11-8 DESIGN GUIDELINES OF PRECAST PRESTRESSED GIRDERS

Introduction

Precast prestressed concrete girder bridges can be an economical and preferred solution when bridge projects face constraints such as, but not limited to, the following:

- Falsework restrictions
- Limited construction time
- Limited vertical clearance
- Minimum traffic disruptions
- Environmental impact requirements
- Complex construction staging
- Utility relocation
- Preservation of existing roadway alignment
- Maintaining existing traffic
- Future deck replacement

Furthermore, precast girders can be more effective and economical when the girder quantity is large and details are repeatable. Project Engineers are encouraged to consider precast prestressed concrete girder superstructures as an alternative during the planning phase.

General Design

Precast prestressed concrete girders must be designed to satisfy the requirements of serviceability, strength, and extreme-event limit states specified in the AASHTO LRFD *Bridge Design Specifications* and California Amendments. The fatigue limit state need not be checked for fully prestressed concrete girders.

In general, the design of precast prestressed concrete girders includes the following: establish bridge geometry, select girder section and materials, calculate loads and load effects, determine prestressing force and losses, perform flexural design, perform shear design, check anchorage zone, and estimate camber and deflection.

The precast manufacturer is responsible for the design of the girder for handling, shipping, and erection. However, as part of the shop drawing review process, the designer must confirm that the girder details conform to the original design.

Typical Girder Section

Caltrans standard precast prestressed concrete girder sections are available in Chapter 6 of the Caltrans *Bridge Design Aids*. Standard precast prestressed girder XS sheets are provided at the Caltrans' Standard Detail Drawings website (see References).

Materials

The minimum concrete compressive strength at release, f'_{ci}, and minimum 28-day concrete compressive strength, f'_c, for precast prestressed girders must be determined during design, shown on the plans, and rounded to the nearest 0.1 ksi. The specified concrete strengths, f'_{ci} and f'_c, must not be less than 4.0 ksi. For high strength concrete girders, f'_{ci} may be as large as 7.0 ksi and f'_c may be as high as 10.0 ksi. In special circumstances, the initial concrete strength at release up to 8.5 ksi can be achieved with extended curing time.

The temporary tensile stress at transfer in areas other than the precompressed tensile zone, such as the top flange at girder ends, is permitted to be a maximum of $0.24\sqrt{f'_{ci}}$ (ksi) if bonded reinforcement or prestress strands are provided to resist the tensile force in the concrete. This normally results in a reduced f'_{ci}.

Precast prestressed concrete girder design is normally based on the use of 0.6-inch diameter, 270 ksi low relaxation strands, for economy. Use of 0.5-inch diameter strands is less common. However, $^3/_8$ in diameter strands may be used for stay-in-place precast deck panels. If epoxy coated prestressing strands are required, a note should be shown on the design plans, and the corresponding section of the Standard Special Provisions should be used.

Deformed welded wire reinforcement (WWR) is permitted for use as shear reinforcement in design. WWR conforming to ASTM A497 and Caltrans Standard Special Provisions must be specified, based on a maximum tensile strength of 60 ksi. Reinforcing bars may be used to supplement WWR where needed. Design plans should specify WWR when used.

Flexural Design and Prestressing Force

Flexural design for precast prestressed concrete girders includes design of the girders at the service limit state to satisfy stress limits, followed by a check of the girders at the strength limit state to provide adequate moment resistance under ultimate conditions. The mid-span section is usually subjected to positive moments and designed as simply supported for loads resisted by in a non-composite condition, and as continuous for loads resisted by in a composite condition. In multi-span continuous bridges, the superstructure is generally designed for continuity under live load and superimposed dead loads at bent locations. Except in spliced-girder systems, negative moment reinforcement is required in the deck over the bents to resist these loads. The member at the bent locations is treated as a conventionally reinforced concrete section and is designed to be continuous when determining both negative and positive moments due to loads applied after continuity has been established.

For the service limit state design, flexural tensile and compressive stresses are limited to the values as specified in AASHTO LRFD Article 5.9.4 and California Amendments Table 5.9.4.2.2-1. Prestress losses are calculated per AASHTO LRFD Article 5.9.5. The refined estimates method for time-dependent losses is recommended in final girder design. For the strength limit state design, flexural resistance is provided per AASHTO LRFD Article 5.7.3.2.

To satisfy stress limits, harping of strands or debonding of strands is often used in design and construction.

When harping strands, the centroid of the prestressing strands varies along the member length. In this case, the centroid of the prestressing strands is determined for design purposes and provided on the design plans. In addition, the centroid of harped strands used for design should allow a ±3 in tolerance at girder ends to aid the manufacturer in establishing final strand positions that will accommodate the strand anchorages. This requires the designer to investigate the design for multiple scenarios. A tabulation of different P_{jack} values for individual girders must be shown when P_{jack} varies between girders by more than two strands.

Debonding a portion of the strands at the girder ends may also be used to satisfy stress limits at release. Typically, strands that are straight along the entire length of the girder are selected for debonding to increase fabrication efficiency and economy. The number of these partially debonded strands is limited by the AASHTO LRFD *Bridge Design Specifications* and CA Amendments Article 5.11.4.3. The actual prestressing strand pattern of a girder needs to be shown on the plans when partially debonded strands are specified.

Temporary strands in the top flange of the girder may be used to help reduce the number of debonded strands in the bottom of the girder while maintaining concrete stresses within allowable limits at release. Temporary strands in the top flange of the girder may also be used to resist stresses and enhance stability during shipping. Temporary top strands may be pretensioned and bonded for approximately 10 ft to 15 ft at girder ends and debonded along the middle portion of the girder to avoid reducing the effect on bottom strands in the precompressed tensile zone. Temporary strands should be cut before the cast-in-place intermediate diaphragm or concrete deck is placed. A blockout at the top of the girder at mid-span is normally required to allow cutting top strands.

For continuous precast girder spans on bridges with drop bent caps or for post-tensioned spliced girders joined at bent caps, bottom prestressing strands or reinforcing bars may be extended and conservatively designed to carry positive bending moments due to live load, creep, shrinkage, temperature and other restraint moments. Extended bottom strands or reinforcing bars may be hooked between the girders in the diaphragms at the bent caps to ensure adequate development. These strands and reinforcing bars may also be designed to resist earthquake-induced forces.

For some longer span bridges, girder design may require addition reinforcement to satisfy the strength limit state requirements. However, additional mild reinforcement may not be feasible due to space limitations. In such cases, the number of prestress strands may be increased to sufficiently enlarge its moment capacity at ultimate condition. When the number of strands is

increased for this reason, the pretensioned P_{jack} force can remain unchanged for serviceability by reducing the jacking stress to less than $0.75f_{pu}$. If this approach is taken, a note must be added to the "Prestressing Notes" of bridge plans stating the lower jacking stress as a percentage of the ultimate tensile strength of the prestressing steel.

Shear Design

Shear design is performed using the sectional method specified by the California Amendments 5.8.3.4.2 and 5.8.3.4.3. For skewed bridges, live load shear demand in the exterior girder of an obtuse angle is magnified in accordance with AASHTO LRFD Article 4.6.2.2.3c. A shear correction factor is not required for dead loads. Due to the need for field bending stirrups, #4 or #5 stirrups are preferred, but should not exceed #6.

Interface shear is designed based on the shear friction provisions of AASHTO LRFD Article 5.8.4. For precast girder bridges, interface shear design is considered across the interface between dissimilar materials such as the top of the girder and the bottom of the deck slab, at the interface between girder ends and diaphragms at abutments or bents, or at spliced construction joints for spliced girders. A $^1/_4$-inch intentionally roughened surface or shear key at construction joints must be provided to increase the friction factor and thus enhance the interface shear capacity.

End Blocks and End Splitting Resistance

Due to an increase in girder weight and overall cost, end blocks should only be used where required for such as dapped end girders at inverted-tee bent cap locations or at anchorage areas for post-tensioned spliced precast girders. If an end block is required, it is recommended that only one end block be used per girder. The block length should be 1.0 to 1.5 times the height of the girder. If two end blocks are required, the design plans should specify the minimum end block lengths to provide the manufacturer flexibility to adjust forms.

The end splitting (i.e. bursting) resistance in the pretensioned anchorage zone must be designed based on AASHTO LRFD Article 5.10.10. For spliced precast girders where post-tensioning is directly applied to the girder end block, general zone reinforcement must be provided in the end block per AASHTO LRFD Article 5.10.9.

Deflection and Camber

Per Caltrans Standard Special Provisions, the Contractor is responsible for deflection and camber calculations and any required adjustments for deck concrete placement to satisfy minimum vertical clearance, deck profile grades, and cross slope requirements. Design plans must provide non-factored instantaneous values of deflection components due to deck weight and barrier rail weight. These deflection components are used to set screed grades in the field.

Accurately predicting deflections and camber in girders is difficult because the modulus of elasticity of concrete varies with the strength and age of the concrete. Also, the effects of creep and shrinkage on deflections are difficult to estimate. Therefore, time-dependent camber calculations shown on the shop drawings need not be reviewed for accuracy but should be used for reference only.

Adequate haunch depth must be provided to allow the Contractor to adjust screed grades to meet the designed profile grades. For long span girders or long span spliced girders, the deflection should be checked to ensure that the bridge camber is upward under both short term and long term conditions. The typical section sheet of the bridge plans is to show structure depth as:

(1) minimum structure depth at centerline of bearing at the supports, including girder depth, deck thickness, plus calculated haunch thickness, and
(2) minimum structure depth at mid-span, including girder depth, deck thickness, plus any minimum haunch thickness the designer may choose.

The recommended minimum haunch at mid-span can range from ½-inch to 1-inch by considering cross slope. Girder design must be based on the minimum depth when computing capacity of the section.

Intermediate Diaphragms

Although intermediate diaphragms may not be required per AASHTO LRFD Article 5.13.2.2, they improve distribution of loads between girders and help stabilize the girders during construction. It is recommended that girder lengths over 80 feet usually use one intermediate diaphragm, located at mid-span. Girder lengths over 120 feet should use two or three intermediate diaphragms. Intermediate diaphragms should be used for high skewed bridges. For bridges with skews 20 degrees or less, either normal or skewed intermediate diaphragms may be provided. For bridges with skews greater than 20 degrees, intermediate diaphragms normal to the girders are preferred as they can be staggered.

Spliced Precast Girders

A spliced girder is a long precast prestressed concrete girder that is fabricated in segments. These segments are assembled into a single girder or a continuous girder at the project site. Post-tensioning is generally used to connect the girder segments longitudinally to provide continuity. Additionally, if designed properly, the post-tensioning can be used to create an integral superstructure-to-substructure connection to meet seismic performance requirements.

Design of spliced girders, which combines pretensioned concrete girders with post-tensioning techniques, offers the following benefits:

- Rapid construction with the use of precast elements reduces congestion, traffic delays, and total project cost.
- Longer span lengths reduce the number of piers and minimize environmental impact.
- Fewer joints in the superstructure improve structural performance, including seismic performance, reduce long-term maintenance costs, and increase bridge service life.
- The use of post-tensioning for continuity minimizes bridge superstructure depth, improving vertical clearance for traffic or railway.
- The smaller amount of required falsework minimizes construction impact and improves safety for the traveling public and construction workers.
- Increased girder spacing reduces the number of girder lines and total project cost.

In addition to the requirements of AASHTO LRFD Article 5.14.1.3, some general design and construction challenges include:

- Spliced girder bridge design normally consists of precast pretensioned girders and post-tensioning. Therefore, prestress notes for both pretensioning and post-tensioning must be shown on the bridge plans.
- Precast girders can be either spliced in place using temporary towers or spliced on the ground near the project site and then erected.
- The designer must take into account the construction sequence and staging. Temporary supports and locations must be considered and designed properly as these affect the girder section, span length, and pretensioning and post-tensioning force. Temporary support locations and reactions for each stage of construction must be shown on the contract plans or provided in the Special Provisions.
- The service limit state must be addressed in design considering both temporary and final concrete stresses in girder segments at each stage of pretensioning and post-tensioning as well as all applicable loads during construction. The strength limit state only needs to be considered for the final construction stage.
- Post-tensioning may be applied to precast girders before and/or after placement of the deck concrete. When post-tensioning is applied to the girders both prior to and after placement of the concrete deck, construction is referred to as two-stage post-tensioning.
- In general, one-stage post-tensioning is relatively simple in design and construction and is mostly used with bridge span lengths less than approximately 140 feet. Normally, it is desirable to apply all of the post-tensioning after the deck becomes part of the composite deck-girder section. However, when the full post-tensioning force is applied prior to deck placement, this allows for future deck replacement or can meet other project specific requirements. In this one-stage approach, the post-tensioning force and girder compressive strength (f'_c) are usually higher than that required for post-tensioning applied to the composite section or for two-stage post-tensioning.

- When the bridge span length exceeds approximately 140 feet and/or the precast girders are in segments, two-stage post-tensioning typically results in a more efficient bridge system. The first-stage post-tensioning force is designed to control concrete stresses throughout the continuous span for loads applied before the second stage of post-tensioning. The second stage post-tensioning force is usually designed for superimposed dead loads and live loads. Benefits of the two-stage post-tensioning method include: lower required pretensioning force, more efficient total post-tensioning force for the structure, lower required f'_{ci} and f'_{c} for the precast girder, and better deflection control.
- Prestress losses due to the effects of pretensioning, post-tensioning, and possible staged post-tensioning must be considered. Time dependent software may be used to properly account for prestress losses associated with multiple stages.
- Instantaneous deflections due to post-tensioning at different stages shall be shown on the contract plans. These deflection values are used to set screed grades in the field.
- The post-tensioning tendon profile must be shown on the design plans. Although a specific tendon placement pattern need not be provided in the bridge plans, the designer shall develop at least one workable tendon placement solution at all locations along the span, including at anchorages. Tendon duct size must be based on a duct area at least 2.5 times the total area of prestressing strands. The outside duct diameter shall be less than one half the girder web width per AASHTO LRFD Article 5.4.6.2 and California Amendments 5.4.6.2 and must be shown on the design plans. The maximum number of strands per tendon shall not exceed nineteen 0.6″- diameter strands. In design, a duct size should be assumed and taken into consideration for web shear capacity reduction per AASHTO LRFD Article 5.8.2.9 and California Amendments 5.8.2.9.
- Wet closure joints between girder segments are usually used instead of match-cast joints. The width of a closure joint must not be less than 24 inches and must allow for the splicing of post tensioning ducts and rebar. Web reinforcement (A_v/s) within the joint should be the larger of that provided in the adjacent girders. The face of the precast segments at closure joints must be intentionally roughened or cast with shear keys in place.

References

1. AASHTO LRFD *Bridge Design Specifications*, 6th Edition, 2012, American Association of State Highway and Transportation Officials, Washington, DC.
2. California Amendments (2013) to AASHTO LRFD *Bridge Design Specifications*, 6th Edition.
3. Caltrans *Bridge Design Aids* Chapter 6 (2012)
4. Caltrans' Standard Detail Drawings website (http://www.dot.ca.gov/hq/esc/techpubs/manual/bridgemanuals/bridge-standard-detail-sheets/index.html)
5. Caltrans *Bridge Design Practice* Manual Chapter 8 (2013)

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