NEW DEVELOPMENTS IN PRECASTCONCRETE BRIDGES IN WASHINGTON STATE

Bijan Khaleghi, PhD, SE Washington State Department of Transportation, Olympia, WA

ABSTRACT
A complete review of Washington State Department of Transportation (WSDOT) long-span precast pretensioned girders, trapezoidal Tub girders, spliced-girders, and precast slabs are presented in this article. Design criteria, detailing, shipping and handling issues of using precast members in bridge construction are discussed. A comprehensive review of the recent changes to the AASHTO Load and Resistance Factored Design (LRFD) Bridge Design Specifications is presented and WSDOT's design policy improving design, constructability, and durability of concrete bridges are presented. The use of High Performance Concrete (HPC), High Strength Concrete (HSC) in the fabrication and construction of prestressed concrete girders have resulted in economical bridge designs with longer span lengths, increased girder spacing and shallower girders. The enhanced strength and material parameters of HSC result in improved long-term mechanical properties, early high strength, volume stability, reduced creep and shrinkage of concrete.

Keywords: Bridge, Concrete, Prestressed Girder, HPC, HSC, LRFD, Span Capability

INTRODUCTION

The development of long span precast prestressed girders has allowed bridge designers to solve the problem of lengthening spans using construction materials they prefer. Long span prestressed girders eliminate the need for falsework, reduce on-site construction activities, reduce environmental impacts at water crossings, and minimize hazards, delays, and inconveniences to the traveling public. Use of precast spliced-girders provides structurally efficient bridges and a greater economy.

High performance concretes with higher compressive strengths have become economically viable for use in a wide variety of prestressed girder applications. Prestressing plants in many regions possess the necessary ingredients, curing process and the expertise required to consistently manufacture high strength concrete with the strength of concrete at transfer in the range of 7.5 ksi to 8.5 ksi, and 10.0 ksi at 28-day. High performance concrete with compressive strength of 4.0 ksi at 28 days is typically used for all WSDOT bridge decks.

PRECAST PRETENSIONED GIRDERS

Majority of bridges in Washington State are prestressed girder bridges. In Washington State, the use of prestressed I-girders started in the 1950's. Since then WSDOT has developed standard girders for composite and non-composite sections to facilitate economical design and construction. The complete description of standard prestressed girders and their span capability is presented in WSDOT Bridge Design Manual (BDM) [1], and could be downloaded from WSDOT website at: http://www.wsdot.wa.gov/eesc/bridge/index.cfm. Both AASHTO LRFD Bridge Design Specifications[2] and BDM are used for the design of prestressed girders.

In 1997, long span deep prestressed girders[3] in both pretensioned and post-tensioned spliced-girders versions were added to the WSDOT inventory. In 2001, a newly developed pretensioned trapezoidal tub girder, commonly called "bath-tubs", was adopted. In 2004, wide flange pretensioned I-girders[4] were added.

The cross sections of deep prestressed girders and pretensioned trapezoidal tub girders used for composite superstructures are shown in Figures 1 and 2 respectively. The span capabilities for these types of girders are presented in reference 1.
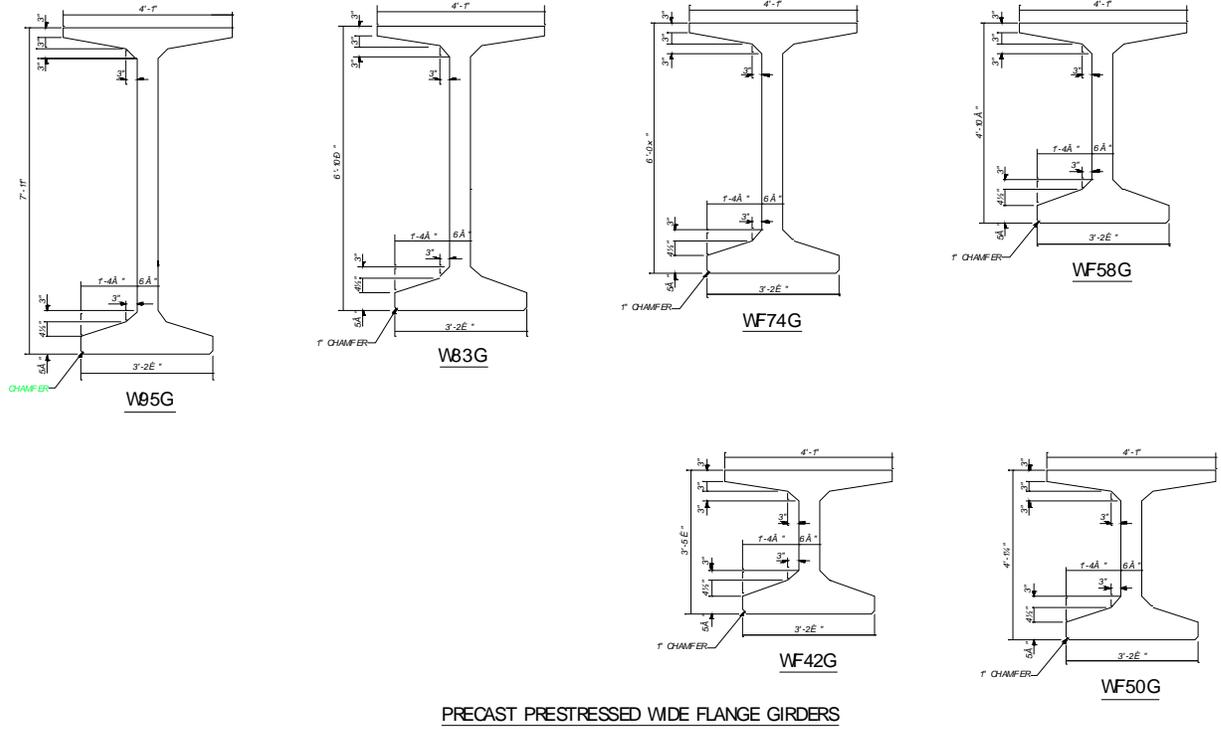
PRECAST PRESTRESSED WIDE FLANGE GIRDERS

Fig. 1 WSDOT Wide Flange Prestressed Girders
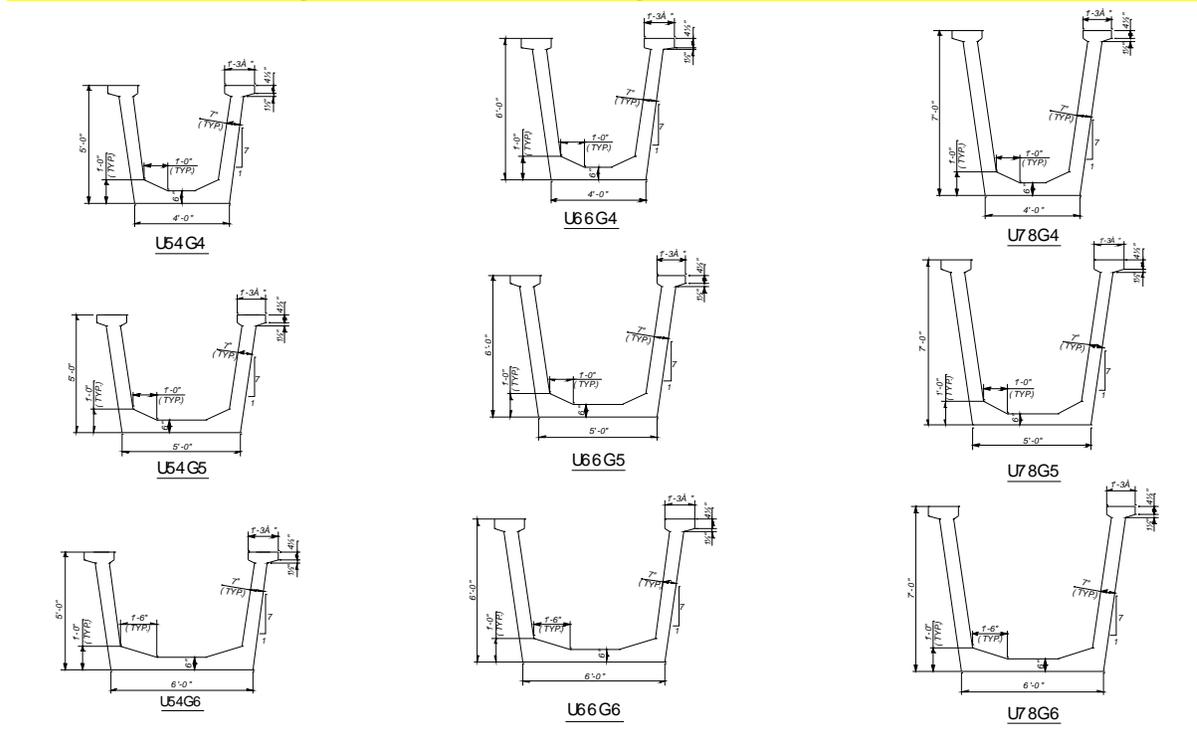

Fig. 2. WSDOT Trapezoidal Tub Girders

ADVANTAGES OF HSC

High strength concrete allows longer span lengths, using standard cross-sectional shapes. An increase in concrete release strength from 5.5 to 7.5 ksi at transfer along with the use of 0.6 in. diameter strands allowed an increase in span length of approximately 20% for all WSDOT Prestressed I-girder series[4].

Concrete strength at the time of prestress transfer has a significant effect on girder spacing. For a span length of 155 ft, the girder spacing can be increased from 6 ft to 8 ft by increasing the concrete strength at release from 5.5 ksi to 7.5 ksi. For this case, the girder spacing can be increased by 33%. A practical implication of this is, for a 60 ft wide slab with 3 ft overhangs, that an entire line of girders can be eliminated from the bridge. Similarly, for WSDOT Tub girder, the use of 7.5 ksi concrete as compared to 5.5 ksi concrete can increase girder spacing from 6 ft to 10 ft for a typical span length of 80 ft.

The use of shallower prestressed girders becomes extremely important, specifically when vertical clearance is a design constraint. In addition to gaining vertical clearance, using the smaller girder section provides savings in the cost of fabricating and shipping the girders.

In advocating shallower girders and wider girder spacing, some concerns may be expressed regarding time-dependent deflections and/or camber. The use of shallower sections may require more attention in design and detailing to reduce creep and shrinkage. For example, bridge designers may wish to incorporate top temporary strands or other design features to negate time-dependent effects.

POST-TENSIONED SPLICED GIRDERS

The post-tensioned precast spliced girders allow longer span lengths compared to that of conventional precast pretensioned girders. Post-tensioned spliced-girders have proven to be a cost-effective structural system for medium to long span bridges[5].

The CIP closures for spliced-girders are recommended to be located away from the point of the maximum moment to minimize flexural stress across the joint. The closure dimensions must be large enough to allow post-tensioning tendon duct splicing, but should be short enough to minimize the effect of lower strength and younger concrete used for the closure compared to the rest of the girder. A 2 ft CIP closure is typically used for WSDOT standard spliced girders.

Providing alternative designs to long span prestressed girders based on the limitations in shipping weight and prestressing may eliminate the excessive cost through competitive biding[1]. Currently the alternative design requirement criteria established by WSDOT are as follows:

Bijan Khaleghi "2007 Concrete Bridge Conference"

1. Prestressed girders with shipping weight of less than 156 kips should be designed and detailed as one-piece pre-tensioned prestressed girders.
2. Prestressed girders with shipping weight between156 and 200 kip should be designed and detailed for both pre-tensioned and post-tensioned spliced-girder alternatives.
3. Prestressed girders with shipping weight exceeding 200 kip should be designed and detailed as post-tensioned spliced-girders only.

PRECAST DECK PANELS

Speed of construction in some cases requires the use of (Stay-In-Place) SIP deck panels for bridge deck construction. The minimum required slab thickness is 8.5 in., including 3.5 in. for SIP deck panels and 5.5 in. for CIP topping. Transverse cracking at the joint between adjacent panels and longitudinal cracks along the length of the girder at the edge of SIP deck panels have been observed. Longitudinal cracking is probably the most significant problem associated with the use of SIP deck panels because it can result in a reduction in deck stiffness over the girders that could compromise the deck's load-transfer mechanism. To minimize the negative consequences of using SIP deck panels. WSDOT limits the use of SIP deck panels for bridge construction as follows[1]:

1. Simple span precast prestressed I-girders and trapezoidal Tub girders.
2. Positive moment region between the points of contraflexure of continuous spans made with prestressed girders.
3. Continuous spans of precast spliced girders with longitudinal post-tensioning.

In bridge widening and staged construction, the SIP deck panels may not be used in the bay adjacent to the existing structure because of the requirement for CIP concrete closure. Use of SIP deck panel for these cases may prevent the unrestraint movement of adjacent girders and placement of concrete closures. SIP deck panels may not be used on steel plate girder of steel box girders.

CAST-IN-PLACE DECK SLABS

Performance and longevity of concrete deck slabs have always been a major concern for bridge deck slabs. Although the mix design, placing and curing of cast-in-place concrete deck slabs have significantly been improved in recent years, the inspection of WSDOT bridge decks indicates that cracking is still apparent in bridge decks. To improve the durability of bridge decks, the following revisions are made to the use of epoxy coated reinforcement in bridge structures:
- Epoxy coated reinforcement shall be specified for both top and bottom layer rebar of all bridge deck slabs without longitudinal post-tensioning.
- Epoxy coated reinforcement shall be specified for only top layer rebar of bridge deck slabs with longitudinal post-tensioning.

The above recommendations are applicable to all prestressed girder, concrete box girder, and steel girder bridges with concrete deck slabs. It should be noted that a combination of black bars and epoxy coated bars have successfully been used in WSDOT bridge deck slabs without any adverse effects. This could be related to the concrete mix or the environmental conditions that deck slabs are exposed to.

WSDOT is also considering increasing the minimum slab thickness from 7.5 in. to 8 in. to allow larger size aggregates for the slab concrete mix. It is believed that increasing the solid to paste ratio in concrete mix reduces concrete deck cracking.

RESTRICTION ON CAST-IN-PLACE CONCRETE SUPERSTRUCTURES

The performance and longevity of CIP concrete box girder bridges have recently been a major concern for bridge structures. Cracking in CIP reinforced box girders are flexural in nature and an inherent part of reinforced concrete design. CIP concrete box girders are designed for ultimate strength and checked for distribution of reinforcement for service conditions and control of cracking. This means that concrete is allowed to crack under applied loads but cracking is within the code prescribed limits. Open cracks in box girders result in rebar corrosion and concrete deterioration, affecting the longevity of bridge superstructures. Requiring post-tensioning for CIP box girders eliminates both cracks and corrosion, and improves structural performance. WSDOT has recently set the following design policy to restrict the use of CIP concrete box girders for bridge superstructures:

This design policy is to restrict the use of cast-in-place (CIP) reinforced concrete (RC) box girders for WSDOT bridge superstructures. Listed below are guidelines to follow when considering CIP box girders for bridge superstructure.

1. Post tensioning shall be considered for all new CIP reinforced concrete single-span or multi-span box girder bridges. Partial Prestressing shall not be considered for design of CIP concrete box girders.
2. CIP reinforced concrete box girders could still be used for widening of existing reinforced concrete box girder bridges.
3. CIP reinforced concrete box girders may be used for bridges with tight curvatures or irregular geometry.
4. Epoxy coated reinforcement shall only be used for top layer slab reinforcement if post-tensioning employed.

The above requirements apply equally to CIP reinforced concrete T-beam and slab bridges. These types of superstructures, however, are not encouraged for bridge superstructures.

SEISMIC DESIGN REQUIREMENTS

The most common types of connections for precast prestressed girder bridges are fixed connections for high seismic zones, and hinge connections for moderate seismic zones.

Precast columns could be used if monolithic moment resistant connections that meet seismic design and detailing requirements are provided.

Monolithic connection between the superstructure and substructure components is the key to seismic resistant precast concrete bridge systems[1]. Lack of monolithic action causes the column tops to behave as pin connections resulting in substantial force demands on the foundations of multi-column bents, particularly in areas of moderate to high seismic activities. Developing a moment connection between the superstructure and substructure reduces the moment demand at the base of the column.

The essence of a seismic resistant connection is to transfer the plastic moment demands at the top of the column into the superstructure without yielding either the connection itself or the girder ends. To achieve this, both the connection and the girder ends must be designed to provide a design strength exceeding the required strength from the forces transferred. The connection should also be detailed to ensure adequate distribution of the longitudinal moment from the top of the column to girders. Fig. 3 shows a typical monolithic moment resistant connection used for WSDOT precast girder bridges.
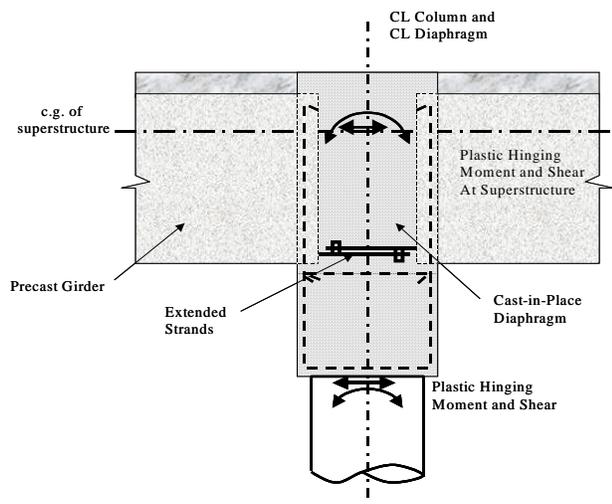

Fig. 3:   Typical Moment Resistant Connection

The connection shown in Fig. 3 is for continuous spans with fixed moment resistant connection between super and substructure at intermediate piers. Cast-in-place diaphragm is completed in two stages to ensure precast girder stability after erection, and completion of diaphragm after slab casting and initial creep occurs. Adequate number of extended strands and reinforcing bars are provided to ensure performance of the connection in seismic events. The design assumptions for fixed diaphragms are:

1.   All girders of adjoining spans are the same depth, spacing, and type.

2. Provide reinforcement for negative moments at intermediate piers in the deck due to live loads and superimposed dead loads of traffic barrier and pedestrian walkway. WSDOT designs girders as simple span for both dead and live loads.
3. Determine resultant plastic hinging forces at centroid of superstructure.
4. Determine the number of extended strands to resist seismic positive moment. Extended strands shall be sufficiently developed into the CIP diaphragm to develop the required seismic positive moment.
5. Design the CIP diaphragm stirrups to resist the resultant seismic forces at centroid of diaphragm.
6. Design longitudinal reinforcement at girder ends for interface shear friction.

Extended strands at the end of prestressed girders are used for continuous spans at diaphragms, and are not applicable to continuous spans using hinge diaphragms. The effect of time dependent positive moments from creep and shrinkage should be considered for more accurate determination of the positive moment capacity. A minimum of 4 extended strands is provided regardless of design requirements.

Precast girders are often supported on elastomeric bearing pads at end piers. Semi integral cantilever abutments are used for shorter bridges, and L abutments for longer bridges are typically used for precast girder bridges. Bridge ends are free for longitudinal movement but restrained for transverse seismic movement by girder stops. The bearings are designed to be accessible so that the superstructure can be jacked up to replace the bearings after a major seismic event.

In L-shape end piers, the minimum displacement requirements at the expansion bearing should accommodate the greater of the maximum displacement calculated from a displacement analysis or a percentage of the empirical seat width, N, specified in Equation 1.

$$N = (8 + 0.02L + 0.08H) (1 + 0.000125 S^2) \qquad (1)$$

Where:

$N =$ minimum support length, in
$L =$ bridge length to the adjacent expansion joint, or to the end of the bridge, ft
$H =$ average height of abutment wall supporting the superstructure, ft
$S =$ skew angle of the support measured normal to span, deg

The Seismic Design Criteria permits hinge seat width in "well-balanced frames" to be evaluated as shown below.

$$N = \Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 4 \text{ in.} \qquad (2)$$

HANDLING AND SHIPPING OF PRESTRESSED GIRDERS

During the preliminary design phase, it is prudent to investigate the route for adequate clearance. One common obstacle is vertical clearance of bridges over sag vertical curves. The height of a prestressed girder section sitting on a trailer is of concern when considering overhead obstructions on the route to the jobsite.

Long span prestressed girder bridges may result in increased cost due to fabrication, shipping and erection of girders. The comfortable weight limitation with trucking equipment currently available in Washington State is approximately 156 kips. Product weights of up to 200 kip can be hauled with currently available equipment at a limited rate if the jobsite is in close proximity to the fabrication plant.

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. Once the girder is set in place, the free span between supports is usually increased and can lead to stability problems. When long girders are erected, they should immediately be braced at the ends for wind stability.

The trapezoidal tub cross sections, because of its large width and increased lateral stiffness, do not have a tendency to roll. The shape of its cross section provides a large moment of inertia about its vertical axis. Additionally, it can be lifted from four pick-up points, so the beam's self weight tends to resist end rotations.

EFFECT OF TEMPORARY STRANDS

Another key component in increasing the span capability is the use of temporary top prestressing strands. The introduction of prestressing strands in top flange of girders, which are released prior to placing the deck concrete, have several beneficial effects on the girder design. The temporary top strands are tensioned along with the permanent strands. When the prestress force of the temporary top strands is transferred to the girder, it reduces the compressive stress in the bottom of the girder caused by the permanent strands, thus reducing the required concrete strength at prestress transfer. The temporary top strands also reduce the instantaneous deflection and long-term camber. By reducing the instantaneous deflection, the center of mass of the girder is closer to the roll axis thus improving the girder's stability during lifting and transportation. The reduction in long-term camber results in a reduction in the volume of concrete required for the cast-in-place deck haunches.

Long prestressed girders can become laterally unstable when handled and shipped. 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. Temporary strands are released after erection of girder to its final position and just prior to placing the diaphragm concrete by cutting or burning the strands.

The top temporary strands are used to ensure stability of prestressed girders during shipping

and handling. The timing and releasing sequence of temporary strands are important in both design and construction of prestressed girders. The following sequence shall be considered in the design and detailing of contract plans for prestressed girder bridges when temporary strands are used:

1. Top temporary strands shall be released after erection and bracing of prestressed girders, before casting intermediate diaphragms.
2. The time lapse between the release of top temporary strands and deck slab casting shall not exceed 60 days. Contractor shall submit supporting calculations in case slab casting is delayed beyond 60 days.
3. The "A" dimension calculations should be changed accordingly to reflect the sequence of load application and time intervals. The distance from the top of the slab to the top of the girder at centerline bearing is represented by the "A" Dimension.
4. The revised default time intervals for creep calculations in the PGSuper program, reflecting these modifications, now are:
   - Release of top temporary strands: 90 days
   - Casting Diaphragms: 90 days
   - Casting deck slab: 120 days
   - Weight of diaphragms: 30 days (90 to 120 days)

TIME DEPENDENT PARAMETERS

WSDOT use the refined estimate of time-dependent prestress losses as standard practice for standard precast, pretensioned members with cast-in-place slab subject to normal loading and pretensioned with low relaxation strands. The Approximate Estimate of Time-Dependent Prestress Losses may be employed for calculations of temporary prestress losses.

The final deflection of prestressed girders is taken as the summation of the elastic deflections and the long-term effect of time-dependent parameters at different construction stages. Prestress force produces moment and axial force in the girder tending to bow the girder upward. Girder dead load resists this upward deflection, but is over powered by the deflection due to prestressing and continues to deflect upward due to creep effect. The result for a non-composite girder prior to slab casting is a net upward deflection. The long-term downward deflections after slab casting, in the most part, are compensated for by the long-term upward deflection due to prestressing[7].

To obtain a smooth riding surface on the bridge deck, the deflection due to the weight of slab and diaphragms indicated as "screed camber" is added to the profile grade elevation of deck slab. Many measurements of actual structure deflections have shown that once slab is cast, the girder tends to acts as it is locked in position.

The girder orientation effect accounts for the difference in slope between the roadway surface and the top of the girder. Girders such as I-beams are oriented with their Y axis plumb. Other girders such as U-beam, box beam, and slabs are oriented with their Y axis

normal to the roadway surface. The orientation of the girder with respect to the roadway surface, and changes in the roadway surface along the length of the girder (super elevation transitions) define the girder orientation effect.
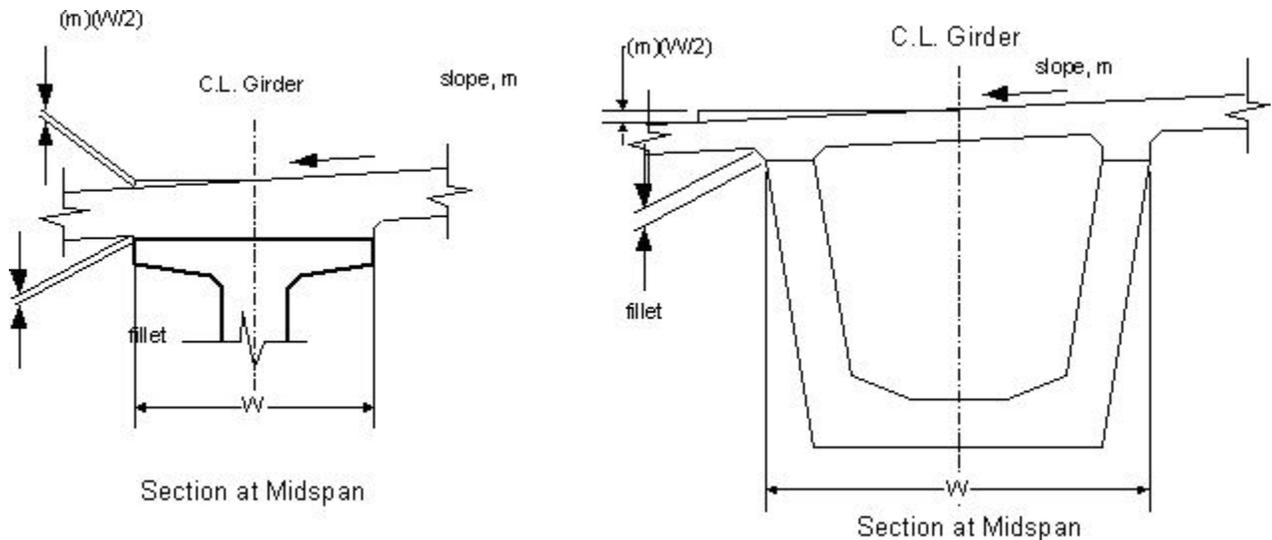

Fig. 4. Wide Flange and Trapezoidal Tubs Girder Orientation

DESIGN CRITERIA

WSDOT designs prestressed girders as simple span for both single and multiple spans with continuous slab at intermediate piers. Continuity deck reinforcement at the intermediate pier is designed for negative moments due to the live load and the superimposed dead loads. Girders are required to support their self-weight, the weight of the cast-in-place slab, formwork, and diaphragms during slab casting. By designing and constructing the bridges as simply supported members, connection and support details are greatly simplified. This results in an ease of construction that provides an economical construction with a long service life.

The vehicular live load is taken as the AASHTO LRFD HL-93 notional truck with dynamic load allowance and live load distribution factor as required by the specifications. When distribution factors are computed by the lever rule, the multiple presence factors are taken as required by the LRFD Specifications with the exception that the multiple presence factor for one lane is reduced from 1.2 to 1.0 per WSDOT design practice.

FLEXURAL RESISTANCE

The ultimate flexural capacity is based on the AASHTO LRFD specification. For prestressed girder bridges the strain compatibility approach is used if more precise calculations of flexural capacity are required[6].

Article 5.5.4.2 of the AASHTO LRFD Bridge Design Specifications classifies concrete members as "prestressed" and "non-prestressed" and assigns resistance factors accordingly. This classification does not properly account for the difference in quality of workmanship between precast and cast-in-place construction. The quality of workmanship of cast-in-place post-tensioned construction does not compare to that of precast construction and, as such, does not warrant the same resistance factor. Concrete members will be classified as "precast" and "cast-in-place". The following table summarizes the flexural resistance factors in tension-controlled concrete members.

Flexural Resistance Factor for Tension-Controlled Concrete Members:

		Precast Members	Cast-in-Place Members
Conventional Construction (other than Segmentally constructed Bridges)	Mild Reinforcement	1.0	0.9
	Prestressed	1.0	0.9
	Spliced Girders	0.95	

For members in the transition zone between tension-controlled and compression-controlled, the flexural resistance factor shall be taken as follows:

For precast members:

$$0.75 \leq \varphi = 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) \leq 1.0 \tag{3}$$

For cast-in-place members:

$$0.75 \leq \varphi = 0.650 + 0.15\left(\frac{d_t}{c} - 1\right) \leq 0.9 \tag{4}$$

For precast spliced girders with cast-in-place closures:

$$0.75 \leq \varphi = 0.616 + 0.20\left(\frac{d_t}{c} - 1\right) \leq 0.95 \tag{5}$$

For precast spliced girders with cast in-place closures the flexural resistance factor is taken 0.95 for strains greater than or equal to 0.005, and transition down to 0.75 for strains less than or equal to 0.002 This approach is very similar to the one taken for flexural resistance factor of segmental construction in Article 5.5.4.2.2.

The flexural resistance factor for compression controlled members, regardless of the method of construction, continues to be taken as 0.75. This is due to the limited use of precast compression-controlled members in bridge construction. However, with the increasing demand for precast compression members for accelerated bridge construction, the flexural resistance factor for compression controlled members may soon be revised.

CONCLUSIONS

1. The use of HSC in prestressed girders permits longer spans for prestressed girder bridges. Longer spans require fewer intermediate supports, reducing environmental impacts at water crossings and improve traffic safety at locations with high traffic congestion. .

2. Prestressed girders made with HSC, because of their higher capacities, can also permit wider girder spacing. This can lead to lower construction costs through fabrication, shipment and erection of fewer precast girders, which minimizes the hazards and inconveniences associated with shipping large precast girders through the highway system.

3. The precast prestressed concrete bridge system is an economic and effective system for rapid bridge construction. Precasting eliminates traffic disruptions during bridge construction while maintaining quality and long-term performance.

4. Precast bridge with monolithic connections meeting the AASHTO LRFD seismic design and detailing requirements could safely be used in seismic zones.

REFERENCES

1. Bridge Design Manual, Publication No. M23-50, Washington State Department of Transportation, Bridge and Structures Office, Olympia, Washington, 2007.
2. AASHTO-LRFD Bridge Design Specifications, forth Edition, 2007.
3. Seguirant, S.J., "New Deep WSDOT Standard Sections Extend Spans of Prestressed Concrete Girders", PCI JOURNAL, V. 43, No. 4, July-August 1998.
4. Seguirant, S.J., Brice, R.A, Khaleghi, B. High Performance Pretensioned Girder Concrete Bridges in Washington State. PCI JOURNAL, Vol. 48, March/April 2003
5. Jugesh Kapur and Bijan Khaleghi Post-Tensioned Spliced Girder Bridges In Washington State, PTI Journal, Vol. 1, June, 2002
6. Seguirant, S.J., Brice, R.A, Khaleghi, B. Flexural strength of Reinforced and Prestressed Concrete T- Beams. PCI JOURNAL, Vol. 50, January/February 2005
7. Myint Lwin and Bijan Khaleghi Time-Dependent Prestress Losses in Prestressed Girders using High &Performance Concrete. TRB 1594, Washington D.C.