# Prefabricated Decked Girders for Accelerated Bridge Construction in Washington State

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## ABSTRACT

*Prefabricated bridge elements and systems (PBES) offers significant cost and time savings, improved safety, and convenience for traveling public. Precast/prestressed concrete decked girders are prefabricated bridge systems that are often used for accelerated bridge construction (ABC). Precast/prestressed decked concrete girders are composed of concrete bulb-tee girder with an integral deck that is cast and prestressed with the girder. A thin cast-in-place concrete slab with minimal reinforcement placed over the top surface is used for mainline bridges to improve the long term performance of precast decked bridges.*

*This paper describes the optimized cross sections of precast decked system for long span bridges with PBES/ABC consideration. Design, fabrication, transportation, construction of precast decked systems is investigated. This paper addresses factors such as the connections between adjacent units, performance of longitudinal joints, continuity for live load, and the benefit of cast-in-place slab on decked members..*

**Keywords:** Bridge, Precast, Decked Members, Optimizations, Connections

## INTRODUCTION

The current use of high performance concrete (HPC) in the fabrication of prestressed concrete girders has resulted in improved economy through the use of longer spans, increased girder spacing, and shallower superstructures. HPC also improves durability and resistance to cracking, while decreasing permeability and the effects of volume change due to shrinkage and creep. The design of high performance precast, pretensioned concrete girders has presented several new challenges, including difficulties in the fabrication, shipment, and erection of long, slender girders.

The recent development of long-span precast, prestressed concrete girder standards has allowed WSDOT and other bridge owners to extend the span capability of the construction material they prefer to use. Long-span precast, prestressed concrete girders can eliminate the need for falsework, reduce on-site construction activities and schedules, reduce environmental impacts at water crossings, and minimize hazards, delays, and inconvenience to the traveling public.

High performance/high strength concrete has recently become a standard material for the fabrication and construction of long-span precast, prestressed concrete girder bridges in Washington State. Girder concrete strengths of 7.0 ksi at prestress transfer and 9.0 ksi at 28 days 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].

## WSDOT STANDARD GIRDERS

In Washington State, the use of prestressed I-girders started in the early 1950s. At that time, construction of highways and freeways was greatly accelerated under the Interstate Highway Program. The challenge was to quickly and cost effectively build grade separations at highway crossings. The economy, quality of fabrication, and ease in construction of pretensioned I-girder bridges met the challenge.

Today, over 80 percent of new highway bridges in Washington State are prestressed concrete girder bridges. The current WSDOT standard pretensioned I-girder designations are WF36G, WF42G, WF50G, WF58G, WF66G, WF74G, WF83G, WF95G, and WF100G are shown in Figure 1, and their section properties are listed in Table 1. (References 3,4, and 5)

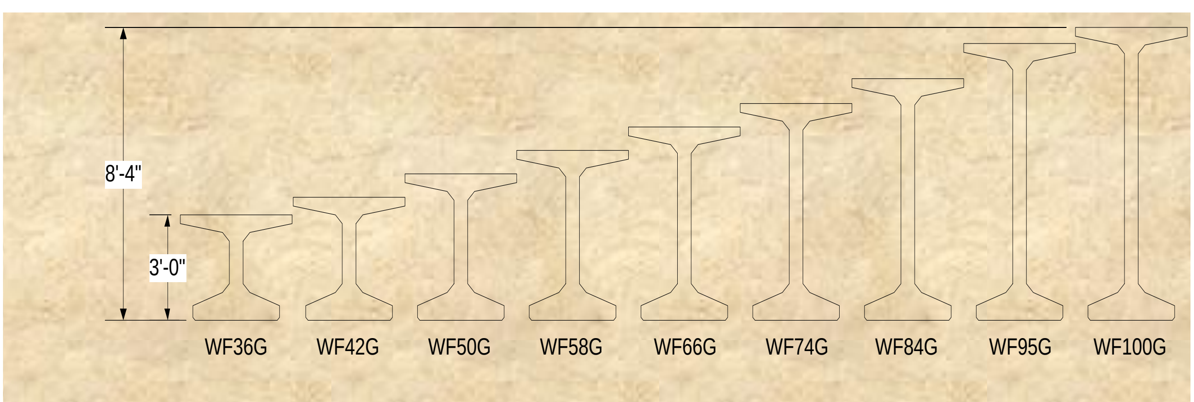

Figure 1. WSDOT Wide flange Pretensioned Girders.

Table 1. Section Properties of WSDOT Wide Flange Pretensioned Girders.

| Girder | Height (in) | Area ($in^2$) | $Y_b$ (in) | $Y_t$ (in) | I ($in^4$) | $S_b$ ($in^3$) | $S_t$ ($in^3$) |
|---|---|---|---|---|---|---|---|
| WF36G | 36.000 | 690.781 | 17.536 | 18.464 | 124771.6 | 7115.0 | 6757.7 |
| WF42G | 42.000 | 727.531 | 20.360 | 21.640 | 183642.4 | 9019.7 | 8486.3 |
| WF50G | 50.000 | 776.531 | 24.151 | 25.849 | 282559.4 | 11699.6 | 10931.2 |
| WF58G | 58.000 | 825.531 | 27.967 | 30.033 | 406265.7 | 14526.6 | 18636.5 |
| WF66G | 66.000 | 874.531 | 31.804 | 34.196 | 556339.2 | 17493.0 | 16268.9 |
| WF74G | 74.000 | 823.531 | 35.657 | 38.343 | 72018.4 | 20594.8 | 19152.5 |
| WF83G | 82.625 | 976.359 | 39.829 | 42.796 | 959395.9 | 24088.0 | 22417.7 |
| WF95G | 94.500 | 1049.094 | 48.905 | 45.595 | 1328994.7 | 29147.8 | 27175.1 |
| WF100G | 100.000 | 1082.781 | 48.274 | 51.726 | 1524912.4 | 31588.9 | 29480.4 |

## WSDOT WIDE FLANGE DECK BULB TEE GIRDERS

The new WSDOT standard pretensioned wide flange deck bulb tee girders span up to 210 ft based on the cross section dimensions and shipping weight limitations. The Deck Bulb Tee cross section varies both in width and depth to accommodate the desired span length and bridge width. The variation in width of top flange is from 4.0 to 8.0 ft, and the variation in depth is from 36 to 100 in., as shown in Figure 2, and the span capability of WSDOT pretensioned wide flange deck bulb tee girders is shown in Table 2.

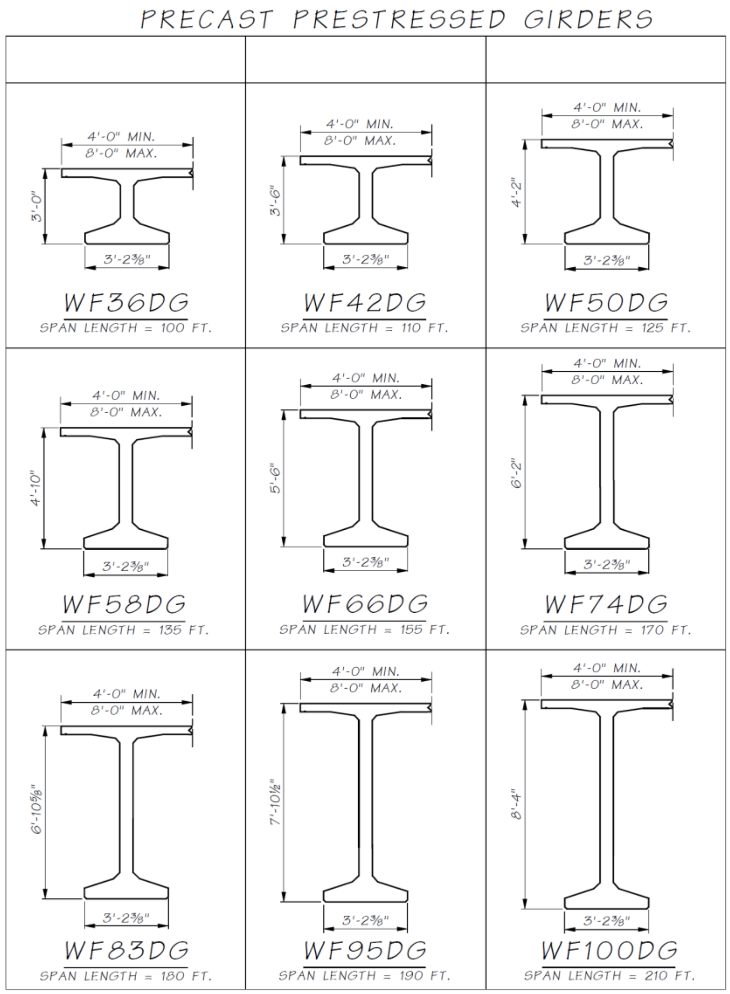


Figure 2. WSDOT Pretensioned Wide Flange Deck Bulb Tee Girders.

Table 2. Span Capability of New WSDOT Pretensioned Wide Flange Deck Bulb Tee Girders

| Girder Type | Flange Width (ft) | CL Bearing to CL Bearing (ft) | Unit Weight (kips/FT) | Shipping Weight (kips) |
|---|---|---|---|---|
| WF36DG | 4.0 | 100 | 0.728 | 72.81 |
| | 8.0 | 80 | 0.998 | 79.84 |
| WF42DG | 4.0 | 120 | 0.782 | 93.88 |
| | 8.0 | 90 | 1.052 | 94.71 |
| WF50DG | 4.0 | 140 | 0.836 | 117.14 |
| | 8.0 | 100 | 1.106 | 110.67 |
| WF58DG | 4.0 | 155 | 0.891 | 138.12 |
| | 8.0 | 120 | 1.161 | 139.33 |
| WF66DG | 4.0 | 165 | 0.945 | 156.00 |
| | 8.0 | 130 | 1.215 | 158.01 |
| WF74DG | 4.0 | 175 | 0.999 | 174.97 |

| | | | | |
|---|---|---|---|---|
| | 8.0 | 140 | 1.269 | 177.78 |
| WF83DG | 4.0 | 190 | 1.054 | 200.30 |
| | 8.0 | 150 | 1.324 | 198.63 |
| WF95DG | 4.0 | 190 | 1.1085 | 210.63 |
| | 8.0 | 155 | 1.378 | 213.68 |
| WF100DG | 4.0 | 205 | 1.163 | 238.41 |
| | 8.0 | 165 | 1.433 | 236.44 |

The design parameters to develop the span capabilities shown in Table 2 are:

- Girder f'ci = 7 5 ksi, f'c = 9 0 ksi
- Topping Slab f'c = 4 0 ksi
- No vertical or horizontal curve
- 2% roadway crown slope
- 6% roadway superelevation for shipping check
- Includes 2" future HMA overlay with density of 140 pcf
- The span capability is controlled by a maximum shipping weight of 240 kips.

Table 3 shows the structural efficiency of the WSDOT wide flange deck bulb tee girders using Guyon's equation[6] for structural efficiency. Guyon's equation is based on the cross-sectional properties of the girder and is expressed as equation 1:

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

where ρ is the efficiency factor, $y_t$ and $y_b$ are the distance from the center of gravity of the section to the top and bottom fibers of the girder, respectively, and r is the radius of gyration of the cross section, and is equal to $I/A_g$. Higher ρ represents higher girder efficiency.

On average, the girders with the wider flanges provide 20 percent more span capability than the current standards. In the future, the current WSDOT standard deck bulb tee girders may be phased out in favor of the new wide flange deck bulb tee girders.

It should be noted that Guyon's equation corresponds to a service load stress optimization of the precast section only, and does not represent the efficiency of the composite girder/slab section, or the flexural strength efficiency.

Table 3: Efficiency of Wide Flange Deck Bulb Tee Girders

| Type | Flange Width (ft) | Depth (ft) | Area ($ft^2$) | $I_z$ ($ft^4$) | $Y_b$ (ft) | $\rho = \frac{r^2}{y_b y_t}$ |
|---|---|---|---|---|---|---|
| WF36DG | 4 | 36 | 654.8 | 114815 | 16.65 | 0.54 |

| | | | | | | |
|---|---|---|---|---|---|---|
| | 8 | 36 | 897.8 | 166637 | 21.25 | 0.59 |
| WF42DG | 4 | 42 | 703.8 | 279712 | 20.6 | 0.90 |
| | 8 | 42 | 946.8 | 389470 | 25.62 | 0.98 |
| WF50DG | 4 | 50 | 752.8 | 444609 | 24.56 | 0.95 |
| | 8 | 50 | 995.8 | 612303 | 29.99 | 1.02 |
| WF58DG | 4 | 58 | 801.8 | 609505 | 28.51 | 0.90 |
| | 8 | 58 | 1044.8 | 835135 | 34.36 | 0.98 |
| WF66DG | 4 | 66 | 850.8 | 774402 | 32.46 | 0.84 |
| | 8 | 66 | 1093.8 | 1057968 | 38.73 | 0.92 |
| WF74DG | 4 | 74 | 899.8 | 939299 | 36.41 | 0.76 |
| | 8 | 74 | 1142.8 | 1280801 | 43.09 | 0.84 |
| WF83DG | 4 | 83 | 948.8 | 1104196 | 40.37 | 0.68 |
| | 8 | 83 | 1191.8 | 1503634 | 47.46 | 0.75 |
| WF95DG | 4 | 95 | 997.8 | 1269092 | 44.32 | 0.57 |
| | 8 | 95 | 1240.8 | 1726466 | 51.83 | 0.62 |
| WF100DG | 4 | 100 | 1046.8 | 1433989 | 48.27 | 0.55 |
| | 8 | 100 | 1306.1 | 1817333 | 54.56 | 0.56 |

WSDOT requires a minimum 5 in. thick class 4000D cast-in-place (CIP) concrete topping with one layer of epoxy coated reinforcement for all decked members used in mainline bridges with average daily traffic (ADT) of 50,000 and more. Use of CIP topping eliminates the potential for reflecting cracking due to the longitudinal joints between decked members. Concrete overlay could be used in lower ADT cases.

Deck replacement for decked members may require shoring of the structure. The CIP toping increases the durability of decked members, and does not require shoring in case of potential deck replacement.

## WSDOT DESIGN CRITERIA AND PRACTICE

WSDOT's prestressed concrete girder bridges are designed using the current AASHTO LRFD[2] Bridge Design Specifications and additional criteria detailed in the WSDOT Bridge Design Manual[1].

## APPLICABLE LIMIT STATES

The following limit states are used in the design of prestressed girders:

- Temporary Stresses
    - At transfer and erection = 1.0 DC, where DC is the self-weight of the member

only.

° At shipping = 1.2 DC or 0.8 DC, where DC is the self-weight of the member only.

- Service I = 1.0 DC + 1.0 DW + 1.0 (LL + IM)
  - For tension outside the longitudinal precompressed tensile zone, and compression stresses, after losses.
  - For compression stresses after losses due to live load plus one-half the sum of effective prestress and permanent loads.
- Service III = 1.0 DC + 1.0 DW + 0.8 (LL + IM)
  - For tension in the longitudinal precompressed tensile zone, after losses.
- Strength I = 1.25 DC + 1.5 DW + 1.75 (LL + IM)
  - For ultimate flexural and shear capacity.

The limit state load modification factor, η, for ductility, redundancy and operational importance is taken as 1.0 for all prestressed girder bridges.

## DESIGN CONSIDERATIONS

Current WSDOT design practice does not allow any tension in the precompressed tensile zone at the Service III limit state. Allowable stresses for prestressing strands are $0.75f_{pu}$ at transfer and $0.8f_{py}$ at the service limit state.

- Section Properties Policy
  - Design Prestressed Girders With Gross Section Properties
- Allowable Tension Stress Policy
  - Design Prestressed Girders for Tensile Stress To 0.0 Ksi.
- Continuity Policy
  - Design Prestressed Girders As Simple Span For all Dead Load And Live Load Regardless Of The Actual Continuity At Intermediate Piers.
- Refined Method of Estimating Prestress Losses
- Design for future overlay

WSDOT design policy for prestressed girders and decked members has resulted in more durable structures. Some of the reasons for the conservative design policies are:

a. Periodical change in Bridge Design specifications. AASHTO design specifications have been changed from allowable stress design (ASD) to (LFD) and to (LRFD).
b. Reserve capacity for girders damaged by over height collisions[8]. The over height load collisions on prestressed girder bridges often results in broken/spliced strands.
c. The zero tension policy ensures that prestressed girders remain uncracked under service load conditions and overloads, resulting in longer service life.
d. Increased shear capacity. Increase in prestressing results in higher shear capacity, with lower θ and higher β.

e. The conservative design assumptions result in longer service life and lesser life cycle cost.
f. Designing for future overlay reduces concerns if bridges are overlaid during their service life.

## DEFLECTION AND CAMBER OF PRESTRESSED GIRDERS

The final deflection of prestressed girders is taken as the summation of the elastic deflections and the long-term effects of time-dependent parameters at different construction stages. Figure 3 shows the idealized deflection diagram for a composite pretensioned girder with temporary top strands.

To obtain a smooth riding surface, the elastic deflection due to the weight of the deck slab indicated as “screed camber” is added to the profile grade elevations of the deck. Many measurements of actual superstructure deflections have shown that once the deck slab is cast, long-term deflections of the composite deck/girder system are minimal. This has been shown to be the case for both single span bridges, and multiple span bridges made continuous with mild steel reinforcement in the deck over the interior piers.

The reason for this stabilization is that once the deck slab becomes composite with the girder, the stiffness of the system is significantly increased. For superstructures made continuous, the continuity further stiffens the composite deck/girder system. Also, the tendency of the girder to creep upward with time is offset by the dead load, plus shrinkage of the concrete in the deck slab.

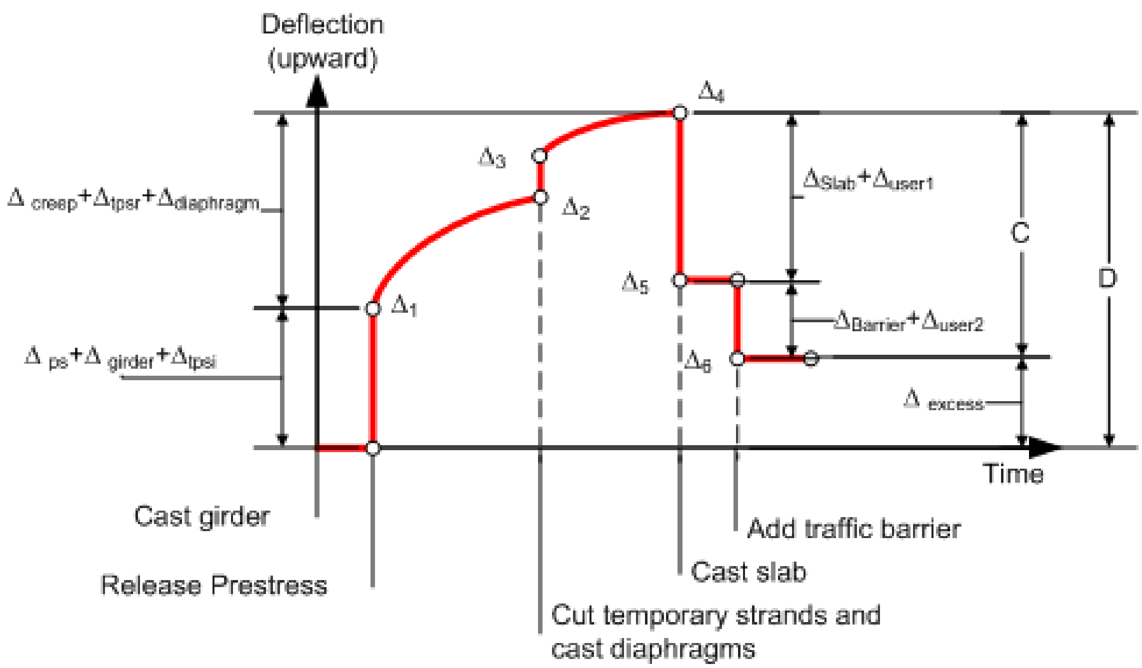


Figure 3. Idealized deflection diagram for WSDOT pretensioned girders

## DESIGN FOR CONTINUITY

WSDOT designs continuous pretensioned girders for both flexure and shear using an envelope of simple span and continuous span behavior. Prestressed girders are designed for positive moments from dead and live loads under service and strength limit states as if the girders were simple spans. Deck reinforcement at intermediate piers is designed for negative moments due to continuous live and superimposed dead loads under the strength limit state.

Connection details and construction sequencing are both simplified by enveloping simple and continuous span designs. Several methods are currently used to create various levels of continuity. These methods are intended to control cracking between the bottom of the girder and the interior pier diaphragm due to positive restraint moments that develop from creep and shrinkage of the precast girder. These cracks reduce the ability of the connection to develop the required negative moments.

One of these methods uses positive moment reinforcement projecting from the bottom flange of the girder into the diaphragm to resist the positive restraint moments. The amount of reinforcement required depends upon the age of the girder assumed in design. Younger girders require more reinforcement than older girders. Thus, not only is the additional reinforcement necessary, but the age of the girders also becomes a potential schedule impact for the contractor.

In general, these negative moments overcome the positive restraint moments caused by shrinkage and creep of the precast concrete girders. The disadvantages of this method are the cost of the splicing operation and the potential schedule impact of placing the deck after the interior diaphragm has reached its design strength.

Although the cost of simple span girders is marginally higher than girders designed for reduced positive moments, the overall construction method has proven to be very cost effective. Also, experience has shown that the conservative nature of the design results in very durable structures with improved overload capacity.

For lateral loads, the connection at the intermediate piers depends upon the seismic zone where the bridge is located. Seismic Zones 3 and 4 are assigned to bridges in Western Washington, while Seismic Zone 2 is generally assigned to bridges in Eastern Washington.

Fixed integral diaphragms (moment resisting) are used at the intermediate piers of continuous prestressed girder bridges located in the higher seismic zones. In this case, prestressing strands projecting from the bottom flange of the girder are anchored into the diaphragm, developing the positive moments necessary for plastic hinging in the tops of the columns. Figure 4 shows WSDOT standard details for the fixed integral diaphragm.

Figure 4. Semi-raised fixed integral diaphragm

Both integral and hinged diaphragms are semi-raised, which allows the dead load of girder, wet slab, haunches, diaphragms, and forms to be carried by the lower crossbeam, while the live and superimposed dead loads are carried by the full depth crossbeam. This type of construction eliminates the need for falsework and temporary supports. Semi-raised crossbeams are cast in two parts: the lower part that provides seat for precast girders, and the upper part that provides superstructure continuity. The semi-raised crossbeam shown in Figure 4 is considered integral moment resisting superstructure-substructure connection capable to resist strength and extreme-I limit state forces.

## HANDLING OF DECK BULB TEE GIRDERS

The ability to handle and ship long prestressed concrete girders is influenced by many factors, including weight, length, height, lateral stability, and mode of transportation[3]. The impact of these variables is discussed in Reference 7. For many years, the WSDOT Standard Specifications have contained provisions for the handling and shipping of prestressed concrete girders.

These provisions did not contemplate the extended spans that have been made possible through the use of HPC. This section discusses modifications made to WSDOT's handling and shipping criteria in light of the use of HPC, as well as some design and field experience gained from the first few bridge projects.

### Weight of Long-Span HPC Girders

For many years, the unit weight of concrete, including reinforcement, used to calculate the weight of precast girders was taken as 160 pcf. However, it was found from measurements of actual W83G girders that the unit weight for this class of girder is closer to 165 pcf.

Measurements taken of the as-cast cross section dimensions were shown to be in very close agreement with the plans, indicating that no significant form spread had occurred. Calculations that include the actual unit weight of concrete, steel, and concrete displaced by steel found that the majority of this difference was the larger quantity of steel typical for this class of girder.

The weight of long-span HPC girders is the primary factor in determining whether a girder can be shipped and how much it will cost. The comfortable net weight limitation with trucking equipment currently available in Washington State is approximately 275 kips.

To provide for the most competitive bidding atmosphere, WSDOT has established the following alternative design criteria for prestressed concrete girder bridge projects:

- Prestressed concrete girders with shipping weights less than 200 kips are designed and detailed as conventional one-piece pretensioned girders.
- Prestressed concrete girders with shipping weights between 200 and 275 kips are designed and detailed for both pretensioned and post-tensioned spliced girder alternatives.
- Prestressed concrete girders with shipping weights exceeding 275 kips are designed and detailed as post-tensioned spliced girders. In this case, a pretensioned one-piece alternative proposed by the contractor will be considered if it can be shown that the girders can be safely shipped and erected.

## Concrete Compressive Strength Requirements

The concrete compressive strength requirements for handling and ship- ping frequently govern the design of pretensioned girders, rather than the demands at the service and strength limit states.

Calculations indicate that this strength level is also adequate for the compressive stresses at the service limit state. Since the girders are designed as simple spans with no allowable tension in the precompressed tensile zone at the service limit state, the concrete strength is irrelevant for this criterion.

Based on requirements for lateral stability, stripping the girder from the form with no temporary top strands would require a concrete strength at release of 7.9 ksi. With six temporary top strands, this could be reduced to 7.4 ksi. Shipping with the temporary top strands in place would require a concrete strength of 8.6 ksi. All of these strengths supersede the strength required by the in-place girder design.

On bridge projects using long, heavy prestressed concrete girders, WSDOT investigates shipping and erection during the preliminary design phase to ensure that the bridge can be reasonably constructed. On some projects with restricted access, girder lengths have become an issue, resulting in a spliced girder design where a one-piece pretensioned girder could otherwise have been used. Height restrictions have generally been circumvented with alternate routes or detours. Where required, WSDOT places a special provision in the project specifications describing the findings of the preliminary investigation. This provision includes information on shipping routes, estimated permit fees, escort vehicle requirements, Washington State Patrol requirements, and estimated permit approval time.

**Lateral Stability**

WSDOT specifies the use of temporary top strands to improve the stability of long slender girders during handling and shipping. These strands are either pretensioned along with the permanent strands or are post-tensioned sometime after the forms are stripped. The choice of pretensioning or post-tensioning is left to the manufacturer, depending on the production scheme to be used.

Pretensioned temporary strands are bonded within the end 10 ft of the girder only, and are unbonded throughout the remainder of the girder length. Post-tensioned temporary strands are anchored with mono-strand anchor plates at one end, are bonded within 10 ft of the other end, and are unbonded elsewhere. Table 4 shows the design parameters for handling and shipping of pretensioned girders.

Table 4: Design parameters for handling and shipping of pretensioned girders.

| Parameter | Lifting from casting bed | Shipping |
|---|---|---|
| Factor of safety against cracking | 1.0 | 1.0 |
| Factor of safety against failure | 1.5 | — |
| Factor of safety against rollover | — | 1.5 |
| Impact factors (upward and downward) | — | 0.8 or 1.2* |
| Roll stiffness of trailer | — | 32,000 kip-in. per radian for Wg < 164 kips<br>40,000 kip-in. per radian for 164 < Wg < 182 kips<br>48,000 kip-in. per radian for 182 < Wg < 200 kips |
| Maximum superelevation | — | 6 percent |
| Girder sweep tolerance | 1/16 in. per 10 ft | 1/8 in. per 10 ft |
| Lifting device or truck support lateral tolerance | 0.25 in. | 1.00 in. |

Mono-strand ducts used for the temporary strands are oversized and sealed to prevent binding or bonding of the strands when cut. Measurements taken of strand retraction after cutting indicate that the system allows the strands to fully relax after release.

The introduction of temporary strands to the top flange also has beneficial effects on the design of prestressed girders. 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 deck concrete and lower dead load moments. Also, the temporary reduction in the eccentricity of the total prestress reduces the compressive stresses in the girder at release and consequently reduces the required concrete release strength.

When pretensioned temporary top strands are used or when the strands are post-tensioned shortly after release, the effect of the strands on the long-term camber should be considered in the design. Most HPC girders can be stripped from the forms without temporary strands, at the expense of higher release strengths. However, many HPC girders will require temporary strands for shipping.

In cases where the temporary top strands are not considered in the design and where their effects on long-term camber would be detrimental to construction, these strands can be post-tensioned shortly before shipping, thus minimizing their effects on camber.

Failure to release the temporary prestress force may have adverse effects on the structural behavior of the girder. WSDOT requires all temporary strands to be visibly flagged before the girders are shipped to the job site, and the bridge plans give instructions for releasing the temporary strands.

The vertical slope of the webs creates one additional handling consideration for precast tubs. The vertical reactions from the picking devices result in a horizontal component of force across the section, tending to fold the webs toward each other. Analysis of the effects of these horizontal reactions on the tub cross section is tricky. In many cases, removable horizontal steel struts are installed between the pick points to resist the horizontal component of thrust

## CONNECTIONS

The most widely used longitudinal connection between precast concrete members is a combination of a continuously grouted shear key and welded transverse ties spaced at intervals from 4 ft to 8 ft on-center. This type of connection is intended to transfer shear and prevent relative vertical displacements across the longitudinal joints. However, there is also a perception of cracking and leakage with this joint, indicating that an improvement was necessary. To avoid the longitudinal cracking and leackage, WSDOT requires a 5 in. minimum thick cast-in-place slab with one layer of reinforcement in transverse and longitudinal directions. The CIP slab allows to eliminate the need for grouted key, and to provide continuity at intermediate piers.
The CIP concrete topping shall be class 4000D concrete with one layer of #4 epoxy coated reinforcement in both transverse and longitudinal direction spaced at 1'-0" maximum. The requirement for 5 in. CIP concrete topping does not eliminate the need for welded ties and grouted keys. This type of cross-section may be considered composite for structural design calculations.

## DESIGN TOOLS

To facilitate the rapid design of prestressed concrete girders in accordance with AASHTO LRFD and WSDOT criteria, the WSDOT Bridge and Structures Office has developed design aids and computer software tools that can be downloaded from the WSDOT Web site at http://www.wsdot.wa.gov/eesc/bridge.

One of the first steps in the preliminary design of a prestressed girder structure is to determine the span configuration, girder size, girder spacing, and level of prestressing. With so many potential combinations, this can be a challenging task. To help arrive at an efficient bridge configuration, WSDOT publishes span capability charts in its Bridge Design Manual. Using the span capability charts, design engineers can quickly compare design alternatives and, thereby, choose a suitable bridge configuration.

## COMPUTER SOFTWARE TOOLS

WSDOT has developed several bridge engineering computer software titles. The most popular title, PGSuper™, is used to design and perform specification compliance checking for pretensioned girders. The flexural design feature determines the number and configuration of prestressing strands and required concrete compressive strengths.

The specification checking feature evaluates girders for compliance with strength, service, and detailing criteria in accordance with AASHTO LRFD specifications and the WSDOT Bridge Design Manual. Girders are also evaluated for overstress and instability during handling and transportation.

To facilitate the design of continuity reinforcement in deck slabs, the QConBridge™ program can be used to determine negative moments due to live load and superimposed dead load. The above programs could be downloaded from:
**http://www.wsdot.wa.gov/eesc/bridge/software/**

## CONCLUSIONS

The availability of high strength concrete and new wide flange deck bulb tee girders enables WSDOT engineers to design bridges with longer span lengths, fewer girder lines, and shallower girder sections, depending on the parameters of a particular project. Longer spans permit the use of fewer supports, which reduces environmental impacts at water crossings and improves traffic safety, especially at locations with high traffic congestion. Fewer girders resulting from increased girder spacing reduce fabrication, transportation, and erection costs. Shallower girders made possible by higher strength concrete create economies in the construction of approach embankments and abutments and improve vertical clearance. Based on the AASHTO LRFD and WSDOT design criteria outlined in this paper, the primary variables limiting the design of wide flange deck bulb tee girders are the required concrete compressive strength at release of prestress, and shipping weights. Also, the concrete compressive strength required for shipping may govern the specified design strength. The AASHTO LRFD Refined Estimates of Time-Dependent Losses method significantly overestimates the long-term prestress losses in wide flange deck bulb tee girders.

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