CHALLENGES OF LONG-SPAN PRESTRESSED GIRDER BRIDGES IN WASHINGTON STATE

ABSTRACT

The rising need to upgrade the aging highway bridge system requires the development of innovative solutions that will lead to durable bridges with low life cycle costs. Long-span bridges made with higher strength concretes are often used in environmentally sensitive terrains, water crossings, and in locations with traffic and geometrical restrictions. Precast pretensioned girders are beneficial for long span bridges due to their ease of construction, lower life-cycle cost and durability.

The applicability of the AASHTO LRFD Bridge Design and Construction Specifications to long span prestressed girders is unclear and needs to be studied. The suitability and benefits of larger diameter prestressing strands, high strength concretes, ultra-high performance concretes, and light weight aggregate concrete for long span pretensioned girders should be investigated.

This paper discusses the design, fabrication, shipping and handling associated with long-span precast pretensioned girders. The current WSDOT practice for design and construction of long span precast pretensioned girder superstructures is presented in this paper.

Keywords: Bridge, Girder, HSC, Long-Span, Design, Fabrication, Handling

INTRODUCTION

The recent development of long span prestressed girders has allowed the Washington State Department of Transportation (WSDOT) and other bridge owners to solve the problem of lengthening spans using construction materials they prefer. Long span prestressed girders made with high performance concrete (HPC) eliminate the need for falsework, reduce on-site construction activities, reduce environmental impacts at water crossings, and minimize hazards, delays, and inconvenience to the traveling public.

The use of high strength concrete (HSC) and 0.6-in. diameter strand in the fabrication of precast, pretensioned concrete bridge girders has resulted in an improved economy through the use of longer spans, increased girder spacing or fewer girder lines, and shallower superstructures. The design, however, presents the engineer with several challenges regarding fabrication, handling, shipping, and erection of long, slender girders. Seguirant[6] presents techniques to overcome many of these challenges, such as the use of temporary top strands to improve lateral stability.

Ultra high performance concrete (UHPC) is a newly developed concrete material that provides very high strength and low permeability, which could provide major improvements over conventional high HPC bridges in terms of structural efficiency, durability and life cycle cost-effectiveness. UHPC yields a considerable reduction in the number of girders, girder size and span lengths in comparison to HPC bridges.

PRECAST PRETENSIONED GIRDERS

The majority of bridges in Washington are prestressed girder bridges. The current WSDOT standard precast pretensioned wide flange girders span up to 240 ft, and standard precast pretensioned trapezoidal tub girders span up to 180 ft. The span capabilities take into account the girder cross section dimensions, shipping stability, and hauling weight limitations. Fig. 1 shows a typical precast prestressed girder bridge project in Washington.

HSC is used as a standard material for the fabrication and construction of long span prestressed concrete girder bridges in Washington. Concrete strengths of 7.5 ksi at prestress transfer and 9.0 ksi at final are the current upper limits. Higher concrete release strengths up to 8.5 ksi are possible if curing is extended to an every-other-day cycle.

Fig. 1: A Long Span Precast Girder Bridge Project in Washington State

The cross sections of some of WSDOT standard wide flange girders and trapezoidal tub girders used in composite superstructures are shown in Figs. 2 and 3. The complete description of WSDOT standard pretensioned girders and post-tensioned spliced girders are presented in Reference 3. The time-dependent prestress losses in prestressed girders built with high performance concrete are discussed in Reference 4. The proposed simplified method for shear design of prestressed bridge girders using LRFD specifications is discussed in Reference 5. Reference 6 introduces the newly developed WSDOT wide flange deep precast prestressed concrete girders. The benefits of high performance concrete for precast prestressed girders in Washington State are discussed in Reference 7. Reference 8 discusses the flexural strength of reinforced and prestressed concrete T- Beams, and Reference 9 discusses the prestressed girder design optimization.

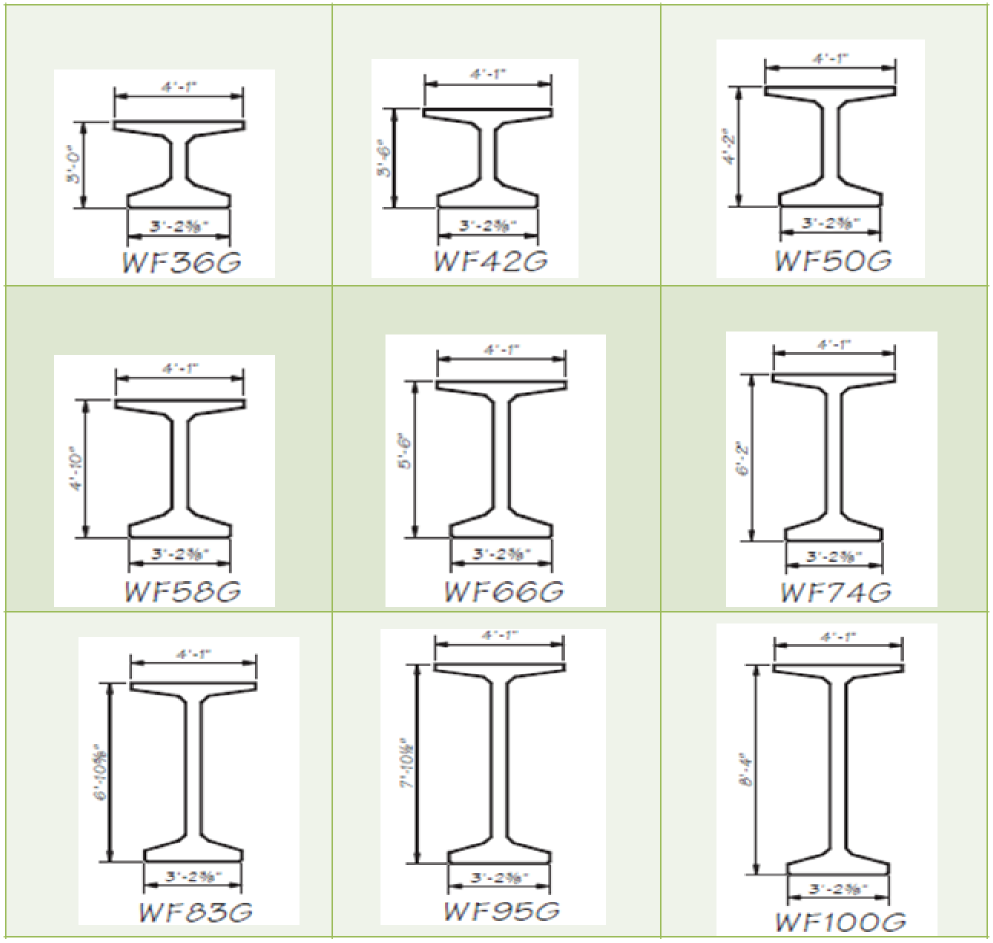


Fig 2. WSDOT Wide Flange Precast Pretensioned Girder types

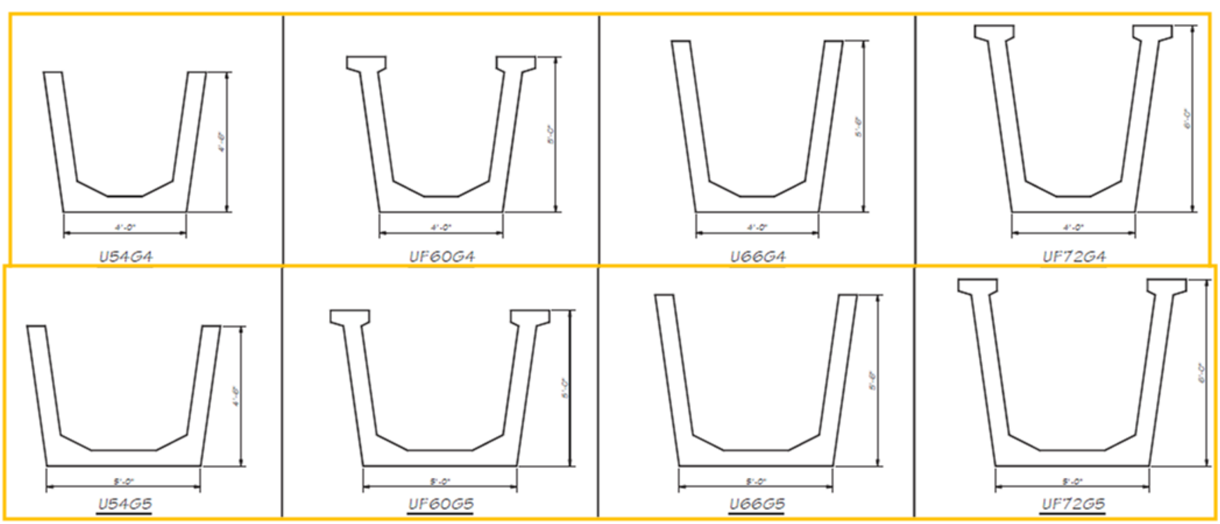


Fig. 3: WSDOT Precast Pretensioned Trapezoidal Tub Girder Types

Span Capability of WSDOT wide flange precast pretensioned girders for various girder spacings is shown in Table 1. The design criteria used for developing Table 1 are:

- Design for straight interior girders without any vertical or horizontal curves
- Simple span girder design with span length of centerline to centerline bearings
- Relative humidity of 75% under normal exposure
- Strength of concrete for deck slab 4.0 ksi
- Strength of concrete for girders 7.5 ksi at transfer and 9.0 ksi at final
- 0.6" diameter prestressing strand grade 270 for pretensioning strands.
- Includes 2" future HMA overlay with density of 140 pcf

Table 1: Span Capability of Wide Flange Girders for Different Girder Spacing

Girder Type	Girder Spacing (ft)	Span CL-to-CL Bearing (ft)	Deck Thickness (in)	Shipping Weight (kips)
WF36G	6	105	7.5	79
	8	95	7.5	76
	10	85	7.5	68
	12	80	7.5	64
WF42G	6	120	7.5	96
	8	105	7.5	88
	10	95	7.5	80
	12	90	8.0	75
WF50G	6	135	7.5	119
	8	120	7.5	107
	10	110	7.5	98
	12	105	8.0	94
WF58G	6	150	7.5	141
	8	140	7.5	132
	10	130	7.5	122
	12	120	8.0	113
WF66G	6	165	7.5	159
	8	150	7.5	149
	10	140	7.5	139
	12	130	8.0	130
WF74G	6	175	7.5	178
	8	160	7.5	168
	10	150	7.5	157
	12	140	8.0	147
WF83G	6	190	7.5	204
	8	175	7.5	188

	10	160	7.5	177
	12	150	8.0	166
WF95G	6	215	7.5	220
	8	195	7.5	208
	10	190	7.5	196
	12	165	8.0	185
WF100G	6	240	7.5	240
	8	225	7.5	227
	10	215	7.5	215
	12	195	8.0	203

STRUCTURAL EFFICIENCY OF WIDE FLANGE PRECAST GIRDER

The WSDOT Wide Flange Precast Pretensioned Girders are among the most efficient sections used in the industry. The structural efficiency of WSDOT wide flange girders using Guyon's equation is shown in Table 2. The Guyon equation is based on the cross sectional properties of the girder and is expressed as:

$$\rho = \frac{r^2}{y_b y_t} \qquad (1)$$

Where ρ is the efficiency factor, y_t and y_b are the centroid location measured from the top and bottom of the girder respectively, and r is the radius of gyration of the cross section, expressed as: $r = \sqrt{\frac{I}{A}}$. An increase in the $\frac{I}{A}$ ratio will result in greater girder efficiency that can be achieved by reducing the cross sectional area while increasing the moment of inertia.

The structural efficiency factor can be adjusted with the ratio of span length, *L* over the unit weight of girders, *w*, as shown in Equation 2:

$$\rho = \frac{r^2}{y_b y_t}\frac{L}{w} \qquad (2)$$

The second term in Equation 2 allows for further improvement of structural efficiency by either increasing span length or reducing unit weight. Increasing span length could be achieved by using more efficient materials such as UHPC. The use of UHPC, because of its superior structural properties, allows further optimization of the girder cross section using thinner web, top and bottom flange, resulting in an increase of 50% in structural efficiency. Girders made with structural light weight concrete (LWC) can reduce the unit weight of precast girders by 20% as compared to normal weight concretes (NWC). The modified structural efficiency of wide flange precast pretensioned girders are shown in Table 2.

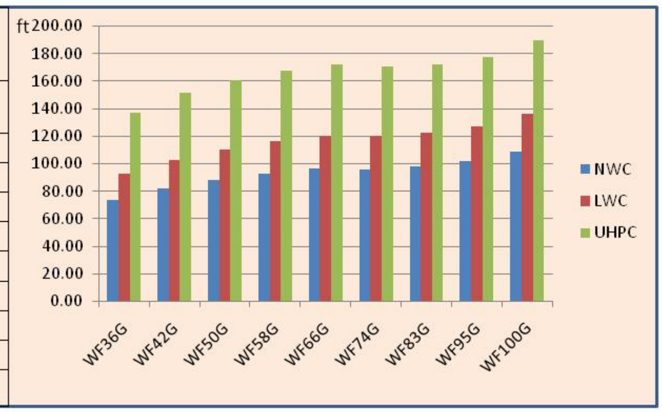
Table 2: Structural Efficiency of WSDOT Wide Flange Precast Pretensioned Girders

Type	Depth (in)	Area (in2)	I_z (in4)	Y_b (in)	W_t (k/ft)	Span Capability (ft)	$\rho = \frac{r^2}{y_b y_t}$	$\rho = \frac{r^2}{y_b y_t}\frac{L}{w}$ (ft^2/kips)
WF36G	36	690.8	124772	17.54	0.792	105	0.56	73.96
WF42G	42	727.5	183642	20.36	0.834	120	0.57	82.44
WF50G	50	776.5	282559	24.15	0.89	135	0.58	88.42
WF58G	58	825.5	406266	27.97	0.946	150	0.59	92.91
WF66G	66	874.5	556339	31.8	1.002	165	0.58	96.33
WF74G	74	923.5	734356	35.66	1.058	175	0.58	96.20
WF83G	82.63	976.4	959393	39.83	1.119	190	0.58	97.87
WF95G	94.5	1049.1	1328995	45.6	1.202	215	0.57	101.62
WF100G	100	1082.8	1524912	48.27	1.241	240	0.56	109.07

As shown in Table 2, the structural efficiency of WSDOT wide flange girders made with conventional HSC is slightly below the target value of 100 ft^2/kip. This could be increased by introducing more efficient materials such as UHPC or LWC, as shown in Table 3.

Table 3: Comparison of Structural Efficiency between NWC, LWC and UHPC

Girder Type	$\rho = \frac{r^2}{y_b y_t}\frac{L}{w}$		
	NWC	LWC	UHPC
WF36G	73.96	92.44	137.35
WF42G	82.44	103.05	151.76
WF50G	88.42	110.52	161.04
WF58G	92.91	116.13	167.61
WF66G	96.33	120.41	172.28
WF74G	96.20	120.25	170.70
WF83G	97.87	122.33	172.31
WF95G	101.62	127.02	177.21
WF100G	109.07	136.34	189.44



WSDOT DESIGN CRITERIA FOR PRESTRESSED GIRDER

Precast prestressed girder bridges are designed as simple spans for all transient and permanent loads for both simple and continuous spans. Continuity reinforcement shall be provided at intermediate piers for transient and superimposed dead loads applied after completion of the bridge deck.

AASHTO LRFD 5.14.1.4 “Bridges Composed of Simple Span Precast Girders Made Continuous” allows for some degree of continuity for loads applied on the bridge after the continuity diaphragms have been cast and cured. This assumption is based on the age of the girder when continuity is established, and the degree of continuity at various limit states. Both degree of continuity and time of continuity diaphragm casting may result in contractual and design issues. Designing these types of bridges for the envelope of simple span and continuous spans for applicable permanent and transient loads is a conservative approach to addressing these issues.

Girder types and spacing shall be identical in adjacent spans. Girder types and spacing may be changed at expansion joints at intermediate piers. For continuous structures, the same type and number of prestressed girders shall be used.

The stress limits for the design of precast prestressed girders are shown in Table 3. One or more of the conditions may govern design. These stress limits supersede the AASHTO LRFD Specifications and are more conservative in some cases. Allowable tensile stresses at service limit states in the precompressed tensile zone shall be limited to zero. This prevents cracking of the concrete during the service life of the structure and provides additional stress and strength capacity for overloads.

Table 3: Allowable concrete stresses for the service and fatigue limit states

Condition	Stress	Location	Allowable Stress
Temporary Stress at Transfer and at Lifting from Casting Bed	Tensile	In areas other than the precompressed tensile zone and without bonded reinforcement	$0.0948\sqrt{f'_{ci}} \leq 0.2\ (ksi)$
		In areas with bonded reinforcement sufficient to resist tensile force in the concrete	$.19\sqrt{f'_{ci}}\ (ksi)$
	Compressive	All Locations	$0.65\ f'_{ci}$
Temporary Stress at Shipping and Erection	Tensile	In areas other than the precompressed tensile zone and without bonded reinforcement	$0.0948\ \sqrt{f'_c}\ (ksi)$
		In areas other than the precompressed tensile zone and with bonded reinforcement, plumb girder with impact	$0.19\sqrt{f'_c}\ (ksi)$
		In areas other than the precompressed tensile zone and with bonded reinforcement, inclined girder without impact	$0.24\sqrt{f'_c}\ (ksi)$
		In areas other than the precompressed tensile zone and with bonded reinforcement, after temporary top strand detensioning	$0.19\sqrt{f'_c}\ (ksi)$
	Compressive	All locations	$0.65\ f'_c$
Final Stresses at Service Load	Tensile	Precompressed tensile zone	0.0
	Compressive	Effective prestress and permanent loads	$0.45\ f'_c$
		Effective prestress, permanent loads and transient loads	$0.60\ f'_c$
Final Stresses at Fatigue Load	Compressive	Fatigue I Load Combination plus one-half effective prestress and permanent loads per AASHTO LRFD 5.5.3.1	$0.40\ f'_c$

For long span precast pretensioned concrete girders, the total number of permanent prestressing strands (straight and harped) is limited to 100 - 0.6″φ strands. Harping points are set at 0.4 times the girder length from either end. The forces on the hold-down units are developed as the harped strands are raised. The hold-down device provided by the fabricator must be able to hold the vertical component of the harping forces. Normally, two or more hold-down units are required. Standard commercial hold-down units have been preapproved for use with particular strand groups. Strand location at the end and midspan of a typical prestressed girder is shown in Fig. 4. Strand location and layout are identical for all long span wide flange prestressed girders.

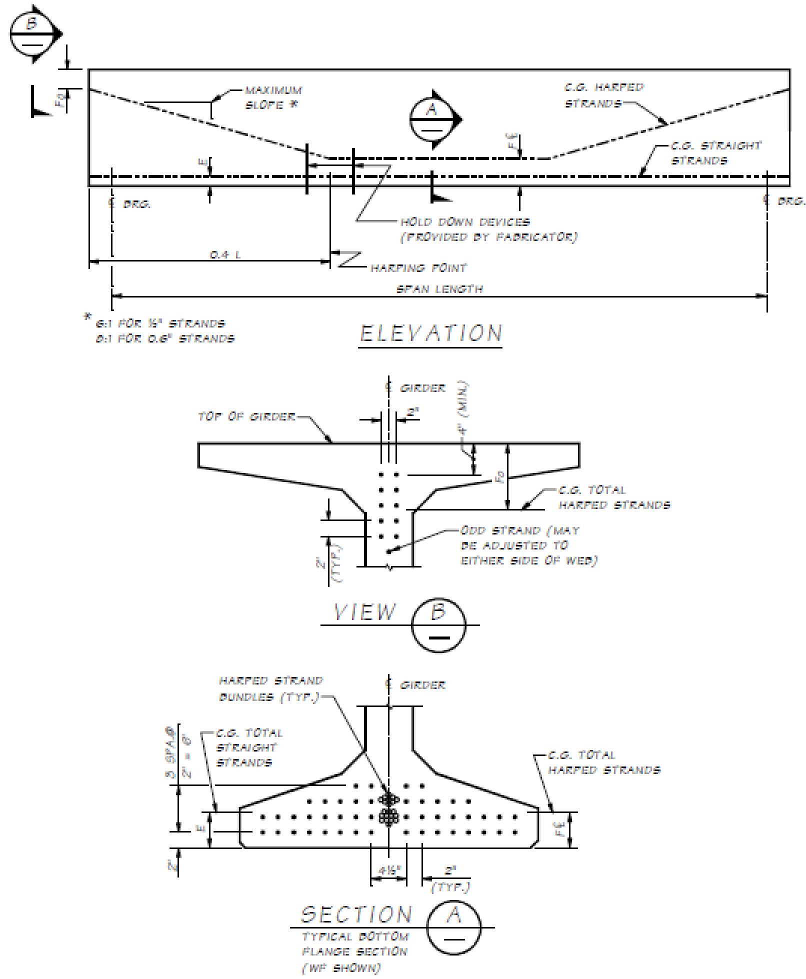


Fig. 4: Typical Prestressed Girder Configuration

INTERMEDIATE DIAPHRAGMS FOR PRECAST GIRDER BRIDGES

WSDOT requires intermediate diaphragms for all prestressed girder bridges. The location and number of diaphragms are shown below:

- 1/5 points of span for span lengths over 160′-0″
- ¼ points of span for span lengths 120′-0″ to 160′-0″

- ⅓ points of span for span lengths 80′-0″ to 120′-0″
- Midpoint of span for span lengths 40′-0″ to 80′-0″
- No diaphragm requirement for span lengths less than 40′-0″

Prestressed concrete girder bridges are often damaged by over-height loads. The damage may range from spalling and minor cracking of the bottom flange or web of the prestressed concrete girder to a loss of a major portion of a girder section.

Based on research done by WSU[10] the use of intermediate diaphragms for wide flange I-shape and deck bulb tee prestressed concrete girder bridges shall be as follows:

a. Full depth intermediate diaphragms shall be used for Interstate bridges, and other bridges crossing over roads of ADT > 50000
b. Either full depth or partial depth intermediate diaphragms may be used for all other precast girder bridges

The use of full or partial depth intermediate diaphragms in bridge widenings shall be considered on a case-by-case basis depending on the width of the widening and number of added girders.

WELDED WIRE REINFORCEMENT IN PRECAST PRESTRESSED GIRDERS

Welded wire reinforcement can be used to replace mild steel reinforcement in precast prestressed girders. The yield strength shall be greater than or equal to 60 ksi. The design yield strength shall be 60 ksi. Welded wire reinforcement shall be deformed and shall have the same area and spacing as the mild steel reinforcement that it replaces. Shear stirrup longitudinal wires (tack welds) shall be excluded from the web of the girder and are limited to the flange areas as described in AASHTO LRFD 5.8.2.8. Longitudinal wires for anchorage of welded wire reinforcement shall have an area of 40% or more of the area of the wire being anchored.

DEFLECTION CALCULATION FOR PRESTRESSED GIRDERS

Flexural members are designed to have adequate stiffness to limit deflections or any deformations which may adversely affect the strength or serviceability of the structure at service load plus impact. The minimum superstructure depths are specified in AASHTO LRFD Table 2.5.2.6.3-1 and deflections shall be computed in accordance with AASHTO LRFD 5.7.3.6.2.

WSDOT requires two levels of girder camber at the time the deck concrete is placed, denoted as D_{40} and D_{120}. The "D" dimension is the computed girder deflection at midspan (positive upward) immediately prior to deck slab placement. The concept is to provide the contractor with lower and upper bounds of camber that can be anticipated in the field. They shall be shown in the Contract Plans to accommodate early girder placement and slab casting for ABC projects.

The upper bound of camber D_{120} is estimated as the upper bound of expected camber range at a girder age of 120 days after the release of prestress, and is primarily intended to mitigate interference between the top of the cambered girder and the placement of concrete deck reinforcement at midspan.

The lower bound of camber D_{40} is estimated as the lower bound of expected camber range at a girder age of 40 days (30 days after the earliest allowable girder shipping age of 10 days). To match the profile grade, girders with too little camber require an increased volume of haunch concrete along the girder length. For girders with large flange widths, such as the wide flange series, this can add up to significant quantities of additional concrete for a large deck placement. Thus, the lower bound of camber allows the contractor to assess the risk of increased concrete quantities and mitigates claims for additional material.

Fig. 5 shows a typical pattern of girder deflection with time at centerline span. Portions of this characteristic curve are described below. The paragraph numbers below correspond to the circles on Fig. 5.

1. Elastic Deflection Due to Release of Prestress: The prestress force produces moments in the girder which tends to bow the girder upward. Resisting these moments are girder section dead load moments. The result is a net upward deflection.
2. Creep Deflection Before Cutting Temporary Strands: The girder continues to deflect upward due to the effect of creep.
3. Deflection Due to Cutting of Temporary Strands: Cutting of temporary strands results in an elastic upward deflection. The default time interval for creep calculations for release of top temporary strands is 90 days after the release of prestress during girder fabrication for D_{120} (10 days for D_{40}).
4. Diaphragm Load Deflection: The load of diaphragm is applied to the girder section resulting in an elastic downward deflection. The default time interval for creep calculations for placing diaphragms is 90 days after the release of prestress during girder fabrication for D_{120} (10 days for D_{40}).
5. Creep Deflection After Casting Diaphragms: The girder continues to deflect upward for any time delay between diaphragms and deck slab casting.
6. Deck Slab Load Deflection: The load of the deck slab is applied to the girder section resulting in an elastic downward deflection. The default time interval for creep calculations for placing the deck slab is 120 days after the release of prestress during girder fabrication for D_{120} (40 days for D_{40}).
7. Superimposed Dead Load Deflection: The load of the traffic barriers, sidewalk, overlay, etc. is applied to the composite girder section resulting in an elastic downward deflection.
8. Final Camber.

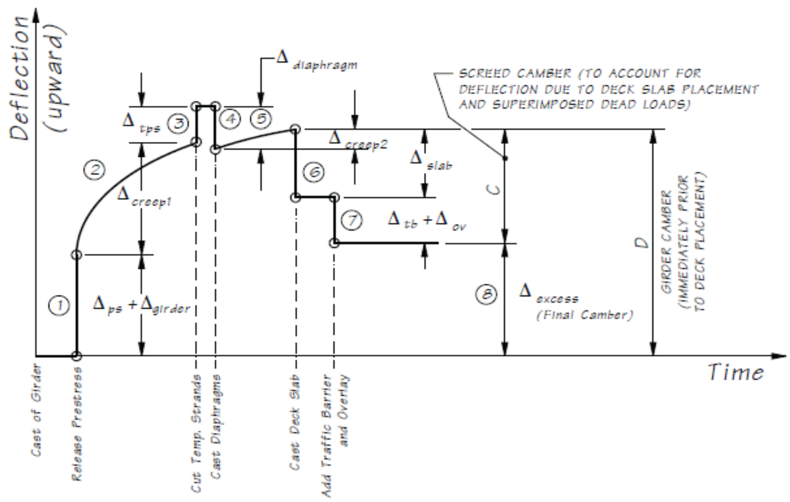


Fig. 5: Prestressed Girder Camber

It may be expected that the above deck slab dead load deflection would be accompanied by a continuing downward deflection due to creep. However, many measurements of actual structure deflections have shown that once the deck slab is poured, the girder tends to act as though it is locked in position. To obtain a smooth riding surface on the deck, the deflection indicated on Fig. 5 as "Screed Camber" (known as "C") is added to the profile grade elevation of the deck screeds. The "C" dimension and the "Screed Setting Dimensions" detail shall be given in the plans. The WSDOT research project entitled "Improving Predictions for Camber in Precast, Prestressed Concrete Bridge Girders" is discussed in Reference 13.

Precast prestressed girders may be pre-cambered to compensate for the natural camber and for the effect of the roadway geometry. Pre-cambering may be beneficial for long span prestressed girders to minimize the additional weight due to concrete pad on top of girder.

DEFLECTION AND CAMBER MULTIPLIERS

Deflections of prestressed concrete beams can be predicted with greater accuracy than those for reinforced concrete beams. Since prestressed concrete is more or less homogeneous and obeys ordinary laws of flexure and shear, the deflection can be computed using elementary methods. However, accurate predictions of the deflections are difficult to determine, since modulus of elasticity of concrete, E_c, varies with stress and age of concrete. Additionally, the effects of creep on deflections are difficult to estimate. For practical purposes, an accuracy of 10 to 20% is often sufficient. Although prestressing can be used advantageously to control deflections, there are cases where excessive camber due to prestress has caused problems. For normal design, in lieu of more accurate methods, the deflection and camber of prestressed members may be estimated by the multipliers shown in Table 4.

Table 4: Multipliers for Estimating Long-term Deflection of Prestressed Concrete Girders

	40 Days		120 Days	
	Non-Composite	Composite	Non-Composite	Composite
Deflection at Erection				
Apply to the elastic deflection due to the member weight at release of prestress	1.85	1.85	1.75	1.75
Apply to the elastic deflection due to prestressing at release of prestress	1.80	1.80	1.70	1.70
Deflection at Final				
Apply to the elastic deflection due to the member weight at release of prestress	2.70	2.40	2.50	2.20
Apply to the elastic deflection due to prestressing at release of prestress	2.45	2.20	2.25	2.10
Apply to the elastic deflection due to the Super Imposed Dead Loads	3.00	3.00	2.75	2.75
Apply to the elastic deflection due to weight of deck slab	----	2.30	----	2.15

SPLITTING RESISTANCE IN END REGIONS OF PRESTRESSED GIRDERS

The splitting resistance of pretensioned anchorage zones shall be as described in AASHTO LRFD 5.10.10.1. For long span pretensioned wide flange girders, the end vertical reinforcement shall not be larger than #5 bars and spacing shall not be less than 2¼″. The remaining splitting reinforcement not fitting within the h/4 zone may be placed beyond the h/4 zone at a spacing of 2¼″. Fig. 6 shows splitting reinforcement detail, girder end zone reinforcement and potential hairline crack due to splitting force.

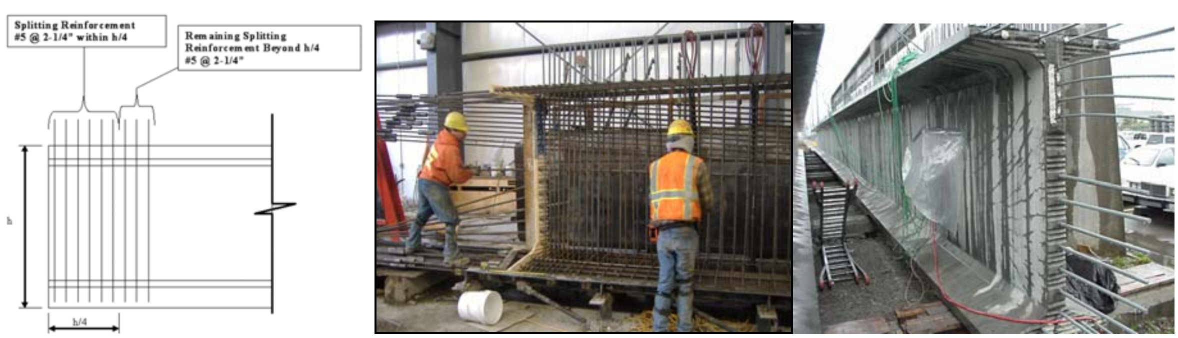


Fig. 6: Splitting Reinforcement at Girder End Zone

FABRICATION AND HANDLING OF LONG SPAN PRECAST GIRDERS

The method selected for strand tensioning may affect the design of the girders. The strand arrangements shown in the plans are satisfactory for tensioning methods used by the fabricators. Harped strands are normally tensioned by pulling them as straight strands to a partial tension. The strands are then deflected vertically as necessary to give the required harping angle and strand stress. In order to avoid over-tensioning of the harped strands by this procedure, the slope of the strands is limited to a maximum of 8:1 for 0.6″ φ strands for long span prestressed girders. The straight strands are tensioned by straight jacking.

Forces on the hold-down units are developed as the harped strands are raised. The hold-down device provided by the fabricator must be able to hold the vertical component of the harping forces. WSDOT does not allow debonding for wide flange pretensioned girders. Fig. 7 shows long span prestressed girders at the fabrication plant and storage yard.

Fig. 7: Long Span Prestressed Girders at the Fabrication Plant and Storage Yard.

GIRDER LATERAL BENDING

Long prestressed girders are very flexible and highly susceptible to lateral bending. Lateral bending failures are sudden, catastrophic, costly, and pose a serious threat to workers and surroundings, and therefore must be guarded against. The girder standard plans state that girders over certain lengths must be laterally braced and that all girders must be handled carefully.

STABILITY OF LONG SPAN PRECAST GIRDERS

Long prestressed girders can become laterally unstable when handled and shipped. Lift points never perfectly coincide with the center of gravity of the girder. As such, the girders tend to roll about the lift axis and deflect laterally until equilibrium is achieved. Camber amplifies this condition. During shipping, the girders are subjected to lateral deflections due to super-elevation along the route. Girders with large top flanges have a tendency to adversely affect the stability of the truck. Fig. 8 shows the shipping of long span prestressed girders by truck and by barge.

Fig. 8: Shipping of Precast Girders

Precast girders shall also be checked during lifting, transportation, and erection stages by the designer to assure that girder delivery is feasible. Impact during the lifting stage shall be 0% and during transportation shall be 20% of the dead load of the girder. Impact shall be applied either upward or downward to produce maximum stresses.

Temporary top strands are used to improve the girder stability during handling and shipping. Temporary top strands are pre-tensioned or post-tensioned shortly after the forms are stripped from the girder. Pretensioned temporary strands are bonded along the end 10 ft (3.05 m) of the girder and unbonded elsewhere. Blockouts are provided at the middle of the girder to allow access to the strands. Temporary strands are released after final placement just prior to placing the diaphragm concrete by cutting or burning the strands. Failure to release the prestress force may have an adverse effect on the structural behavior of the girder. WSDOT requires that all temporary strands be flagged when girders are shipped to the job site, and the bridge plans provide instructions for releasing the temporary strands.

The introduction of temporary strands in the top flange of the prestressed girders also has beneficial effects on the girder design. The temporary top strands reduce the instantaneous deflection and long-term camber, which results in a reduction of the volume of concrete required for the cast-in-place deck haunches. This translates into less material, lower dead load moments, and lower required compressive concrete strength at transfer of prestress.

Lateral stability can be a concern when handling long, slender girders. When the girder forms are stripped from the girder, the prestressing level is higher and the concrete strength is lower than at any other point in the life of the member.

The WSDOT prestressed girder sections are relatively wide and stiff about their weak axes and, as a result, exhibit good stability even at their longer pretensioned lengths. The simplest method of improving stability is to move the lifting devices away from the ends. This invariably increases the required concrete release strength, because decreasing the distance between lifting devices increases the concrete stresses at the harp point. Stresses at the support may also govern, depending on the exit location of the harped strands. Temporary prestressing in the top flange can also be used to provide a larger factor of safety against cracking.

The ability to ship deep girder sections can be influenced by a large number of variables, including mode of transportation, weight, length, height, and lateral stability. Some variables have more influence than others. As such, the feasibility of shipping deep girders is strongly site dependent. It is recommended that routes to the site be investigated during the preliminary design phase.

ALTERNATIVE DESIGN FOR LONG SPAN PRECAST GIRDERS

Long span prestressed concrete girder bridges may bear increased costs due to difficulties encountered during the fabrication, shipping, and erection of such one-piece girders. Providing an alternate spliced-girder design to long span one-piece pretensioned girders may eliminate the excessive cost through competitive bidding. The following procedure for alternative design of prestressed concrete girders in the Plans shall be followed:

- All prestressed concrete girders with shipping weight less than 190 kips shall be shown in the Contract Plans as pretensioned only.
- All prestressed concrete girders with shipping weight between 190 and 240 kips shall include both pre-tensioned and post-tensioned spliced prestressed concrete girder.
- All prestressed concrete girders with shipping weight exceeding 240 kips shall be spliced prestressed concrete girders, with post-tensioning applied after the casting of the girder closures and deck slab.

The designer shall provide shipping support locations in the plans to ensure adequate girder stability. For normal designs, shipping support locations should not be closer than the girder depth to the ends of the girder. The overhangs at the leading and trailing ends of the girders should be minimized and equal, if possible. However, the leading end overhang should not exceed 15′ to avoid interference with trucking equipment. Shipping support locations shall maintain the concrete stresses within allowable limits.

Length between shipping support locations may be governed by turning radii on the route to the jobsite. Potential problems can be circumvented by moving the support points closer together (away from the ends of the girder), or by selecting alternate routes. A distance of up to 130′ between supports is typically acceptable for most projects.

The height of a deep girder section sitting on a jeep and steerable trailer is of concern when considering overhead obstructions on the route to the jobsite. The height of the support is approximately 6′ above the roadway surface. When adding the depth of the girder, including camber, the overall height from the roadway surface to the top of concrete can rapidly approach 14′. Overhead obstructions along the route should be investigated for adequate clearance in the preliminary design phase. Obstructions without adequate clearance must be bypassed by selecting alternate routes.

Expectations are that, in some cases, overhead clearance will not accommodate the vertical stirrup projection on deeper WSDOT standard girder sections. Alternate stirrup configurations can be used to attain adequate clearance, depending on the route from the plant to the jobsite.

ERECTION

A variety of methods are used to erect precast concrete girders, depending on the weight, length, available crane capacity, and site access. Lifting long girders during erection is not as critical as when they are stripped from the forms, particularly when the same lifting devices are used for both. If a separate set of erection devices are used, however, the girder shall be checked for stresses and lateral stability. In addition, once the girder is set in place, the free span between supports is usually increased. Wind can also pose a problem. Consequently, when long girders are erected, they shall immediately be braced at the ends. Generally, the temporary support of the girders is the contractor’s responsibility. Fig. 9 shows the erection of long span prestressed girders for WSDOT bridge projects.

Fig. 9: Erection of WF100G and WF83G Precast Girders

CONSTRUCTION SEQUENCE FOR MULTI-SPAN GIRDER BRIDGES

For multi-span prestressed girder bridges, the sequence and timing of the superstructure construction has a significant impact on the performance and durability of the bridge. Particular attention should be paid to the timing of casting of the lower portion of the pier diaphragms/crossbeams (30 days minimum after girder fabrication) and the upper portion of the diaphragms/crossbeams (10 days minimum after placement of the deck slab). The requirements apply to multi-span prestressed girder bridges with monolithic and hinge diaphragms/crossbeams. The construction sequences for simple spans and for other girder types are similar to the sequences shown in Fig. 10.

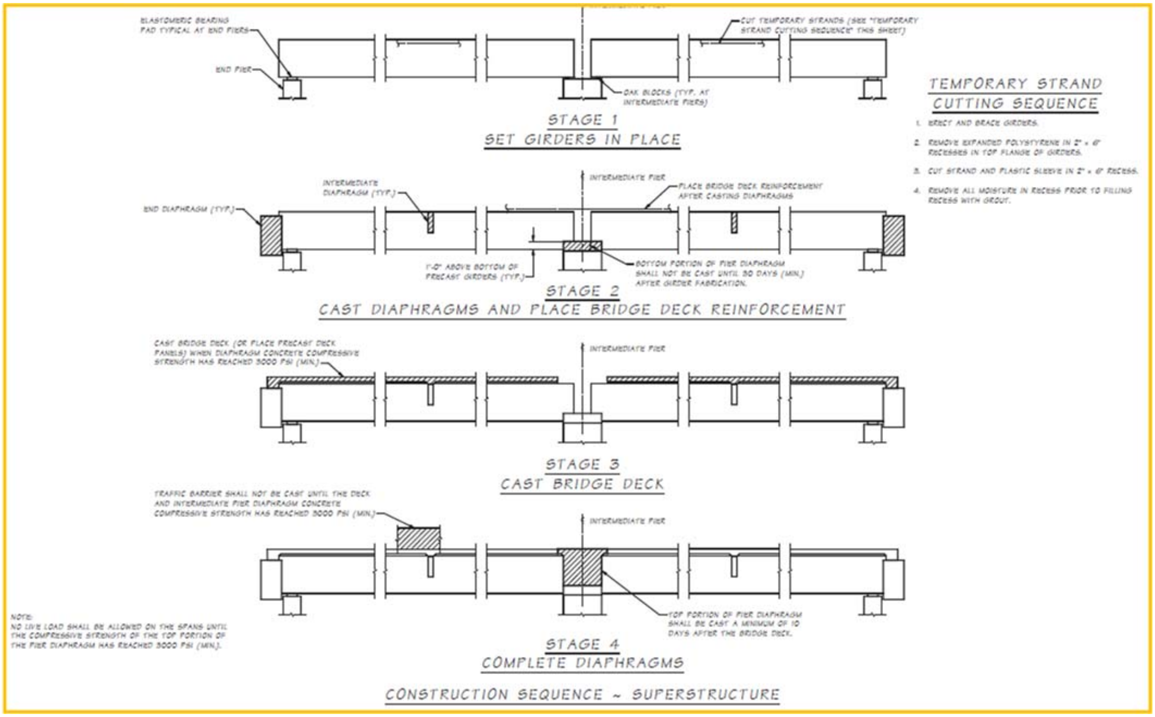


Fig. 10: Construction of a Girder-Slab Bridge Using WSDOT Wide Flange Girders

CLOSING REMARKS

The availability of HSC enables engineers to design bridges with longer span lengths, fewer girder lines, and shallower girder sections. Longer spans permit the use of fewer intermediate piers, which reduces environmental impacts at water crossings and improves traffic safety at locations with high traffic congestion. Fewer girders resulting from increased girder spacing reduce fabrication, transportation, and erection costs.

Long span precast prestressed concrete girders are economical and effective for rapid bridge construction. Longer span girders require fewer intermediate piers and fewer traffic disruptions during bridge construction while maintaining higher quality and long-term performance.

The revised structural efficiency factor takes into account the span length and the unit weight of girders. The revised structural efficiency promotes the use of innovative materials such as UHPC and LWC for precast pretensioned girder bridges.

REFERENCES

1. AASHTO-LRFD Bridge Design Specifications, 5th Edition, 2010.
2. AASHTO-LRFD Bridge Construction Specifications, 5th Edition, 2010.
3. Bridge Design Manual, Publication No. M23-50, Washington State Department of Transportation, Bridge and Structures Office, Olympia, Washington, 2010.
4. Lwin, M. and Khaleghi, B. Time-Dependent Prestress Losses in Prestressed Girders using High Performance Concrete. Journal of Transportation Research Board, No 1594, Washington D.C.
5. Lwin, M. and Khaleghi, B. Proposed Simplified Method for Shear Design of Prestressed Bridge Girders Using LRFD Specifications - Journal of Transportation Research Board, No. 1688, Washington D.C.
6. Seguirant, S.J., "New Deep WSDOT Standard Sections Extend Spans of Prestressed Concrete Girders", PCI JOURNAL, V. 43, No. 4, July-August 1998.
7. Seguirant, S. J., Brice, R. Khaleghi, B., High Performance Precast Prestressed Concrete Girders in Washington State. PCI JOURNAL, Vol. 48. No. 2, March/April 2003
8. Seguirant, S. J., Brice, R., Khaleghi, B., Flexural strength of Reinforced and Prestressed Concrete T- Beams. PCI JOURNAL, Vol. 50, January/February 2005
9. Seguirant, S. J., Brice, R., Khaleghi, B., Making Sense Of Minimum Flexural Reinforcement Requirements: Part 1 - Reinforced Concrete Members, PCI JOURNAL, Vol. 55, Number 3, Summer 2010.
10. Use of Precast Bridge Members in Area of High Seismicity. Journal of Transportation Research Board, No 2131, Structures 2009.
11. Seguirant, S. J., Brice, R., Khaleghi, B., Design optimization and Prestressed Concrete T- Beams. PCI JOURNAL, Vol. 50, January/February 2005
12. Mijia Yang, Pizhong Qiao, F. ASCE, David I. McLean, M.ASCE and Bijan Khaleghi, Effect of Intermediate Diaphragms to Prestressed Concrete Bridge Girders in Over-height Truck Impact, PCI Journal, Winter 2010, PP 58-78.
13. Michael A. Rosa, John F. Stanton, Marc O. Eberhard, Improving Predictions for Camber in Precast, Prestressed Concrete Bridge Girders, Washington State Transportation Center (TRAC) Agreement T2695, Task.