High Strength Prestressed Concrete Bridge Girder Performance

This paper reviews the evolution of prestressed concrete girders in North America and correlates historical behavior to the development and application of high strength concrete for bridge girders. Institutional and production issues affecting the use of high strength concrete are discussed and design strategies to further utilize high strength concrete are presented. The paper concludes that high strength concrete girders offer excellent opportunities for extending the effectiveness of these bridge members. Many prestressing plants already have the capacity to produce high strength concrete girders or can readily develop the high strength mix designs. Lastly, modern mix designs with high cement factors and admixtures are likely to retain the high performance characteristics of older girders.

Prestressed concrete bridge girders represent an early and frequently unrecognized use of high strength concrete. Historically, code restrictions often implied or limited the maximum strength of prestressed concrete bridge girders. Compliance with the release strength and economic requirements for daily production resulted in an actual 28-day concrete strength well above the specified design strength. The actual attained strength in many girders is in the range of high strength concrete. This higher strength allows greater design flexibility and has been used by several states and provinces to increase the span length or to reduce...
the number of girders in a bridge.

Industry response to a recent survey suggests that prestressed concrete bridge girders are the predominant element in establishing overall quality guidelines for the advancement of precast concrete operations. The quality control and specification control imposed by state and federal departments of transportation establish the performance characteristics of these members and often set the bounds for production strength. For years this upper bound was assumed to be 6000 psi (41 MPa).

In the 1970s and early 1980s, considerable attention was given to the advancement of cast-in-place, high strength concrete for the columns of multistory buildings. During this time, production concrete strengths of 14,000 psi (97 MPa) were developed and used. Until recently, there has been little corresponding development of high strength concrete for the prestressed concrete industry.

Research at Construction Technology Laboratories (CTL) and Tulane University is examining the performance of decked bulb-tee beams using 10,000 psi (69 MPa) concrete. High strength concrete girder research is also being conducted at the University of Minnesota and the University of Texas. These projects are examining the strength and mechanical properties of high strength concrete for bridge bulb-tee girders. To gain an insight into the probable performance of these girders, a historical assessment was made of prestressed concrete girders and their mix designs.

This paper examines the development of high strength concrete highway bridge girders in North America. The effect of standard specifications and the economics of beam production provide the framework of the current industry operations. Concrete strength gains are examined to determine if high strength concrete is available in existing prestressing operations. Strength gains beyond the traditional 28-day specification limits are examined. The loading and life cycle of a structure are reviewed to determine if the higher strength concrete has an economical advantage. Modern concrete mix designs are examined to assess their impact on durability, and design strategies to utilize high strength concrete are presented.

HIGH STRENGTH CONCRETE

High strength concrete is not clearly defined and is dependent upon the production capacities and practices in various parts of North America. However, high strength concrete may be defined as any concrete with a specified 28-day strength over 8000 psi (55 MPa). Bridge girder design and construction often consider any concrete in excess of 6000 psi (41 MPa) as the transition point to high strength concrete. The definition of high strength concrete in the precast, prestressed concrete industry is also influenced by the need for a high early strength at transfer of the strand prestress to the girder concrete.

The maximum attainable strength of precast concrete bridge girders varies with the quality of available aggregates and with plant production techniques. This paper examines the development of high strength concrete girders from the perspective of the industry capability to manufacture high quality and high strength members. In many locations, the industry is limited by standard specifications that are lower than the industry can fabricate. Efficiency of design and materials requires that the industry capability be used. For example, the CTL/Tulane study found that high strength concrete could be made from local aggregates and could exceed the existing standard state specifications.

Materials for prestressed concrete bridge girders are usually specified by the AASHTO Standard Specifications for Highway Bridges. The AASHTO Specifications affect the strength of the concrete in two ways. First, the Specifications suggest minimum values for the concrete strength at the time the prestress force is transferred into the beam and they limit service level stresses. Second, the AASHTO Specifications limit the concrete compressive strength to 6000 psi (41 MPa) unless special inspection requirements are imposed. Consequently, few states or suppliers have routinely exceeded the 6000 psi (41 MPa) limitation.

The economics of prestressed concrete girder production are dictated by the efficient use of the labor force and plant equipment. In an operation where the production requirements use all of the available forms, a one-day cycle for beam production is common. This cycle begins with removal of the previous day's girder, continues with cleaning the form and placing the reinforcement, and concludes with casting and curing the concrete. The work needs to be completed so that sufficient time remains for the concrete to cure prior to removal of the girder the next day. Twelve to 16 hours are needed for curing the concrete prior to transfer of the prestressing force to the girder.

If production requirements are low and all the stressing lines are not utilized, then the girders may be left in the form to cure for an extra day or two prior to the transfer of the prestressing force. The amount of time available for curing prior to the application of the prestressing force has a marked influence on the type of cement and the strength of the concrete used in the girder. Extended initial curing time reduces the energy requirements for accelerated curing and can lower production costs.

The design specification of the concrete release and design strength further influences the selection of the concrete used in the girders. The higher the initial prestressing force is specified, the greater are the demands on the initial concrete strength. The initial concrete strength may be "high" if it is high in terms of the final specified concrete strength, e.g., 70 percent of the final strength, or if it is high in absolute value, i.e., $f_{ct} > 5000$ psi (34 MPa) at 18 to 20 hours.

Production operations using a one-day work cycle will require a concrete that gains strength very rapidly in the first few hours after casting. The strength gain is obtained by using more cement, Type III cement, lower water-cement ratios, accelerating the concrete cure by using admixtures, and by heating the concrete after the initial set. Multiple-day production cycles can use Type I cement and ambient curing temperature more effectively and may not have to resort to accelerated curing.
HISTORICAL DEVELOPMENT OF HIGH STRENGTH CONCRETE GIRDERS

A few states and provinces, e.g., Texas, Washington and Ontario, routinely use concrete strengths above 6000 psi (41 MPa) in their bridge girders. Ontario, in particular, has developed a bridge design specification to use 8000 psi (55 MPa) concrete. Illinois, Idaho, Montana and Florida use 6000 psi (41 MPa) concrete in their state specifications. More important to the girder production is the specified release strength. This is the concrete strength at the time of transfer of the prestress to the girder.

Table 1 summarizes both the release strength and the design strength for several states, and provides a projected 28-day strength. The projected 28-day strength is based on the assumption that the one-day release strength is 50 percent of the 28-day strength. The one-day value of 50 percent is high for normally cured concrete, but is reasonable for accelerated curing used in the precast concrete industry. It falls between the ranges given by Neville for strength gain for one- and three-day old concrete. This assumption must be modified for conditions of curing, mix design and test methods. It is presented here to indicate the potential range of strength which may be available to the designer. Actual available design strength must allow for the properties and strength variation within the actual concrete mix.

Several important issues are raised in Table 1. First, approximately 4000 psi (28 MPa) of additional strength is projected to be available for design use. Second, if this strength is available, what is the best method to utilize it in design? Fig. 1 shows a typical strength gain curve for a concrete girder produced in Washington State. The specified 28-day strength is 7000 psi (48 MPa) and the actual strength is 9190 psi (63 MPa).

Allowing for a 1.34 x standard deviation reduction in strength to satisfy the specified performance guidelines, this concrete would meet an 8500 psi (59 MPa) design strength, 1500 psi (10 MPa) above the specified strength. This concrete mix did not use a high range water reducing (HRWR) admixture, but did use a low water-cement ratio and a high cement factor.

Mix designs based on a high cement factor and Type III cement underwent a substantial modification in the period from 1972 to 1976. The escalating cost of energy in those years affected the strategy used by precast beam manufacturers to produce concrete and to

Table 1. State DOT concrete strength specifications.

<table>
<thead>
<tr>
<th>State</th>
<th>Specified $f'_c$</th>
<th>Projected $f'_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Idaho</td>
<td>—</td>
<td>5500 to 6000 psi (38 to 41 MPa)</td>
</tr>
<tr>
<td>Montana</td>
<td>5000 psi (34 MPa)</td>
<td>5500 to 6000 psi (38 to 41 MPa)</td>
</tr>
<tr>
<td>Texas</td>
<td>5000 to 6000 psi (34 to 41 MPa)</td>
<td>7800 to 8000 psi (54 to 55 MPa)</td>
</tr>
<tr>
<td>Washington</td>
<td>5000 to 6500 psi (34 to 45 MPa)</td>
<td>6000 to 7000 psi (41 to 48 MPa)</td>
</tr>
</tbody>
</table>

* Projected strength is based on the assumption that the projected 28-day strength is double the release strength.

† The State of Idaho leaves the determination of $f'_c$ to the precast concrete beam producers.

![Fig. 1. Strength gain above the minimum 28-day strength.](PCI JOURNAL)
cure beams. Prior to 1972, most highway girders were made with relatively high quantities of Type III Portland cement. Concrete containing 700 to 900 lbs of cement per cubic yard (415 to 534 kg/m³) were common. Water-cement ratios were kept as low as possible, consistent with concrete placement. The resulting strength gain curves were similar to Fig. 1, with corresponding strength gains above the local 28-day strength specification.

The 1972 oil embargo and sudden escalation in energy costs made Type III cement and the energy costs of accelerated curing significantly more expensive. At the same time, HRWR admixtures became available. Producers began to use less cement and included HRWR admixtures in their mixes. They improved their quality control and used more sophisticated curing procedures in the manufacture of beams. One consequence of these activities was that the concrete could now be designed to “fit the specification curve.”

Fig. 2 shows the strength gain for steam-cured concrete using an HRWR admixture. The strength gain after release brings the 28-day strength to just above the specified 28-day requirement. The supplemental strength gain is substantially less than that found in Fig. 1. The 480 psi (3.3 MPa) strength increase of the lower curve in Fig. 2 is too small to consider for inclusion in substantial changes of bridge design.

The strength gain that is available for a given mix is a function of the mix design and the accelerated curing. Fly ash substitution for cement, as required for many federally sponsored highway projects, further affects the relative proportions required for release and 28-day strength. These effects are discussed in Ref. 9.

Thus, historically, two types of concrete emerged:

1. Concrete with high quality aggregate and high cement factors continued to have substantial strength gain at 28 days.

2. Concrete with lower cement factors and HRWRs had much less strength gain between 18 hours and 28 days.

This behavior led to an investigation of the strength gain beyond 28 days and the performance of newer high strength concrete girders.

PRESTRESSED CONCRETE GIRDER PERFORMANCE

Strength Gains Beyond 28 Days

Since concrete continues to gain strength with time, additional strength gain beyond the 28-day standard may be available. In considering strength gain in excess of 28 days, it is noted that the initial uses of high strength concrete in building design were specified at 56 and 90 days. Since many precast girder bridges do not need their full strength until well past 56 days, a 56-day concrete strength is a pragmatic possibility. Fig. 3 provides an indication of the strength gain in concrete specimens cured for 84 days. Significant additional strength is available in this example.

The post-28-day strength gain is a function of the mix design. As demonstrated earlier (see Fig. 2), accelerated curing and the use of HRWR admixtures may substantially limit any post-28-day strength gain. Mixes that gain their strength from a high cement factor, e.g., Type I cement and/or which use pozzolanic fly ash, are more favorable candidates for supplemental strength gain.

Strength gain of moist-cured cylinders is widely documented for laboratory conditions. Field conditions offer highly variable curing environments and it is logical to question if the laboratory strength gain is available in actual structures. Tests at CTL on a 25-year-old prestressed concrete girder, removed from the Illinois Tollway, provided data indicating that a 5000 psi = 6.895 MPa.
psi (34 MPa) increase in compressive strength occurred over the originally specified 5000 psi (34 MPa) design strength. Tests of two 30-year-old bridge girders in Belgium indicated a 77 percent increase in compressive strength beyond the original specified strength. The actual strength at the time of testing was 13,800 psi (95 MPa).

**Durability**

In addition to strength gain, prestressed concrete girders show excellent durability characteristics. Evaluations of state and federal highway bridge inventory records indicate that prestressed concrete beams outperform alternative highway structural members. The studies also suggest that the girders outperform the decks. This leads to the conclusion that the girders remain suitable for their design objectives, even in those instances when the bridges are being replaced for other reasons.

A portion of this enhanced performance comes from the strength increases in the girder. If modern mixes have cut back on cement and use HRWR admixtures, can we expect concrete to have the same strength increase and durability? To answer this question, a study of actual bridge girder strength gain was conducted.

**BULB-TEE STRENGTH GAIN CALIBRATION**

An assessment of actual strength gain of girders in the field was conducted in 1984. This study correlated the actual compressive strength gain in the girder to concrete test cylinders. Eighteen bulb-tee girders were manufactured for a bridge in Portland, Oregon. The girders were 84 ft (25 m) long and had a specified 28-day concrete strength of 7500 psi (52 MPa) based on moist-cured cylinders. Concrete strength was monitored using laboratory-cured cylinders, field-cured cylinders, cores and Schmidt hammer readings. Data were collected for six months on two of the girders. The bulb-tee girder is indicative of the new generation of highway girders and is made with concrete containing a high dosage of admixtures.

**Laboratory Cylinders**

Seven 6 x 12 in. (150 x 300 mm) cylinders were thermocoupled to the girder to ensure that the cure temperatures were identical to the girder. Two thermocouple-controlled cylinders were tested at 18 hours to validate the detensioning strength. The remaining cylinders were moist-cured in a saturated lime bath at 73.4 ± 3°F (23 ± 1.7°C) until testing. One cylinder was tested at 14 days and two were tested at 28 days.

Twenty-eight additional cylinders were prepared but not thermocoupled to the girder. Twenty-six cylinders were placed immediately adjacent to the controlled cylinders to duplicate the curing conditions as closely as possible. Two cylinders were cured away from the beam at ambient room temperature to record and monitor the concrete strength without heat curing. Fifteen cylinders were moist-cured with the control cylinders and 14 cylinders were field-cured adjacent to the girder. Two field-cured and two moist-cured cylinders were tested at detensioning. Two cylinders from each group were tested when the beams were stripped from the form, at seven, 28, 90 and 180 days, and at shipment on day 40.

**Schmidt Hammer**

The Schmidt impact hammer correlates the concrete strength with the dynamic rebound off the concrete surface. To obtain valid results, the hammer must be correlated to the actual concrete mix. Schmidt hammer readings were taken each time a cylinder was tested. The cylinder was preloaded...
in the test machine to about 30 percent of its strength capacity. The preload ensured that the cylinder was adequately secured. The hammer accuracy and reproducibility were ±15 percent for all tests. Corresponding readings were taken on the girders up to the shipping day. Readings were taken on the thicker portion of the girder, i.e., near endblocks, to avoid dissipation of the impact energy in the thin webs and flanges.

Cores

At 197 days, three cores were taken through the deck of each girder. The cores were taken over the web of the beam. The core drill had an inside diameter of 3/8 in. (87 mm). A 2 to 1 length to diameter ratio was the test objective and corrections were made according to ASTM C42 for the actual core dimensions. Cores were tested dry and no correction was made to core strength based on size difference between the core and the 6 x 12 in. (150 x 300 mm) cylinders.

Calibration Results

Table 2 summarizes the test results and Fig. 4 provides a graphical interpretation of the strength growth. Using the thermocoupled moist-cured cylinders as the project standard, the following observations may be made. The Schmidt hammer overestimated the moist-cured cylinder strength in the first week but provided an accurate indication of the overall strength gain for the girder. This suggests that as the concrete hydration process matures, the Schmidt

![Fig. 4. Correlation of cylinder and impact hammer strength.](image)

<table>
<thead>
<tr>
<th>Time</th>
<th>Beam 1</th>
<th>Beam 2</th>
<th>Beam 1</th>
<th>Beam 2</th>
<th>Beam 1</th>
<th>Beam 2</th>
<th>Beam 1</th>
<th>Beam 2</th>
</tr>
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<tbody>
<tr>
<td>Release</td>
<td>6090</td>
<td>6140</td>
<td>5255</td>
<td>4980</td>
<td>3395</td>
<td>2840</td>
<td>5900</td>
<td>5540</td>
</tr>
<tr>
<td>18 hours</td>
<td>1</td>
<td>1</td>
<td>6210</td>
<td>5370</td>
<td>7305</td>
<td>6930</td>
<td>7370</td>
<td>7410</td>
</tr>
<tr>
<td>4 days</td>
<td>1</td>
<td>2</td>
<td>5990</td>
<td>5670</td>
<td>7740</td>
<td>6920</td>
<td>8850</td>
<td>8550</td>
</tr>
<tr>
<td>storage</td>
<td>1</td>
<td>2</td>
<td>8120*</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7 days</td>
<td>1</td>
<td>2</td>
<td>8080*</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>14 days</td>
<td>Beam 1</td>
<td>Beam 2</td>
<td>9160</td>
<td>9655</td>
<td>7780</td>
<td>7445</td>
<td>9743</td>
<td>8760</td>
</tr>
<tr>
<td>28 days</td>
<td>Beam 1</td>
<td>Beam 2</td>
<td>9655</td>
<td>9655</td>
<td>7780</td>
<td>7445</td>
<td>9250</td>
<td>9050</td>
</tr>
<tr>
<td>40 days</td>
<td>Beam 1</td>
<td>Beam 2</td>
<td>9655</td>
<td>9655</td>
<td>7780</td>
<td>7445</td>
<td>9743</td>
<td>9050</td>
</tr>
<tr>
<td>90 days</td>
<td>Beam 1</td>
<td>Beam 2</td>
<td>8370</td>
<td>8135</td>
<td>7780</td>
<td>7445</td>
<td>9250</td>
<td>9150</td>
</tr>
<tr>
<td>180 days</td>
<td>Beam 1</td>
<td>Beam 2</td>
<td>8370</td>
<td>8135</td>
<td>7780</td>
<td>7445</td>
<td>9250</td>
<td>9150</td>
</tr>
</tbody>
</table>

* Single cylinder test.
† Compressive strength is the average of two cylinders except for (*), which indicates a single cylinder test. Cylinders are typically 6 x 12 in. (150 x 300 mm).
‡ Average of three cores. All cores were tested at 197 days.
§ Thermocouple-controlled for 12 hours followed by moist cure.
** Thermocouple-cured cylinders are used in plots and are given precedence over moist-cured cylinders.

Note: 1000 psi = 6.895 MPa.

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Table 3. Performance of current high strength concrete mixes.

<table>
<thead>
<tr>
<th>Project application</th>
<th>CTL/Tulane bulb-tee girder</th>
<th>CTL/Tulane 24 in. (610 mm) pile</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Initial trial</td>
<td>Final trial</td>
</tr>
<tr>
<td>Target $f'_c$</td>
<td>10,000 psi (69 MPa)</td>
<td>10,000 psi (69 MPa)</td>
</tr>
<tr>
<td>Cement</td>
<td>Type III</td>
<td>Type I</td>
</tr>
<tr>
<td>752 lb/cu yd (447 kg/m³)</td>
<td>755 lb/cu yd (448 kg/m³)</td>
<td>750 lb/cu yd (446 kg/m³)</td>
</tr>
<tr>
<td>Admixtures</td>
<td>HRWR</td>
<td>HRWR</td>
</tr>
<tr>
<td>Fly ash, Class C</td>
<td>Air entraining</td>
<td>Air entraining</td>
</tr>
<tr>
<td>Microsilica</td>
<td>82 lb/cu yd (49 kg/m³)</td>
<td>50 lb/cu yd (30 kg/m³)</td>
</tr>
<tr>
<td>Strength, psi:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Release</td>
<td>9230</td>
<td>5510</td>
</tr>
<tr>
<td>7 days</td>
<td>9550</td>
<td>7900</td>
</tr>
<tr>
<td>28 days</td>
<td>9820</td>
<td>95 lb/cu yd (56 kg/m³)</td>
</tr>
<tr>
<td>40 days</td>
<td>9780</td>
<td></td>
</tr>
<tr>
<td>56 days</td>
<td>9950</td>
<td></td>
</tr>
<tr>
<td>56 days - cores</td>
<td>9700</td>
<td></td>
</tr>
<tr>
<td>169 days - cores</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* The 28-day strengths represent the average of three cylinder tests each cured differently. The actual test results are: steam-cured, $f'_c = 10,460$ psi; air-cured, $f'_c = 11,874$ psi; and moist-cured $f'_c = 11,640$ psi.

Note: 1000 psi = 6.895 MPa.

hammer becomes a more reliable measure of the concrete strength and is comparable to moist-cured cylinders. The field-cured cylinders significantly underestimated the girder strength.

On the basis of the moist-cured cylinders, 1500 psi (10 MPa) additional strength is available at 28 days and as much as 2500 psi (17 MPa) may be available at 180 days. This represents a one-fifth to one-third increase in potential strength that may be available to the designer.

**Eau Gallie Bridge**

The Eau Gallie Bridge in Florida consists of bulb-tee beams made from a concrete mix which includes fly ash as a partial cement replacement. The concrete release strength is based on a two-day initial cure cycle. The fly ash in the concrete mix will gain strength more slowly than Portland cement and the Type I cement has a greater strength increase after 28 days than does the Type III cement.

The girders were pretensioned for 145 percent of their dead load. A cast-in-place deck was applied at the plant. When the deck strength reached 3500 psi (24 MPa), the girder was post-tensioned to its full prestress capacity. These girders present an alternative option for using slower strength gains in prestressed concrete girders and for optimizing the prestressing forces.

**CTL/Tulane Tests**

The mix designs and strength performance for the CTL/Tulane tests are summarized in Table 3. The girder design had a 36-hour prestress transfer and used a two-day turnover cycle. The post-28-day strength gain is lim-
After normal construction sequencing, deferring the application of full live load for 56 days appears quite reasonable. If the extra strength gain can be specified and utilized during the bridge design, then the increased strength due to aging is particularly valuable. One important argument against extending the strength design age is that a beam may be needed on the job site in a very short time. In such a case, waiting for a 56-day strength is not reasonable.

The same situation occurs with 28-day strength. Occasionally, a beam is needed in advance of the full cure time. If there is a successful history of concrete strength gain for the particular mix being used, then girders are installed on the job site on a short schedule and the strength gain is closely monitored. The same condition exists for a 28- and 56-day strength and is a quality control issue.

**DESIGN USE OF HIGH STRENGTH CONCRETE**

High concrete strength can result from high strength concrete mix designs, excess strength due to high transfer strength requirements and extension of the strength design age of the concrete. Having established a number of conditions where high concrete compressive strength is available, it is now necessary to determine if this strength can be utilized in a design. This assessment is done by generating a number of structural designs using standard precast sections.

Flexural analyses were conducted for simple span and continuous span precast, prestressed concrete girder bridges. The parameters used in these designs included girder length, girder spacing and concrete strength. The bridges were loaded with the AASHTO HS-20 truck. The baseline bridge had a concrete strength of 6000 psi (41 MPa) in accordance with the AASHTO Specifications.

Strength increases of 1500 and 2000 psi (10 and 14 MPa) were included to represent potentially available strength increases at 28 and 56 days or obtainable with new mix designs. Strands were stressed to 189 ksi (1.3 GPa) and the load was transferred at 18 hours. Losses were held at 35 ksi (240 MPa) for all beams. Effects of elastic shortening and steel relaxation were included.

Allowable concrete stresses were:

\[
\begin{align*}
 f'_{cl} &= 0.60 f'c \\
 f'_{t} &= \text{no limit established} \\
 f'_{c} &= 0.40 f'c \\
 f'_{t} &= 6/\sqrt{f'_{c}}
\end{align*}
\]

Live load deflections were checked against the AASHTO limits.

In conducting the analyses, the following four assumptions were made:

1. Existing standard bridge girder sections were used. No new design shapes were generated.
2. Only the gross shear in the section was checked. As long as the shear was less than \(8/\sqrt{f'_{c}}\), then stirrups were assumed to be satisfactory to carry the shear.
3. Girders were checked for allowable stress and strength conditions. Any small deficiencies in strength were assumed to be made up by supplemental reinforcement.
4. The sections were assumed to be large enough to install all the reinforcement. A detailed check was not made to ensure that all the reinforcement and prestressing fit into the beams.

Parametric studies were conducted for several Washington State Department of Transportation (WSDOT) standard sections and AASHTO box sections. Results for a WSDOT 12 Series girder is shown in Fig. 6. The WSDOT 12 Series girder is optimized for a 120 ft (37 m) span at a concrete strength, \(f'_{c}\), of 6000 psi (41 MPa). At 120 ft (37 m), the limitations of the design of the girder change from the allowable tension in the concrete limiting the span capacity to the allowable compression limiting the span capacity. If a zero tension criterion is used in the bridge design, the girder span capacity is less than the other two criteria.

The parametric curves are typical of all the bridge girder analyses results and allows several conclusions to be drawn. The specific span capacities would vary for different girder configurations.

1. A zero tension criterion substantially limits the benefits gained from higher strength concrete and limits the span capacity of a WSDOT 12 Series girder up to about 130 ft (40 m).
2. The allowable tensile stresses limit the girder span capacity up to about 120 ft (37 m) and to 135 ft (41 m) for higher strength concrete.
3. At the longer length and at the closest girder spacing, the allowable compressive stresses limit the beam capacity. In these ranges, additional compressive capacity is a major benefit. (Note that it is allowable compressive stresses, not compressive failure, that is the limiting condition.)

Examining Fig. 6 demonstrates the benefits of the higher strength concrete.
A girder spacing of 5 ft (1.5 m) and a concrete stress of 6000 psi (41 MPa) allows a simply supported beam to span 130 ft (40 m). Increasing the concrete strength to 7500 psi (52 MPa) allows the same girder to span 144 ft (44 m) without changing the girder spacing. This is a 10 percent increase in the capacity of the girder.

Conversely, looking at a 135 ft (41 m) span, the girder spacing can be increased from 4.5 ft (1.4 m) at 6000 psi (41 MPa) concrete strength to nearly 6 ft (1.8 m) at 7500 psi (52 MPa) concrete strength. This is a 33 percent increase in girder spacing. In certain circumstances, this increase may be sufficient to eliminate a line of girders from a bridge section, thereby making prestressed concrete bridges more cost-effective for the bridge designer.

### STRATEGIES FOR USING HIGH STRENGTH CONCRETE

The preceding section demonstrated that significant performance improvements could be accomplished using high strength concrete. However, the performance increases are not “free.” The additional capacity in some cases comes at the expense of additional prestress. The additional prestress increases the initial concrete strength requirement at transfer and begins a cycle of increased demand for more concrete strength. Therefore, the following strategies are presented in order to utilize the higher concrete strength that is currently available in most parts of North America:

1. Raise the girder strength requirements if a line of girders in the bridge may be eliminated. Examination of Fig. 6 suggests that as the space between the girders increases, a given bridge may be constructed with fewer girders. This would make the concrete bridge far more competitive with steel alternatives.

2. Examine the historical strength gain to determine if a higher final design strength is available. If a significant strength gain is present, high strength concrete may be “free.”

3. Increase the cure time before the prestress transfer to two days in order to obtain a higher initial strength compared to the design strength requirements. This is most economically accomplished if alternate stressing lines may be used.

4. If cementitious fly ash or Type I cement is used in the mix, consider using a 56-day strength to allow the cementitious material to react more completely.

5. Use a combination of pre- and post-tensioned concrete. Tee-, I- and bulb-tee sections are partially prestressed to allow handling in the plant. The remaining prestress is applied as post-tensioning and is applied to the partially cured concrete. The post-tensioning will have lower losses since it is applied to higher strength concrete. Addition of an in-plant cast-in-place deck (see Fig. 7) allows a portion of the field construction to be transferred to the plant and allows the supplemental dead load to be carried by the plant post-tensioning applied to the larger composite section.

6. Use continuous construction. Continuity reduces the midspan moment demand and allows the negative moment to be carried by reinforcement in the bridge deck.

7. When evaluating the condition of older prestressed concrete bridges, consider the time-dependent strength...
not just a laboratory phenomenon. The excess of the specified strength. Test data indicate that this strength is available in the completed structure and is display an actual concrete strength in load capacity than the original design. The increased strength may allow the bridge to continue to function at its full capacity or for a higher load capacity than the original design.

CONCLUSIONS

Many prestressed concrete girders display an actual concrete strength in excess of the specified strength. Test data indicate that this strength is available in the completed structure and is not just a laboratory phenomenon. The additional strength is available to the designer and has economic benefits in the form of longer spans and greater girder spacing. The analyses in this paper and past performance of prestressed concrete girders suggest that high strength concrete and bulb-tee girders are very cost-effective bridge members.

New concrete mixes with high cement factors, low water-cement ratios and containing microsilica should provide concrete with good durability characteristics. Therefore, the continued high performance of precast, prestressed concrete girders is likely. Strategies are provided to both use higher strength concrete mixes and to capitalize on residual strength of existing concrete mixes.

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