

Figure 5 Bridge Section with 12 ft girder spacing.

Precast-prestressed girder bridges that are simple span for girder and deck dead loads and made continuous for superimposed dead loads and live loads are analyzed. The span configuration studied consists of two spans of equal length. The maximum positive moment under uniform load occurs approximately 60% of the span length from the interior support[2,3]. Design evaluations are performed at this location.

When evaluating owner adopted policies that do not account for continuity, a simple span bridge is used. The design evaluations are performed at mid-span.

WSDOT WF-Series girders are shown in Figure 6. Gross section properties for the WF-Series girders are given in Table 1. Transformed section properties for the WF-Series girders, with one level of prestressing per girder size, are given in Table 2. Transformed section properties depend on the number and location of prestressing strands in the section considered, and on the concrete modulus of elasticity at the time considered. The properties shown are at midspan at 28 days.

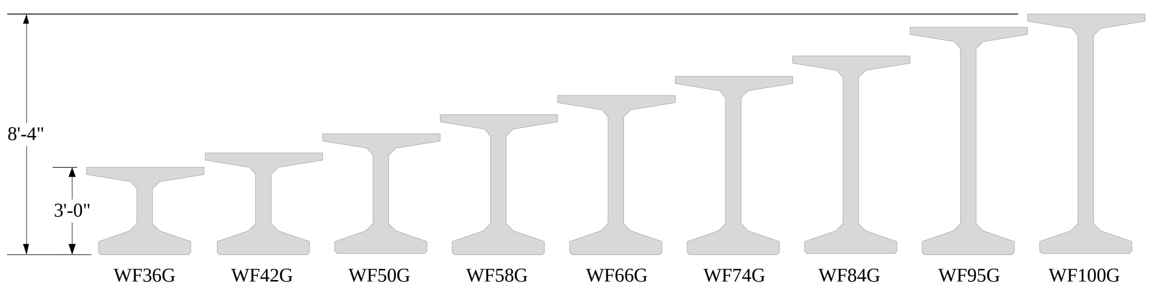

Figure 6 WSDOT WF-Series Girders

Table 1 WF-Series Girder Gross Section Properties

Girder	Height (in)	Area (in^2)	Y_b (in)	Y_t (in)	I (in^4)	S_b (in^3)	S_t (in^3)
WF36G	36	691	17.5	18.5	124772	7115	6758
WF42G	42	728	20.4	21.6	183642	9020	8486
WF50G	50	777	24.2	25.8	282559	11700	10931
WF58G	58	826	28.0	30.0	406266	14527	18637
WF66G	66	875	31.8	34.2	556339	17493	16269
WF74G	74	824	35.7	38.3	734356	20595	19153
WF83G	82.625	976	39.8	42.8	959396	24088	22418
WF95G	94.5	1049	48.9	45.6	1328995	29148	27175
WF100G	100	1083	48.3	51.7	1524912	31589	29480

Table 2 WF-Series Girder Transformed Section Properties at mid-span for f'_c = 9.0 ksi

Girder	0.6" Diameter Strands	A_{ps} (in^2)	Area (in^2)	Y_b (in)	Y_t (in)	I (in^4)	S_b (in^3)	S_t (in^3)
WF36G	50	10.9	731	16.8	19.2	126110	7487	6583
WF42G	50	10.9	768	19.6	22.4	185125	9468	8247
WF50G	55	11.9	821	23.1	26.9	284501	12308	10582
WF58G	55	11.9	870	26.8	31.2	408508	15245	13091
WF66G	60	13.0	923	30.4	35.6	559278	18398	15709
WF74G	60	13.0	872	34.0	40.0	737951	21733	18428
WF83G	65	14.1	1029	38.1	44.6	963784	25327	21623
WF95G	70	15.2	1106	46.7	47.8	1335683	28624	27921
WF100G	70	15.2	1139	46.1	53.9	1531307	33196	28425

The material properties used in this study are given in Table 3. The final concrete strength of 9.0 ksi is used for this study because it correlates to an allowable tensile stress of $0.19\sqrt{f_c'} = 0.570\ ksi$, which is roughly the average allowable tensile stress when considering concrete strengths between 4.0 ksi and 15.0 ksi.

Table 3 Material Properties

Girder Release Strength	$f_{ci}' = 7.0\ ksi$
Girder 28-day Strength	$f_c' = 9.0\ ksi$
Deck 28-day Strength	$f_c' = 4.0\ ksi$
Concrete Density for computing dead load	*0.165 kcf*
Concrete Density for computing modulus of elasticity	*0.155 kcf*
Prestressing strand	0.6" diameter, Grade 270, Low Relaxation Strand
Superimposed Dead Loads (Traffic Barrier)	*0.100 kip/ft/girder*

The following assumptions and simplifications are used in the analysis:

- Centroid of effective prestress force is 5 inches above the bottom of the girder
- All strands are located at the centroid of the effective prestress force for computing transformed section properties
- Deck has a one-half inch sacrificial wearing surface
- Continuity is established when the girder age is at least 90 days so that restraint moment does not need to be computed as specified by AASHTO LRFD Section 5.14.1.4.4.

The baseline for comparing the design policies considered in this study is established by analyzing the bridge configurations described above and determining the maximum span length that satisfies the allowable tension requirements for various levels of prestressing. The baseline analysis is performed using transformed section properties, full continuity for superimposed dead and live loads, and an allowable tensile stress of 0.570 ksi. The baseline span capability and prestressing level is shown in Table 4.

Table 4: Design comparison baseline

		Span Capability (ft)	
Girder	0.6" Diameter Strands	6 ft Spacing	12 ft Spacing
WF36G	50	125.47	100.11
WF42G	50	137.80	110.20
WF50G	55	158.71	127.48
WF58G	55	171.72	138.54
WF66G	60	190.40	154.37
WF74G	60	201.09	163.79
WF83G	65	219.01	179.31
WF95G	70	239.77	197.62
WF100G	70	245.49	202.91

To determine how the design policies effect span capability, the girder spacing and prestressing levels from the baseline analysis are held constant and the maximum span length that satisfies the allowable tension requirements is determined using each owner adopted policy individually as well as all three of the policies taken together. It is assumed that the design is governed by the allowable concrete tensile stress under service loads, which is normally the case. Next the span capabilities and the amount of prestressing from the baseline analysis are held constant and the girder spacing that satisfies the allowable tension requirements using the owner adopted policies is determined. Finally, the span capabilities and girder spacing from the baseline analysis are held constant and the prestressing level satisfying the allowable tension requirements for the owner adopted policies are computed.

DESIGN EQUATIONS

Design equations for precast-prestressed girders are well known and are provided in the available literature[4,5,6]. A brief summary is given here to describe the relationship between the governing stress condition and the parameters that are varied in this study (span length, girder spacing, and amount of prestressing).

The design of prestressed girders is generally governed by tension in the precompressed tensile zone at the Service III limit state. The final state of stress in the girder must satisfy Equation 1 which requires that the sum of the stresses in the precompressed tensile zone caused by externally applied loads and the internal pretension force must be less than or equal to the allowable tension stress.

$$f_b - \frac{P_e}{A} - \frac{P_e e_{ps}}{S_b} \leq K_t \sqrt{f_c'} \qquad (1)$$

Prestress losses are computed in accordance with AASHTO LRFD Section 5.9.5. The total prestress loss is given by AASHTO LRFD Equation 5.9.5.1-1, and is the sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads and losses due to long-term shrinkage and creep of concrete, and relaxation of the prestressing steel.

Losses due to elastic shortening are dependent on span length and the amount of prestressing. Gains and losses due to external loads are dependent on span length and girder spacing.

AASHTO LRFD provides refined and approximate methods for estimating time-dependent prestress losses due to shrinkage and creep of concrete, and relaxation of the prestressing steel. The approximate method was calibrated to slab-on-girder bridge systems using I-girder beam elements[7]. Because the approximate method yields essentially the same result as the refined method for the bridge system considered in this study, a comparison of prestress loss policies is not necessary. The approximate method depends on the level of prestressing while the refined method depends on all the variables considered in this study. The approximate method of predicting prestress losses is used because of its simplicity and its excellent correlation with the refined method.

The design moment is a function of the self-weight dead load, superimposed dead load, and live load moments. All of these moments are a function of span length. The slab dead load and the superimposed dead load moments are also a function of girder spacing.

The bridge sections used in this study conform to the requirements for a type k section shown in AASHTO LRFD Table 4.6.2.2.1-1. The live load distribution for moment in interior girders is given in AASHTO LRFD Table 4.6.2.2.2b-1. The live load distribution factors were developed based on gross section properties[8]. The distribution of live load to a girder is dependent on both span length and girder spacing.

The girder stresses are computed from geometric properties including cross sectional area, centroid, moment of inertia, and section modulus. These geometric properties are a function of the amount and location of reinforcement when transformed section properties are used. Composite section properties depend on the girder spacing.

SECTION PROPERTIES POLICY

AASHTO LRFD Section 5.9.1.4 permits section properties to be based on either gross or transformed sections. Transformed section properties are theoretically correct; however they are much more cumbersome to compute than gross section properties. Transformed section properties vary along the length of the girder due to the varying position of harped strands. They also vary as the number of strands change in iterative design trials.

Designing precast-prestressed bridge girders with gross section properties is much easier. Gross section properties require less effort to compute, they are constant over the length of a girder, and they do not change during design iterations. Designing with gross section properties is a common practice that is used by 76% of the responding bridge owners surveyed.

Gross section analysis will result in a design that is more conservative than designs using transformed properties. The centroid of a gross section is further from the bottom of the girder than the centroid of a transformed section (assuming the centroid of the strands is below the centroid of the gross concrete section). The moment of inertia and the bottom section modulus of the gross section are less than the corresponding properties of a transformed section. The final tension stresses predicted using a gross section analysis will be larger when compared to the prediction using a transformed section analysis.

Figure 7 shows the span capabilities of the WF66G girder at 6 feet and 12 feet spacing for various levels of prestressing. The span capability curves for the gross section analysis are to the left of the curves for the baseline analysis indicating a reduction in span capability when a gross section analysis is used. The reduction in span capability increases as the amount of prestressing increases.

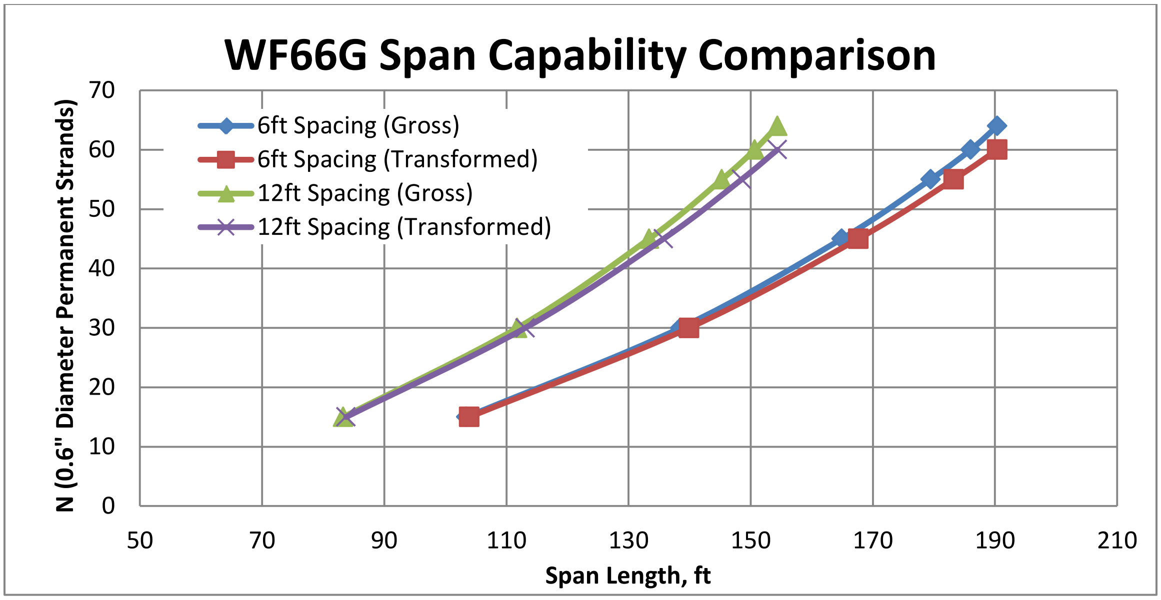

Figure 7 WF66G span capability comparison for section properties policy

Figure 8 compares the baseline girder spacing for a WF66G girder to the girder spacing required for a gross section analysis. A narrower girder spacing results when a gross section analysis is used. This is more pronounced for longer span lengths because a higher level of prestressing is utilized which increases the bottom section modulus of a transformed section and reduces the calculated bottom fiber stresses.

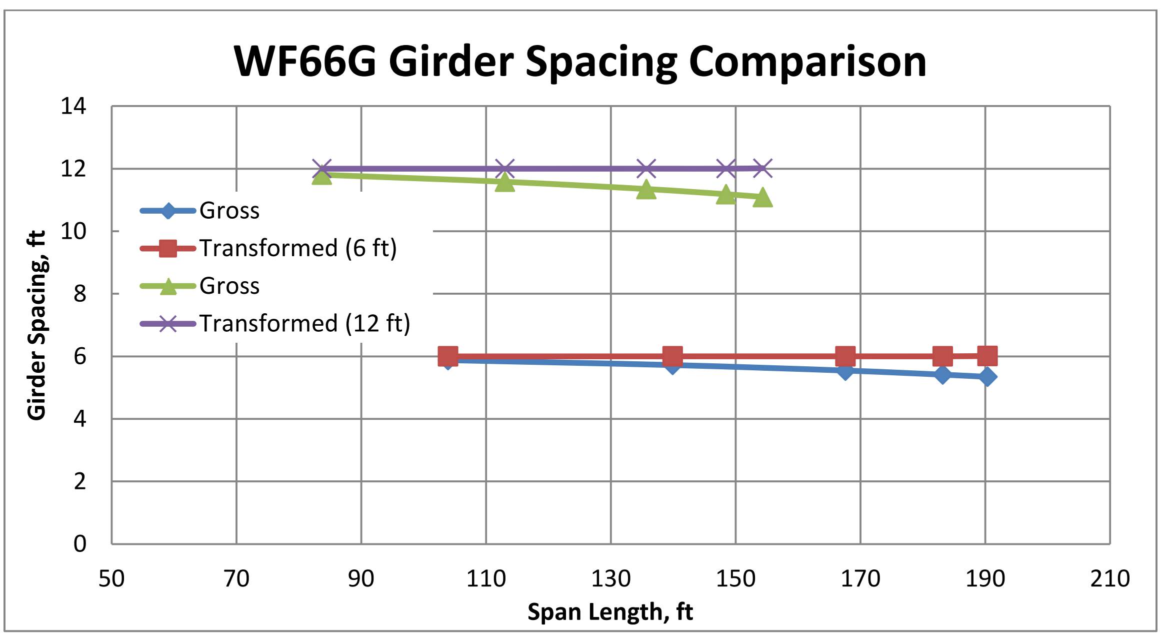

Figure 8: WF66G girder spacing comparison for section properties policy

ALLOWABLE TENSION POLICY

AASHTO LRFD Section 5.9.4.2.2 limits the tension stress in the precompressed tensile zone for the Service III limit state to $0.19\sqrt{f_c'}$ for no worse than moderate corrosion conditions and $0.0948\sqrt{f_c'}$ when subjected to severe corrosive conditions. Some bridge owners have design policies that further restrict the tensile service stresses. 18% of the bridge owners that responded to the survey allow no tension in the precompressed tensile zone at the Service III limit state.

Figure 9 compares span capabilities of the WF66G girder for the baseline analysis and an analysis using an allowable tension stress of 0.0 ksi. The span capability curves for the 0.0 ksi allowable tension stress are to the left of the baseline curves indicating a reduction in span capability. The reduction in span capability decreases as the span length increases due to the higher level prestressing at the longer span lengths.

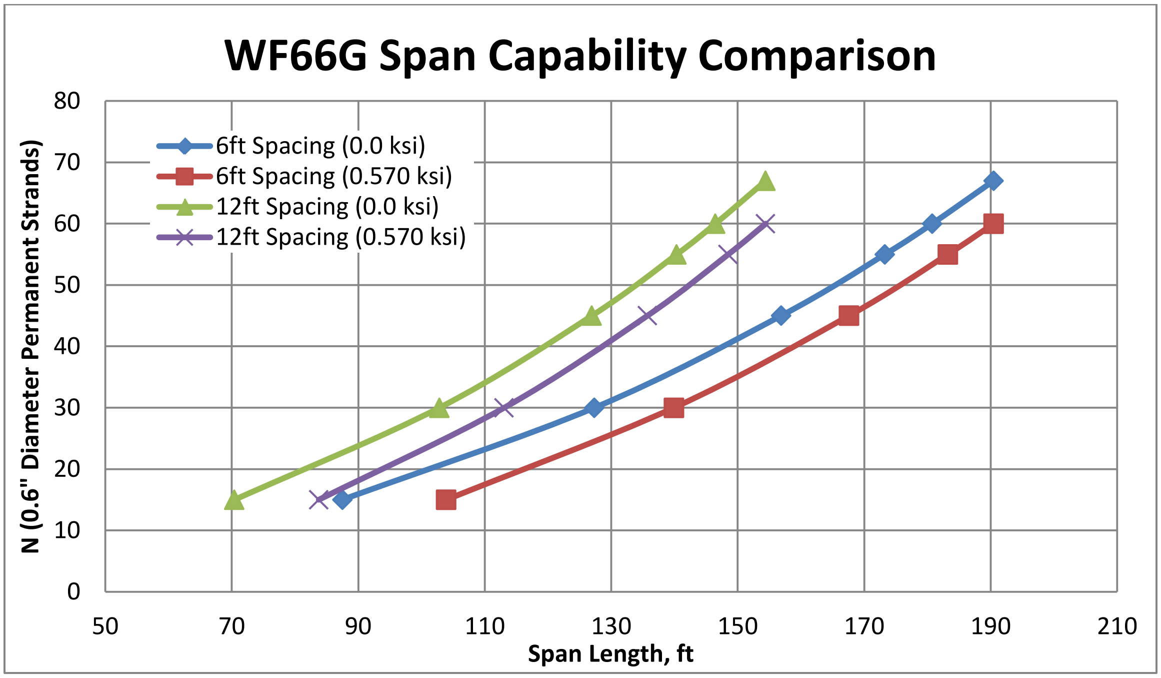

Figure 9 WF66G span capability comparison for allowable tension policy

Figure 10 compares the baseline girder spacing for a WF66G girder to the girder spacing required to satisfy Equation 1 for an allowable tension stress of 0.0 ksi. The girder spacing must be decreased by a larger amount at the smaller span lengths due to the lessor amount of prestressing used in these configurations. In several cases, the required girder spacing is less than the top flange width of a WF-Series girder, which is 49 inches. Bridge configurations that require a girder spacing less than the top flange width cannot be constructed and are thus unattainable. This situation is remedied by increasing the prestressing or using a larger girder section.

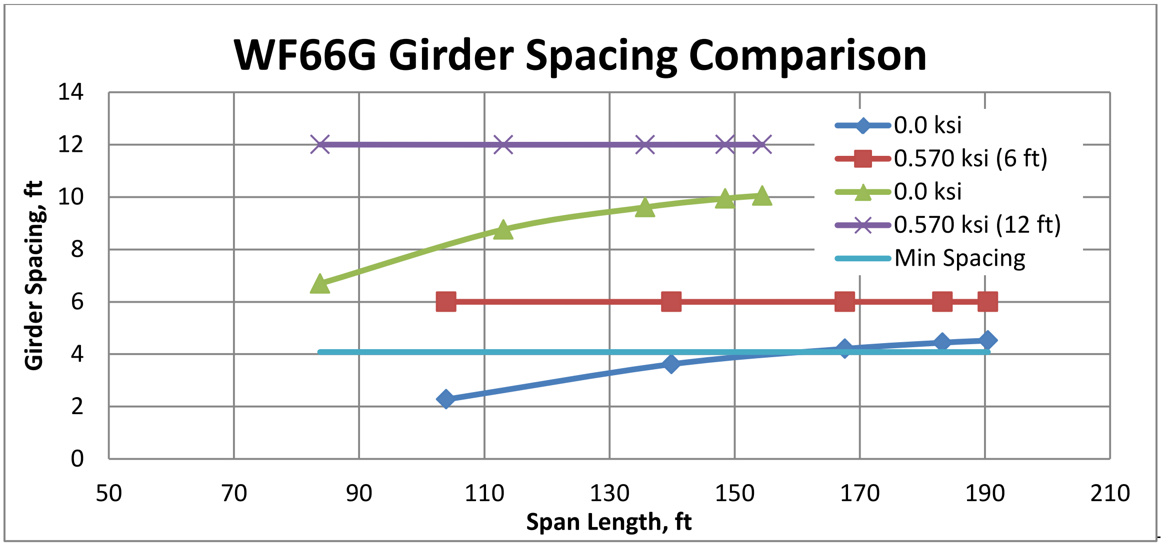

Figure 10 WF66G girder spacing comparison for allowable tension policy

CONTINUITY POLICY

AASHTO LRFD Section 5.14.1.4 address the requirements for bridges composed of simple span precast girders made continuous. These requirements include negative and positive moment connection requirements and the consideration of restraint moments due to time-dependent effects including creep and shrinkage of the girder and shrinkage of the deck slab. If the age of the girder when continuity is established is at least 90 days, the positive restraint moments caused by girder creep and shrinkage and deck slab shrinkage may be taken to be zero. Multi-span bridges composed of precast girders with continuity diaphragms at interior supports that are designed as a series of simple spans are not required to satisfy the requirements of Section 5.14.1.4.

It is important to note that in cases where the owner does not specify that the girder must be at least 90 days old at the time of slab casting, the continuity diaphragms are considered to be only partially effective. This is detailed in AASTHO LRFD Section 5.14.1.4.5. Washington State specifies a minimum age of 10 days for girder shipping and erection and at least 30 days must elapse prior to deck casting. In most scenarios that permit rapid construction, girders should be treated as simple spans for all loads in the service limit states.

42% of the survey respondents indicated the use of a simple span design policy. Washington, Michigan, Pennsylvania, and South Carolina design for the more critical of a fully effective continuity connection and the complete absence of continuity. Approximately 50% of the respondents to a survey by Hastak, et al[9], indicate they own or design bridges using a simple span design policy. The design positive moments for this policy are larger than when continuity is taken into account.

Figure 11 compares the baseline span capabilities of the WF66G girder to span capabilities computed using the simple span design policy. The span capability curves for the bridges designed with a simple span policy are to the left of the baseline curves. The reduction in span capability increases as the span length increases.

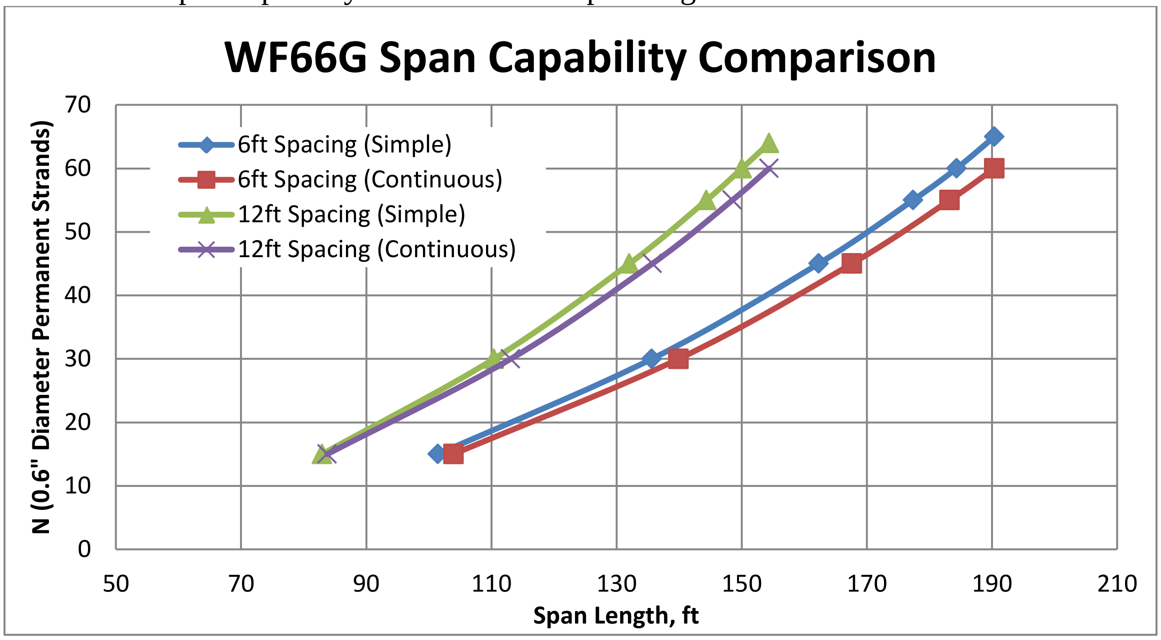

Figure 11 WF66G span capability comparison for continuity policy

Figure 12 compares the girder spacing for a WF66G girder. The prestressing levels and span lengths are chosen so that the required girder spacing for the continuous analysis is either 6 feet or 12 feet. Narrower girder spacing is needed when simple span analysis is used. The reduction in girder spacing reduces slightly as the span length increases.

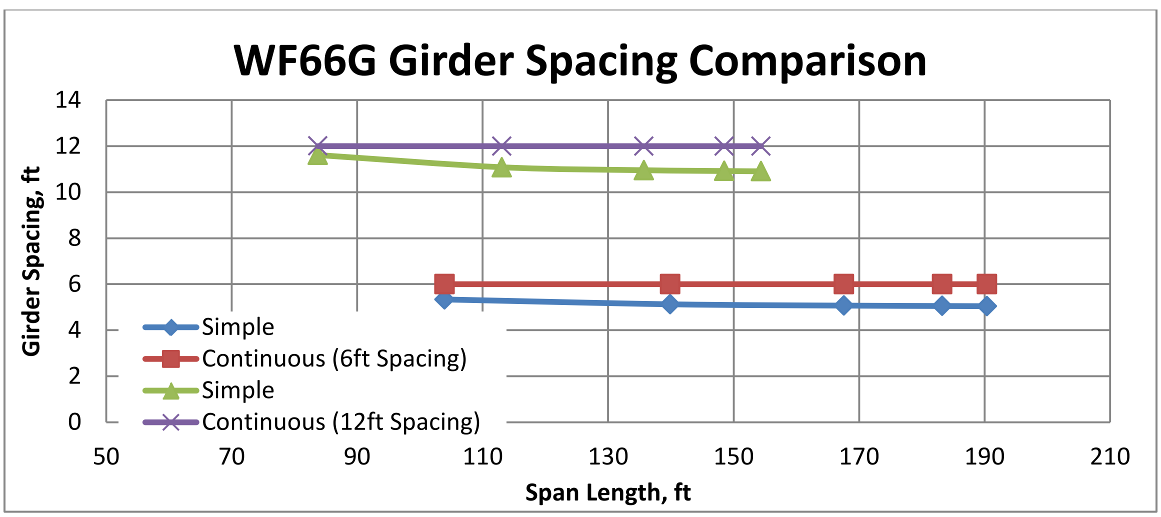

Figure 12 WF66G girder spacing comparison for continuity policy

COMBINDED DESIGN POLICIES

Washington and three other states use all three of the design policies considered in this study. To evaluate the effect of all the design policies together, simple span bridges are analyzed using gross section properties and an allowable Service III tension limit of 0.0 ksi. These results are compared to the baseline analysis for bridges that are made continuous for superimposed dead loads and live load and are analyzed using transformed section properties and an allowable Service III tension limit of 0.570 ksi. Figure 13 compares the span capabilities of the WF66G girder at 6 feet and 12 feet spacing for various levels of prestressing. The span capability curves for the bridges using the owner adopted policies are to the left of the baseline curves indicating a reduced span capability. The reduction in span capability increases as the span length increases.

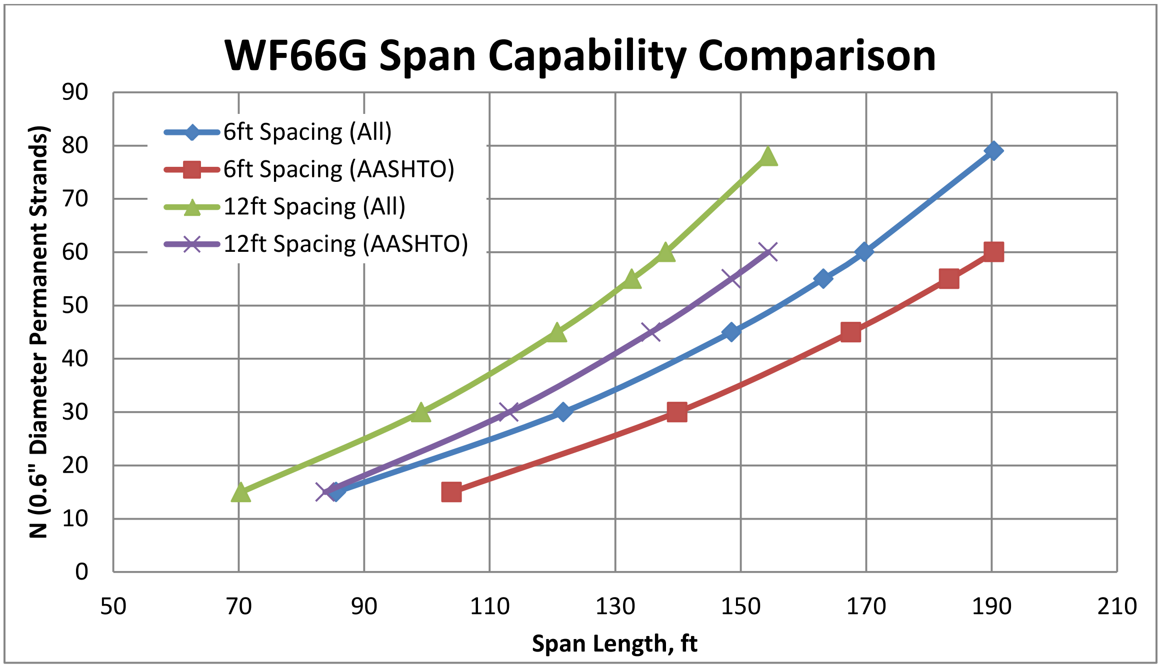

Figure 13 WF66G span capability comparison for all policies

Figure 14 compares the girder spacing for a WF66G girder. For the 6 ft spacing, the required reduction results in spacing that is less than the top flange width, this of course is not attainable.

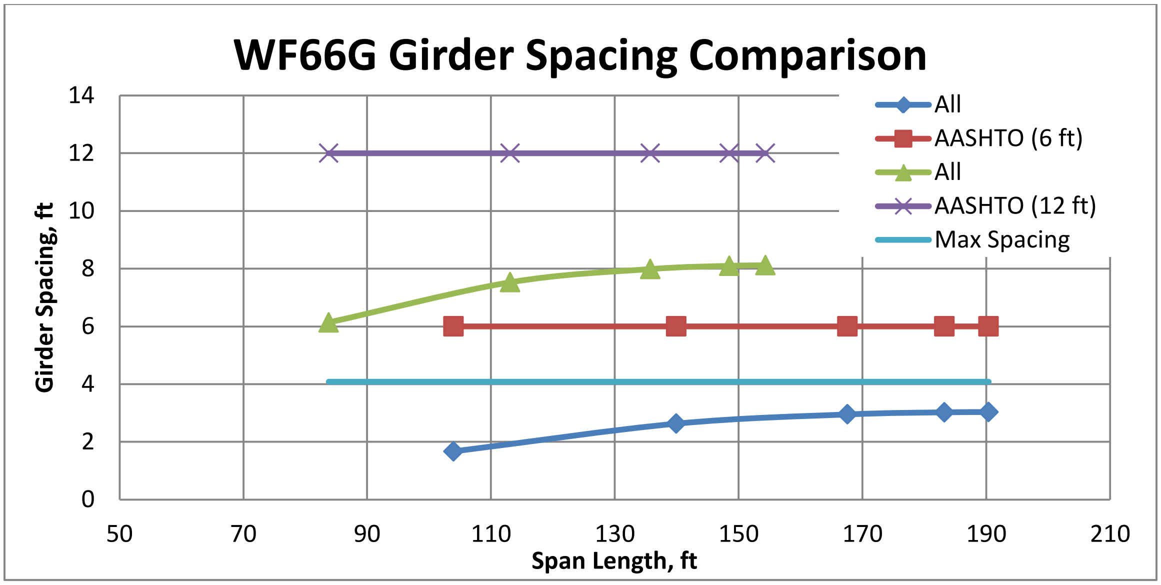

Figure 14 WF66G girder spacing comparison for all policies

DESIGN COMPARISON

Designs for all the WF-Series girders using the owner adopted policies are compared to the baseline designs. The tables in this section list the span capability, girder spacing, prestressing levels, and comparisons to the baseline bridge configurations given in Table 4.

Design results for the section properties policy are listed in Tables 5 and 6 and are compared to baseline bridge configurations that have a 6 ft and 12 ft girder spacing, respectively.

Table 5: Comparison of design variables based on gross section properties with baseline girder spacing of 6 ft

Girder	Span Capability (ft)	% Reduction	Girder Spacing (ft)	% Reduction	# Strands	% Increase
WF36G	121.74	3.0%	5.25	12.6%	54	8.0%
WF42G	134.10	2.7%	5.31	11.6%	54	8.0%
WF50G	154.53	2.6%	5.30	11.7%	59	7.3%
WF58G	167.74	2.3%	5.36	10.7%	59	7.3%
WF66G	186.03	2.3%	5.34	10.9%	64	6.7%
WF74G	196.97	2.0%	5.40	10.0%	64	6.7%
WF83G	214.58	2.0%	5.38	10.3%	69	6.2%
WF95G	235.18	1.9%	5.39	10.2%	74	5.7%
WF100G	241.10	1.8%	5.42	9.7%	74	5.7%

Table 6: Comparison of design variables based on gross section properties with baseline girder spacing of 12 ft

Girder	Span Capability (ft)	% Reduction	Girder Spacing (ft)	% Reduction	# Strands	% Increase
WF36G	97.11	3.0%	10.96	8.7%	54	8.0%
WF42G	107.19	2.7%	11.04	8.0%	54	8.0%
WF50G	124.02	2.7%	11.02	8.1%	59	7.3%
WF58G	135.18	2.4%	11.11	7.4%	59	7.3%
WF66G	150.66	2.4%	11.10	7.5%	64	6.7%
WF74G	160.24	2.2%	11.17	6.9%	64	6.7%
WF83G	175.47	2.1%	11.16	7.0%	69	6.2%
WF95G	193.61	2.0%	11.17	6.9%	74	5.7%
WF100G	199.04	1.9%	11.21	6.5%	74	5.7%

Design results for the allowable tension policy are listed in Tables 7 and 8. The design variables are computed based on an allowable tensile stress of 0.0 ksi and are compared with designs based on an allowable tensile stress of 0.570 ksi.

Table 7: Comparison of design variables based on an allowable tensile stress of 0.0 ksi with baseline girder spacing of 6 ft

Girder	Span Capability (ft)	% Reduction	Girder Spacing (ft)	% Reduction	# Strands	% Increase
WF36G	118.93	5.2%	4.65	22.5%	56	12.0%
WF42G	130.42	5.4%	4.59	23.5%	57	14.0%
WF50G	150.57	5.1%	4.60	23.4%	62	12.7%
WF58G	162.65	5.3%	4.51	24.8%	62	12.7%
WF66G	180.76	5.1%	4.52	24.6%	67	11.7%
WF74G	190.64	5.2%	4.44	26.0%	68	13.3%
WF83G	208.05	5.0%	4.44	26.0%	73	12.3%
WF95G	228.08	4.9%	4.41	26.5%	78	11.4%
WF100G	233.34	4.9%	4.36	27.3%	78	11.4%

Table 8: Comparison of design variables based on an allowable tensile stress of 0.0 ksi with baseline girder spacing of 12 ft

Girder	Span Capability (ft)	% Reduction	Girder Spacing (ft)	% Reduction	# Strands	% Increase
WF36G	94.88	5.2%	10.16	15.3%	56	12.0%
WF42G	104.27	5.4%	10.08	16.0%	57	14.0%
WF50G	120.89	5.2%	10.12	15.7%	62	12.7%
WF58G	131.16	5.3%	10.02	16.5%	62	12.7%
WF66G	146.48	5.1%	10.06	16.2%	67	11.7%
WF74G	155.19	5.2%	9.97	16.9%	68	13.3%
WF83G	170.25	5.1%	9.99	16.7%	73	12.3%
WF95G	187.89	4.9%	9.98	16.9%	78	11.4%
WF100G	192.77	5.0%	9.92	17.3%	78	11.4%

Design results for the continuity policy are listed in Tables 9 and 10. The design variables are computed based on simple span moments for all loads and are compared with designs based on full continuity for simple spans that are made continuous for superimposed dead and live loads.

Table 9: Comparison of design variables based on simple span analysis with baseline girder spacing of 6 ft

Girder	Span Capability (ft)	% Reduction	Girder Spacing (ft)	% Reduction	# Strands	% Increase
WF36G	121.82	2.9%	5.21	13.2%	54	8.0%
WF42G	133.59	3.1%	5.16	14.0%	54	8.0%
WF50G	153.69	3.2%	5.11	14.9%	60	9.1%
WF58G	166.25	3.2%	5.08	15.4%	60	9.1%
WF66G	184.33	3.2%	5.04	15.9%	65	8.3%
WF74G	194.70	3.2%	5.02	16.3%	65	8.3%
WF83G	212.12	3.1%	5.00	16.7%	70	7.7%
WF95G	232.33	3.1%	4.97	17.2%	76	8.6%
WF100G	237.92	3.1%	4.96	17.4%	76	8.6%

Table 10: Comparison of design variables based on simple span analysis with baseline girder spacing of 12 ft

Girder	Span Capability (ft)	% Reduction	Girder Spacing (ft)	% Reduction	# Strands	% Increase
WF36G	98.07	2.0%	11.24	6.3%	53	6.0%
WF42G	107.64	2.3%	11.13	7.2%	53	6.0%
WF50G	124.14	2.6%	11.02	8.2%	59	7.3%
WF58G	134.77	2.7%	10.96	8.7%	59	7.3%
WF66G	150.04	2.8%	10.91	9.1%	64	6.7%
WF74G	159.14	2.8%	10.88	9.4%	65	8.3%
WF83G	174.18	2.9%	10.84	9.7%	70	7.7%
WF95G	191.98	2.9%	10.81	9.9%	75	7.1%
WF100G	197.13	2.8%	10.80	10.0%	75	7.1%

Design results for the all of the owner adopted policies taken together are listed in Tables 11 and 12. The design variables are computed based on gross section properties, allowable tensile stress of 0.0 ksi, and simple span moments for all loads. The results are compared with designs based on transformed section properties, allowable tensile stress of 0.570 ksi, and full continuity for simple spans that are made continuous for superimposed dead and live loads.

The bridge configurations listed in Table 11 required a girder spacing that is less than the top flange width of a WF-Series girder and are thus unattainable. Furthermore, the number of prestressing strands satisfying Equation 1 for girder spacing of 6 ft and 12 ft result in high compression stress at release and require concrete strengths in excess of 7.0 ksi. The greatest release strength required is 7.8 ksi which may be attainable.

Table 11: Comparison of design variables based on all owner adopted design policies with baseline girder spacing of 6 ft. Shaded cells in the Girder Spacing column indicate unattainable bridge configurations because the required girder spacing is less than the top flange width of the girder. Shaded cells in the # Strands column indicate required concrete release strengths in excess of 7.0 ksi

Girder	Span Capability (ft)	% Reduction	Girder Spacing (ft)	% Reduction	# Strands	% Increase
WF36G	111.59	11.1%	3.23	46.2%	67	34.0%
WF42G	122.44	11.1%	3.18	46.9%	66	32.0%
WF50G	141.07	11.1%	3.12	47.9%	73	32.7%
WF58G	152.78	11.0%	3.08	48.7%	72	30.9%
WF66G	169.74	10.9%	3.03	49.5%	79	31.7%
WF74G	179.49	10.7%	2.99	50.2%	78	30.0%
WF83G	195.94	10.5%	2.94	50.9%	85	30.8%
WF95G	215.10	10.3%	2.88	51.9%	91	30.0%
WF100G	220.40	10.2%	2.85	52.5%	90	28.6%

Table 12: Comparison of design variables based on all owner adopted design policies with baseline girder spacing of 12 ft. Shaded cells in the # Strands column indicate required concrete release strengths in excess of 7.0 ksi

Girder	Span Capability (ft)	% Reduction	Girder Spacing (ft)	% Reduction	# Strands	% Increase
WF36G	90.07	10.0%	8.44	29.6%	65	30.0%
WF42G	98.79	10.4%	8.34	30.5%	65	30.0%
WF50G	113.98	10.6%	8.23	31.4%	72	30.9%
WF58G	123.79	10.6%	8.16	32.0%	72	30.9%
WF66G	138.03	10.6%	8.12	32.3%	78	30.0%
WF74G	146.52	10.5%	8.07	32.8%	78	30.0%
WF83G	160.67	10.4%	8.04	33.0%	84	29.2%
WF95G	177.46	10.2%	7.99	33.4%	90	28.6%
WF100G	182.31	10.2%	7.96	33.6%	90	28.6%

BENEFITS OF WSDOT DESIGN POLICIES

WSDOT has a long history of satisfactory performance of prestressed girder bridges. Bridges constructed in the 1950's are still in service with no sign of design deficiency or deterioration of girders. The satisfactory performance and longevity is due in part to conservative and sound design policies used since the early days of prestressed girder bridges in Washington State.

The current AASHTO LRFD recommends a minimum service life of 75 years for bridge structures. Conservative bridge design policies leave a margin of safety for prestressed girder bridges for unforeseen demands over the life of the structure. Supporting reasons for the conservative design policies for prestressed girder bridges include:

1. Historical increase in bridge live load: AASHTO design live loads have been increasing over the past few decades from HS-15 to HS-20 to HS-25, and to HL-93 in 1994.
2. Increasing use of overload trucks: The majority of bridges in Washington State are precast-prestressed girder structures. Virtually every permitted overload vehicle crosses a precast-prestressed girder bridge. Overloads often exceed the AASHTO specified design live loads. The reserve capacity due to conservative design practices allows prestressed girder bridges to withstand the overload trucks. Commerce would be adversely affected if these overloads could not be safely and conveniently moved. It should be noted that trucks carrying long-span prestressed girders are among the heaviest loadings ever permitted in Washington State.
3. Increase in number of traveling lanes: Due to increasing traffic volumes, lane widths on some routes have been often reduced from 12 feet to 10 feet to accommodate more traffic lanes. The reserve design capacity allows prestressed girder bridges to accommodate increased traffic demand and conform to the minimum requirements specified by AASHTO without strengthening or other modifications.
4. Periodic change in Bridge Design specifications: AASHTO design specifications have been changed from allowable stress design (ASD) to load factor design (LFD) and to load and resistance factor design (LRFD). More stringent design requirements have been observed with each change in design specifications.
5. Reserve capacity for girders damaged by over height collisions: The over height load collisions on prestressed girder bridges often results in broken strands that need to be repaired. Prior to repairs being made, the reserve capacity of the undamaged girders helps to keep the bridge in service. The current practice for splicing and re-tensioning broken strands limit the stress level to values lower than the original design. The reserve capacity due to conservative design practices allows repaired prestressed girders to satisfy design requirements.
6. Uncracked concrete under service conditions: The zero tension policy ensures that prestressed girders remain uncracked for flexure under service load conditions and overloads, resulting in longer service life.
7. Increased shear capacity: The conservative policies results in designs that require additional prestressing strands. This increase in prestressing results in higher shear capacity due to the vertical component of the prestress force in harped strands and reduced angle of the diagonal compression strut.
8. Reduced life cycle cost: The conservative design policies require more prestressing strands and possibly an additional line of girders, but results in longer service life and lesser life cycle cost.

The conservative design policies are an inexpensive insurance policy against future events including increasing legal loads, changing specifications, and unforeseen physical distress to the structure. The premium for this insurance policy is a one-time expense for as little as a half a dozen strands to one additional line of girders. This is typically a negligible percentage of overall project costs.

CONCLUSION

This study shows the sensitivity of span capability, girder spacing, and prestressing requirements of typical slab-on-beam wide flange I-girder bridge systems to three common owner adopted design policies. These owner adopted policies are more stringent than the minimum design requirements set forth in the AASHTO LRFD. As expected, the designs using the owner adopted policies result in a structure that is stouter than designs using the AASHTO minimum requirements.

Span capability is the least sensitive and girder spacing is the most sensitive to the owner adopted design policies. Designing based on gross section properties in lieu of transformed section properties has the least overall influence. Reducing the allowable tension stress at the Service III limit state has the greatest overall influence and has the greatest impact on girder spacing requirements.

NOTATION

f_b	=	Limit state stress due to externally applied loads
f_c'	=	28 day concrete strength
$K_t\sqrt{f_c'}$	=	Generalized form of the allowable tension limit from AASHTO LRFD Section 5.9.4.2.2
P_e	=	Effective prestress force
A	=	Area of girder
S_b	=	Bottom section modulus of the non-composite girder
e_{ps}	=	Eccentricity of prestressing strands
A_{ps}	=	Area of prestressing strand
f_{pj}	=	Jacking stress
Δf_{pLT}	=	Long term time dependent prestress losses
Δf_{pT}	=	Total prestress loss
Δf_{pES}	=	Prestress loss and gains due to elastic effects

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