Analysis and Structural Benefits of High Performance Concrete for Pretensioned Bridge Girders

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The maximum lengths for simple-span pretensioned composite girders were investigated analytically using concrete strengths varying from 6 to 15 ksi (41 to 103 MPa), and prestressing strands of 0.5 in., 0.5 in. modified, and 0.6 in. (12.7, 12.7 and 15.2 mm) diameter. Both AASHTO and NU (developed by the University of Nebraska) sections were investigated. Girder spacings were varied from 5 to 11 ft (1.5 to 3.4 m). Use of the 0.6 in. (15.2 mm) diameter strand was more efficient and provided maximum spans up to 40 percent greater than 0.5 in. (12.7 mm) diameter strand. The strength of the deck did not significantly affect the maximum span, and the strength of the girders at release affected maximum span length more at lower design strengths than at strengths greater than 12 ksi (83 MPa). Overall, the use of 0.6 in. (15.2 mm) strand with concrete strengths up to 13 ksi (90 MPa) may be used to provide more efficient prestressed concrete bridge girders.

The purpose of this research was to further investigate the use of high performance/high strength concrete (HPC) for use in precast, pretensioned simple-span bridge girders. While substantial research on this subject has been done in the past, the Georgia Department of Transportation and the Federal Highway Administration wanted substantiation of that past research and specifically wanted to know the effects of using 0.6 in. (15.2 mm) diameter strand with HPC on span length and girder spacing. Other objectives were to study the effect of release strength, strand spacing, cover and composite deck strength on maximum spans and to verify that deflections of long-span pretensioned girders would not be excessive.

A principal need for longer span pretensioned girders is for replacement of center girders in existing highway bridges as illustrated in Fig. 1. As the number of lanes of traffic is increased beneath an existing bridge,
the piers adjacent to the abutments must be moved toward the abutments or removed altogether. The remaining girders must be lengthened. Adjacent structures like service stations restrict the alteration of the existing approach structures and embankments.

Therefore, the depth of the new, replacement bridge girders must be the same as the depth of the old girders. Maximizing the prestressing reinforcement in each girder and use of HPC was considered a possible solution for developing girders with longer spans which maintain the existing shallow depth of the girders and utilize girder spacings greater than about 6 ft (1.83 m).

The scope of the study was limited to analytically investigating AASHTO (American Association of State Highway and Transportation Officials) Types I through IV sections and comparable depth NU (Nebraska University) type girders as illustrated in Fig. 2. The reason for concentrating on AASHTO shapes was to limit the cost effect on precast producers as the bridge designers transition to higher strength concretes and the possible use of 0.6 in. (15.2 mm) diameter strand.

Note that NU girders were developed at the University of Nebraska with the objectives of increasing the efficiency of a girder and of providing a section more adaptable to continuous spans. Investigation of the NU shapes was done to determine if those sections would provide a better long-range solution for long-span girders.

**PARAMETRIC STUDY**

All bridge and girder designs in this research were based upon the 16th Edition of the AASHTO Bridge Design Specifications and used Georgia’s bridge design program PCPSMIR. Prestressing strand size and spacing were varied: strand diameters were 0.5 in., 0.5 in. modified, and 0.6 in. (respective areas of 0.153, 0.167, and 0.217 sq in.) (99, 108, and 140 mm²). Ultimate strengths were 270 and 300 ksi (1862 and 2068 MPa). Strand spacings were 1.75 and 2 in. (44 and 51 mm).

Girder concrete strengths were varied from 6 to 15 ksi (41 to 103 MPa). The composite concrete deck slab was 7 in. (178 mm) thick with a noncomposite 2 in. (51 mm) thick wearing surface; deck concrete strength was either 3.5 or 7 ksi (24 or 48 MPa). Girder spacings were varied from 5 to 11 ft (1.5 to 3.4 m).

The bottom strand cover was 1.5 and 2.5 in. (38 and 64 mm); Georgia often uses the larger cover to protect reinforcement from accidental impact. Bridge loading was HS 20-44 (truck, lane, and military) plus wearing surface and self weight.

The two top prestressing strands in each girder were stressed to only

![Fig. 2. Cross sections of AASHTO Types I through IV and NU 1100 and NU 1350 girders. Note: 1 in. = 25.4 mm.](image-url)
Fig. 3. Maximum span lengths for AASHTO Type I girders.

Fig. 4. Maximum span lengths for AASHTO Type II girders.

Fig. 5a. Maximum span lengths for AASHTO Type III and for NU1100 girders.

Fig. 5b. Maximum span lengths for AASHTO Type III and for NU1100 girders.
5 kips (22 kN) each while the bottom strands were fully stressed. The standard lump-sum prestressing losses were used. The allowable initial and final concrete tensile stresses were 7.5 $f'_c$ and 6 $f'_c$ psi (0.625 $f'_c$ and 0.5 $f'_c$ MPa) while the allowable and final compressive stresses were 0.6 $f_c$ and 0.6 $f_c$ psi for combined live and dead loads (0.4 $f_c$ for sustained loads).

**ANALYTICAL RESULTS AND DISCUSSION**

Figs. 3 through 6 present the composite girder maximum simple-span length versus the girder's concrete compressive strength for AASHTO Types I through IV shapes, respectively, for the four girder spacings and for strand diameters (D) of 0.5 in. modified and 0.6 in. (12.7 and 15.2 mm). The NU1100 shape is included in Fig. 5 for comparison with the Type III girder, and NU 1350 is included in Fig. 6.

For each analysis, the cover was 1.5 in. (38 mm), the strand spacing was 2 in. (51 mm), the deck $f'_c$ was 3.5 ksi (24 MPa), and the concrete compressive strength at release was 75 percent of $f'_c$. The curves are not smooth because strands were added two at a time and because span lengths changed abruptly as a new row of strands was added.

Concrete Strength and Number of Strands

Each figure illustrates that as the concrete strength was increased from 6 to 15 ksi (41 to 103 MPa), the maximum span length increased. This occurs because as the concrete strength was increased, more prestressing strands could be placed in the bottom flange without exceeding the maximum initial compressive stress or the final allowable tensile stress, the two controlling stresses for all bridges. Fig. 7 illustrates a typical relation of the number of strands versus maximum span length for a Type III girder using 0.6 in. (15.2 mm) diameter strand.

Similar plots occurred for all sections and strand sizes. As more strands were placed in the section, the length of the span increased but at a lesser rate until the addition of another strand did not increase the span length. The efficiency of the prestressing was reduced as more strands were added because the eccentricity between the centroid of strands and the girder section was reduced.

Each curve in Figs. 3 through 6 plateaued as the concrete strength was increased. The maximum span plateau was reached at a lower concrete strength for the 0.5 in. (12.7 mm) strand as compared with the 0.6 in.
(15.2 mm) strand. The concrete strength at this plateau level was called the maximum effective girder compressive strength.

This plateau level was defined as the strength where the maximum span was at least 95 percent of the length of the span found using 15 ksi (103 MPa) concrete and where the change in span length for each 1 ksi (7.7 MPa) change in strength was less than 2 percent of the span length found using 15 ksi (103 MPa) concrete. In general, this meant that the change in span length was less than 1 to 2 ft (0.3 to 0.6 m) for each 1 ksi (6.9 MPa) change in strength. Table 1 presents these maximum effective strengths.

### Strand Size, Strength, and Spacing

Figs. 3 through 6 show that use of 0.6 in. (15.2 mm) strand becomes more effective than 0.5 in. (12.7 mm) modified strand in increasing the maximum span length when the concrete strength becomes greater than about 9 to 11 ksi (62 to 76 MPa). When the 0.5 in. (12.7 mm) modified strand was used, the maximum span length became virtually constant for concrete strengths between 9 to 11 ksi (62 to 76 MPa). This "topping out" occurred because the maximum number of strands was placed in the bottom flange of the girder. Additional strands placed in the web did not significantly increase the span length.

With the 0.5 in. (12.7 mm) diameter modified strand, a further increase in concrete strength was not needed to resist a prestressing force which was nearly maximized; therefore, increasing the concrete strength did not significantly affect the span length. Increased concrete strength did increase the allowable tensile capacity and the elastic modulus which resulted in a slight increase in span. Similar results occurred when 0.5 in. (12.7 mm) diameter strand was used. Overall, the use of the 0.6 in. (15.2 mm) strand resulted in maximum span lengths between 5 and 10 percent greater than use of 0.5 in. (12.7 mm) modified strand.

The efficiency of the prestressing was represented by the prestressing index, defined as the product of the prestressing force and the eccentricity between the centroid of the strands and the girder cross section. Fig. 8 illustrates the prestressing index for an AASHTO Type III section versus the number of strands in the section. Three strands are given, 0.5 in. (12.7 mm) with $f_{pu}$ of 300 ksi (2068 MPa) and spacing of 1.75 in. (44 mm), 0.5 in. (12.7 mm) modified with $f_{pu}$ of 270 ksi (1862 MPa) and spacing of 1.75 in. (44 mm), and 0.6 in. (15.2 mm) with $f_{pu}$ of 270 ksi (1862 MPa) and spacing of 2.0 in. (51 mm).

The 0.6 in. (15.2 mm) strand had a greater prestressing index than either the smaller strand with higher strength or the 0.5 in. (12.7 mm) modified strand with closer spacing. Because the prestressing index remained greater with the 0.6 in. (15.2 mm) strand, the 0.6 in. (15.2 mm) strand at

<table>
<thead>
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<th>Spacing</th>
<th>0.6 in. diameter strands</th>
<th>0.5 in. diameter modified strands</th>
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<tr>
<td></td>
<td>11 ft (3.4 m)</td>
<td>9 ft (2.7 m)</td>
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<td>AASHTO</td>
<td></td>
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<tr>
<td>Type I</td>
<td>13 (89.6)</td>
<td>12 (82.7)</td>
</tr>
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<tr>
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<td>Type IV</td>
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<td>11 (75.8)</td>
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<tr>
<td>NU1350</td>
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<td>11 (75.8)</td>
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Table 1. Maximum effective girder compressive strength (ksi, MPa).

Note: 1 in. = 25.4 mm; 1 ft = 0.3048; 1 ksi = 6.895 MPa.
2 in. (51 mm) spacing continued to result in longer maximum length spans than provided by high strength 0.5 in. strand or by 0.5 in. (12.7 mm) modified strand at 1.75 in. (44 mm) strand spacing.

**AASHTO Versus NU Sections**

Figs. 5 and 6 illustrate that the NU sections with about the same depth as an AASHTO section could develop span lengths greater than the AASHTO sections. The NU section permitted a greater number of strands to be placed with a greater eccentricity than did the AASHTO section. Therefore, the flexural resistance of the NU section was greater. Note that the NU span length reached its maximum at lower concrete strengths than the AASHTO sections because in the NU section a lower concrete strength was needed to resist the total precompression when the maximum number of strands was placed in the bottom flange. The NU section was not optimized to take advantage of concrete strengths up to 15 ksi (103 MPa).

**Cover**

When the bottom cover was increased from 1.5 to 2.5 in. (38 to 64 mm), the prestressing index was reduced, and the maximum span lengths were reduced significantly for AASHTO Types I and IV and all NU sections. Less prestressing strands could be placed in the bottom flange, so the flexural resistance was reduced.

**Release Strength**

Spans were examined for the 0.6 in. (15.2 mm) strand when the concrete strength at release of the prestress was reduced from 75 percent of the design compressive strength to 65 percent. It was hypothesized that for the higher strength concrete and for a one-day turnaround at a precast plant, the release strength may have to be lower than normal. For all cases, the maximum span lengths were reduced when the release strength was reduced.

Fig. 9 is an example which compares maximum span lengths for a Type IV girder, concrete strengths, and release strengths equal to 65 and 75 percent of the design strength. The maximum span lengths were reduced more for lower strength concretes than for higher strengths.

For girders at 9 ft (2.7 m) spacing, reducing the release strength from 75 to 65 percent for a 6 ksi (41 MPa) girder resulted in a 9 percent decrease in maximum span, about a 9 ft (2.7 m) span difference; while for a 15 ksi (103 MPa) girder, the reduction resulted in less than a 2 percent decrease in span, about a 2 ft (0.6 m) difference. Therefore, as the concrete strength increased, the effect of a reduced release strength decreased. Other materials research has indicated that the high temperatures which occur in curing HPC with high cement contents may result in release strengths greater than 75 percent of the design strength.

**Girder Spacing**

Figs. 3 through 6 show that higher strength concretes resulted in a wider girder spacing for a given span length; the intersection of a horizontal line across each figure with the span versus $f'_c$ curve gives the increase in girder spacing for an increase in concrete strength. Fig. 10 shows more clearly this relation for a bridge 130 ft (39.6 m) long and 60 ft (18.3 m) wide made with Type IV girders and using 0.6 in. (15.2 mm) strand.

As the concrete strength was increased from 6 to 15 ksi (41 to 103 MPa), the girder spacing was increased from 4.0 to 7.5 ft (1.22 to 2.29 m) and the number of girders was reduced from sixteen to nine. Cook conducted an economic analysis which demonstrated the potential cost savings from widening the girder spacings.
Deflections

The midspan deflection of the example maximum span girders under the HS 20-44 plus impact loading was compared to a maximum allowable deflection of \( L/800 \) (\( L \) = span length). In nearly all cases, the deflections were less than the allowable. For AASHTO sections with girder spacings of 5 ft (1.5 m), using 0.6 in. (15.2 mm) strand and with \( f'_c \) greater than 10 ksi (69 MPa), the deflection limit was exceeded. For NU sections with girder spacings of 5 ft (1.5 m), the deflection limits were sometimes exceeded. In general, deflections did not control the design for most HPC girders.

Deck Strength

Increasing the strength of the deck from 3.5 to 7.0 ksi (24 to 48 MPa) increased the maximum span length for Type IV girders by a maximum of 1 percent. The higher elastic modulus of the deck resulted in the centroid of the composite section being raised which resulted in an increased section modulus.

In a few cases, this caused the critical design condition to change from final tension stress in the bottom of the girder to initial compression at release. In general, the strength of the concrete in the deck was not a significant factor in determining maximum span lengths. Use of HPC for decks may result in more durable bridges.\(^{13}\)

CONCLUSIONS

Based on the results of this investigation, the following conclusions can be made:

1. High performance/high strength concrete may be used to provide more efficient bridge girders which effectively utilize larger diameter prestressing strands.

2. For the AASHTO sections, the maximum spans were increased between 20 and 45 percent when the concrete strength was increased from 6 to 14 ksi (41 to 96 MPa) and when strand size was increased from 0.5 to 0.6 in. (12.7 to 15.2 mm).

3. Use of 0.6 in. (15.2 mm) strand was most effective when girder strengths exceeded 8 ksi (55 MPa).
4. The maximum effective girder concrete strengths given in Table 1 indicated that for Types I through IV girders using 0.6 in. (15.2 mm) strand, concrete strengths greater than 12 to 13 ksi (83 to 90 MPa) did not significantly increase the maximum span lengths.

5. Decreasing the concrete strength at release from 75 to 65 percent of $f'_{c}$, as an example, resulted in less than a 5 percent decrease in maximum span length when concrete strengths were 10 ksi (69 MPa) and greater for Type IV sections and for girder spacings of 7 ft (2.13 m) and greater.

6. The allowable live load deflection of $L/800$ was not exceeded except for girders spaced at 5 ft (1.52 m).

7. The strength of the composite concrete deck had little influence on the maximum span of high strength girders.

**RECOMMENDATIONS**

1. The authors recommend that Table 1 be used to evaluate the maximum concrete strengths which are to be used for the cross sections listed.

2. The authors further recommend that future analytical research should investigate new sections which would optimize the use of 0.6 in. (15.2 mm) and larger strand with concrete strengths greater than 10 ksi (69 MPa).

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**REFERENCES**

13. Lai, James, Kahn, L., Travis, D., Champney, Mark, Prada, J., Shams, M., and Saber, A., "Use of High-Strength/High-Performance Concrete for Precast Prestressed Bridge Girders in Georgia: Materials Study," Georgia Department of Transportation Project No. 9510, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA, April 1999, 255 pp.