Lightweight Concrete Reduces Weight and Increases Span Length of Pretensioned Concrete Bridge Girders

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The maximum lengths for simple-span pretensioned concrete composite girders using high strength lightweight concrete (HSLWC) were investigated analytically using concrete strengths of 8, 10, and 12 ksi (55, 69, and 83 MPa) and prestressing strands of 0.6 in. (15.2 mm) diameter. The use of HSLWC produced spans up to 4 percent longer than the same section made with high strength normal weight concrete (HSNWC). Based on the AASHTO I-girder and AASHTO-PCI bulb-tee sections examined in this study, the reduced girder weight eliminated the need for special transportation "superload" permits. Modified AASHTO-PCI bulb-tees with one extra row of strands in the bottom flange extended the girders' maximum span length by at least 10 ft (3.1 m). In all cases, the use of lightweight concrete caused greater girder deflections, but all of these values were within the AASHTO specified limit of L/800. Overall, the advantages of lightweight concrete with compressive strengths up to 12 ksi (83 MPa) include lower girder weight, relief from special permitting requirements, and longer span lengths.

A recent study by Kahn and Saber[1] on pretensioned concrete bridge girders concluded that the use of high strength, high performance concrete could provide simple span girder lengths up to 40 percent greater than when normal strength [6 ksi (41 MPa)] concrete is used. That study also cited wider girder spacing and greater durability as tangible advantages. These potential advantages, however, may be lost if these longer girders are too heavy to be transported.

In Georgia, when the gross vehicle weight (GVW) – equal to the cargo weight plus the tractor-trailer) – exceeds 150 kips (68,200 kg), a "superload permit" is required. This special permit requires the hauler to adhere to additional restrictions that may include stopping before every bridge, proceeding over the bridge at a speed less than 5 miles per hour (8 km/hr), and having escorts lead and follow the truck along the route.

For Georgia's superload permit cases, the Georgia Department of

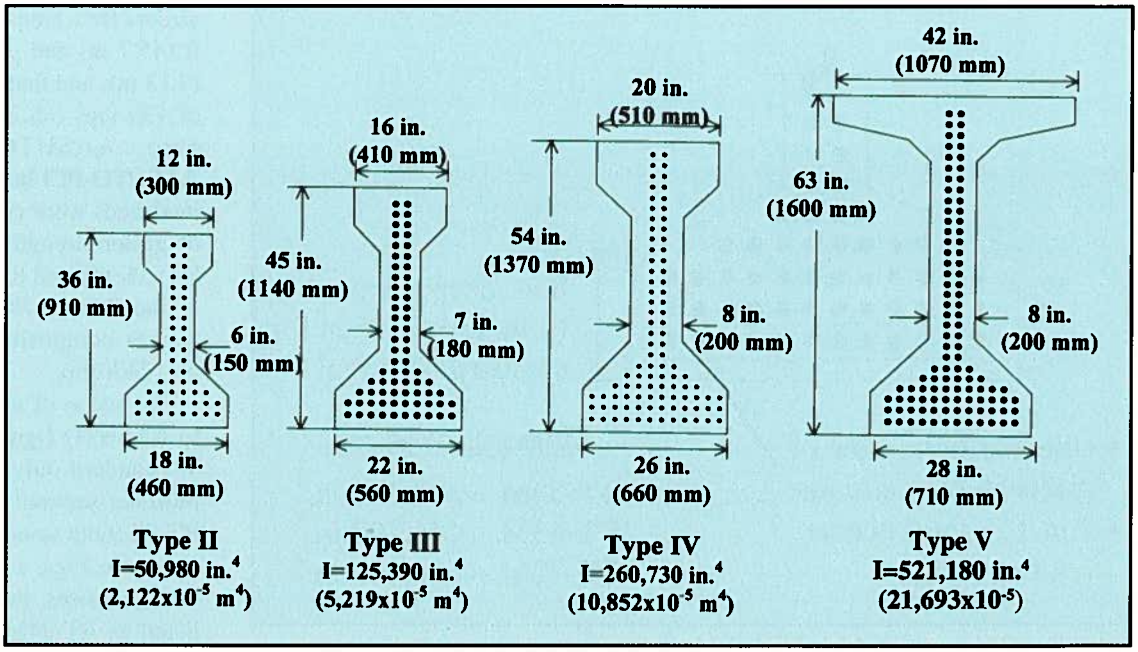

Fig. 1. Cross sections of standard AASHTO I-girders.

Transportation (DOT) must carefully review the route and evaluate the load capacity of each bridge that the truck would cross. In some cases, there may not be an acceptable route for the given GVW. Aside from the restrictions imposed on the transport of the shipment, the required slow rate of speed can significantly disrupt traffic flow over a bridge crossing.

Investigative work on this issue has been minimal, and most of the research that has been published has involved concrete strengths below 8 ksi (55 MPa). The Georgia DOT would like to avoid the issuing of superload permits, yet at the same time it would like to take advantage of the benefits of HPC in pretensioned girders. To this end, a research goal has been set to achieve a 150 ft (45.7 m) long pretensioned girder with a weight that can yield a GVW of less than 150 kips (68,200 kg) – that is, to create a 150 ft (45.7 m) girder that would not require a superload permit.

A further goal is too have a girder spacing of at least 7 ft (2.13 m), which could potentially reduce the total number of girders on a bridge span. An all-lightweight or sand-lightweight concrete would be needed to achieve these goals. Previous research[1] has shown that AASHTO Type IV girders using 0.6 in. (15.2 mm) diameter prestressing strands, a concrete strength of 15,000 psi (103 MPa), and a girder spacing of 5 ft (1.52 m) could achieve the span, but it would result in a GVW of about 185 kips (84,100 kg).

Therefore, the purpose of this research was to determine analytically whether high strength lightweight concrete (HSLWC) could be used to fabricate pretensioned concrete bridge

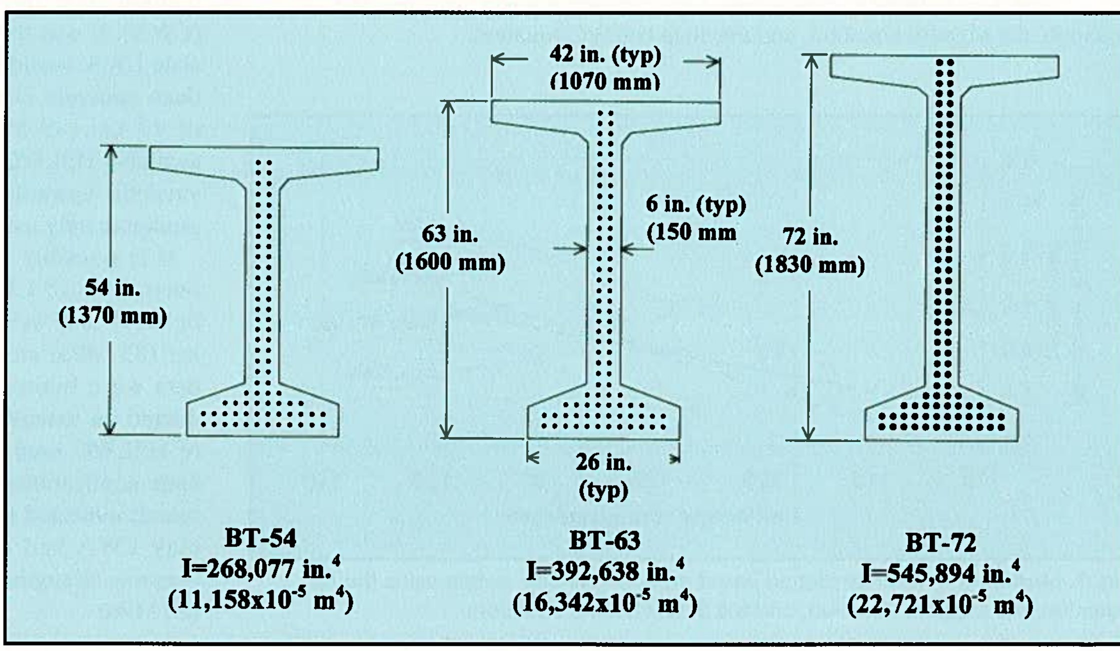

Fig. 2. Cross sections of standard AASHTO-PCI bulb tees.

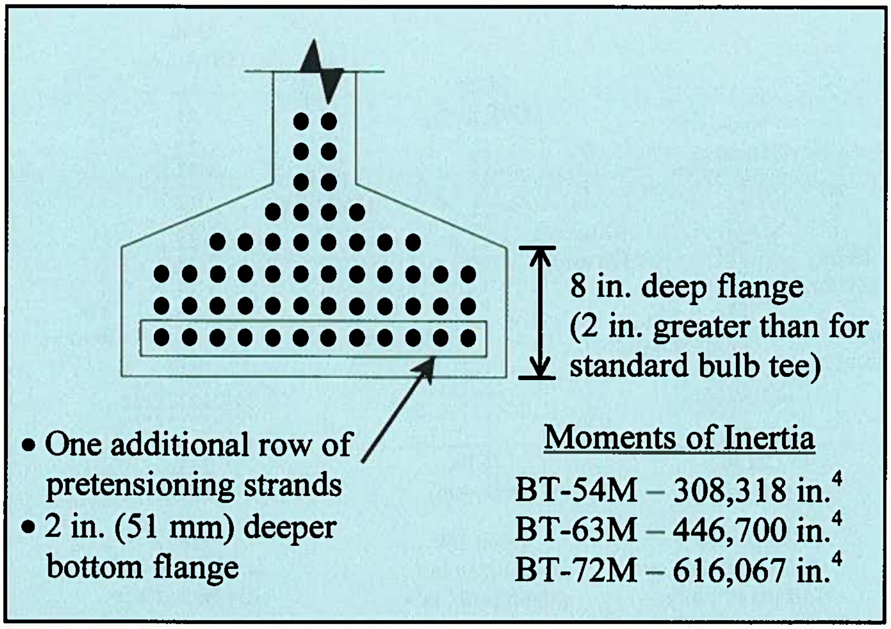

Fig. 3. Detail of a modified AASHTO-PCI bulb tee, illustrating the 2 in. (51 mm) deeper bottom flange and the additional row of prestressing strands.

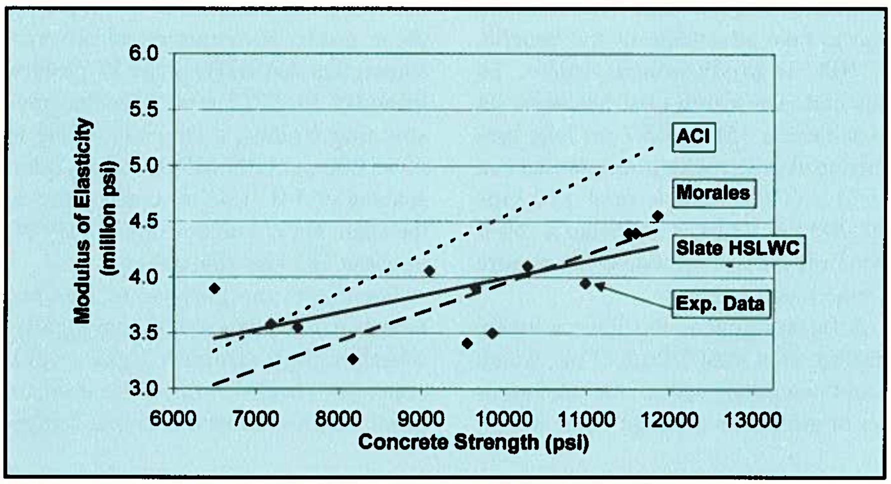

Fig. 4. Modulus of elasticity plotted based on concrete strength using the ACI equation, the Morales equation, and the Slate HSLWC equation.

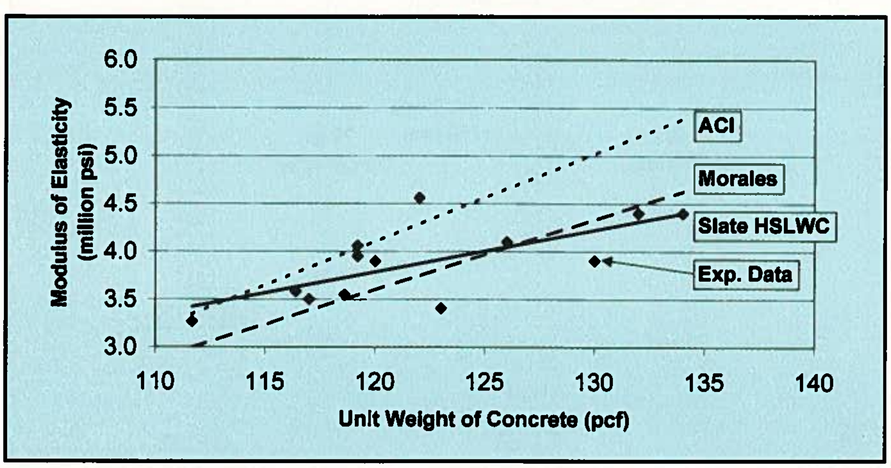

Fig. 5. Modulus of elasticity plotted based on concrete unit weight using the ACI equation, the Morales equation, and the Slate HSLWC equation.

girders for a simple span length of 150 ft (45.7 m) and girder spacing of 7 ft (2.13 m), and that would yield a GVW of 150 kips (68,200 kg) or less. Standard AASHTO I-girders and AASHTO-PCI bulb tees (standard and modified) were considered. The range of girder strengths was 8, 10, and 12 ksi (55, 69, and 83 MPa). The strength of the 7 in. (178 mm) thick, normal weight composite decking was 3500 psi (24 MPa).

The scope of this study was limited to AASHTO I-girder Types II through V (standard only) and AASHTO-PCI bulb-tee sections BT-54, BT-63, and BT-72 (both standard and modified) as shown in Figs. 1, 2, and 3. For reference purposes, modified bulb tees are listed as BT-54M, BT-63M, and BT-72M. The modified bulb-tee sections are identical to the standard bulb-tee sections, except that the bottom flange has a 2 in. (51 mm) greater depth to accommodate one additional row of 12 strands (see Fig. 3).

The inclusion of the modified bulb-tee sections was precipitated by discussions with a number of Georgia precast concrete producers, who already had experience in their fabrication and recommended their use. Prestressing strands were 0.6 in. (15 mm) diameter, 270 ksi (1862 MPa) low relaxation strands spaced at 2 in. (51 mm) on center.

The HSLWC in this study assumed the use of regionally available expanded slate lightweight aggregate (LWA). It was thought that the use of slate LWA would be necessary to produce concrete compressive strengths of 12 ksi (83 MPa). Furthermore, available HSLWC test data indicating strengths upwards of 12 ksi (83 MPa) predominantly used slate LWA.[2-5]

It is possible that HSLWC using other types of LWA (expanded shale or clay) may achieve upwards of 12 ksi (83 MPa) strength, but available data were limited. The authors conducted an extensive literature search of HSLWC used for prestressed concrete applications.[6] The results of that search indicated that neither shale nor clay LWA had ever been used for concrete strengths approaching 12 ksi (83 MPa).

PARAMETRIC STUDY

All girder designs in this study were based on the 16th Edition of the AASHTO Standard Specifications for Highway Bridges[7] and used the Georgia DOT bridge design computer program with modification by the authors to enable the use of HSLWC. Several steps were necessary prior to using the program to design HSLWC bridge girders. It was assumed, for example, that prestress losses would be the same as for normal strength concrete.

Other ongoing research by the authors indicates that for normal weight and lightweight HPC, the creep and shrinkage losses are less than for normal strength concretes. This finding is significant because deflection was a major concern in the designs for this study.

Determination of Modulus of Elasticity

A critical requirement for the design was to accurately predict the modulus of elasticity for HSLWC made using slate LWA at both the time of release (E_{ci}) and at 28 days (E_c). Accurate values of the modulus of elasticity are necessary to compute prestress losses and girder deflections accurately. Experimental data from 13 mixes using slate LWA were used to determine the appropriate modulus values.[2-5, 8-13]

The ACI[14] and AASHTO[7] equation for modulus of elasticity is:

$$E = w_c^{1.5} 33\sqrt{f_c'} \qquad (1)$$

for concrete having a unit weight of 90 to 155 lb per cu ft (1442 to 2483 kg/m^3). When used with HSLWC, Eq. (1) was found to overpredict the modulus of elasticity values. Morales[15] proposed the following equation:

$$E = (40{,}000\sqrt{f_c'} + 1{,}000{,}000)(w_c/145)^{1.5} \qquad (2)$$

to more closely predict modulus values for lightweight concrete (LWC); however, when used with HSLWC made using slate LWA, the calculated values were lower than the experimental results for strengths below 10 ksi (69 MPa) and higher for strengths over 10 ksi (69 MPa).

To more accurately predict the modulus values, the authors developed Eq. (3), similar in form to the Morales equation, but based on a "best fit" analysis of the experimental data from the 13 slate mixes:

$$E = (33{,}000\sqrt{f_c'} + 4{,}000{,}000)(w_c/242)^{0.9} \qquad (3)$$

Figs. 4 and 5 plot the experimental data and the three equations for modulus of elasticity versus concrete strength and unit weight, respectively. Eq. (3) was used for all further analyses.

Girder Design

The Georgia DOT computer program was used to find the maximum span length for each girder type in the study. The variable parameters were concrete strength and concrete unit weight. Other critical variables were kept constant as shown in Table 1.

The concrete strengths used were 8, 10, and 12 ksi (55, 69 and 83 MPa). When designing with high strength normal weight concrete (HSNWC), it was assumed that the concrete weight was 150 lb per cu ft (2403 kg/m^3), and the modulus of elasticity was computed with Eq. (1) based on previous HSNWC research in Georgia.[16] When designing with HSLWC, Eq. (3) was used with different unit weights.

During the study of the 13 slate mixes, it was found that variations in unit weight existed for a given strength.[2-5, 8-13] For this reason, a range of unit weights was established for each HSLWC strength based on observed upper and lower limits. Table 2 shows the range of unit weights used in the study.

The variation in unit weights caused variations in modulus values as well as girder weights. Each bridge and girder design accounted for the differing moduli at release and final conditions. Note that of the strengths shown in Table 2, only the low unit weight mix with 8000 psi (55 MPa) strength qualifies as structural lightweight concrete according to the ACI 318 definition.[14]

Table 1. Critical girder design variables that remained constant.

Allowable top fiber tension stress at release	$3\sqrt{f_c'}$
Allowable final bottom fiber tension stress	$6\sqrt{f_c'}$
Release strength as percent of 28-day compressive strength	75 percent
Composite deck thickness	7 in. (17.8 mm)
Composite deck strength	3500 psi (24.2 MPa)
Pretensioning strand diameter	0.6 in. (15.2 mm)
Pretensioning strand spacing	2 in. (50.8 mm)
Pretensioning strand ultimate strength, f_{pu}	270 ksi (1862 MPa)
Type of pretensioning strand	Low relaxation
Percent of strand ultimate strength allowed at time of pretensioning	75 percent
Girder spacing	7 ft (2.13 m)

Table 2. Slate high strength lightweight concrete (HSLWC) unit weight values.

Concrete strength, f_c' psi (MPa)	Unit weight		
	Low lb per cu ft (kg/m^3)	Average lb per cu ft (kg/m^3)	High lb per cu ft (kg/m^3)
8,000 (55)	113 (1810)	119 (1906)	126 (2019)
10,000 (69)	117 (1874)	124 (1986)	131 (2099)
12,000 (83)	122 (1954)	128 (2051)	135 (2163)

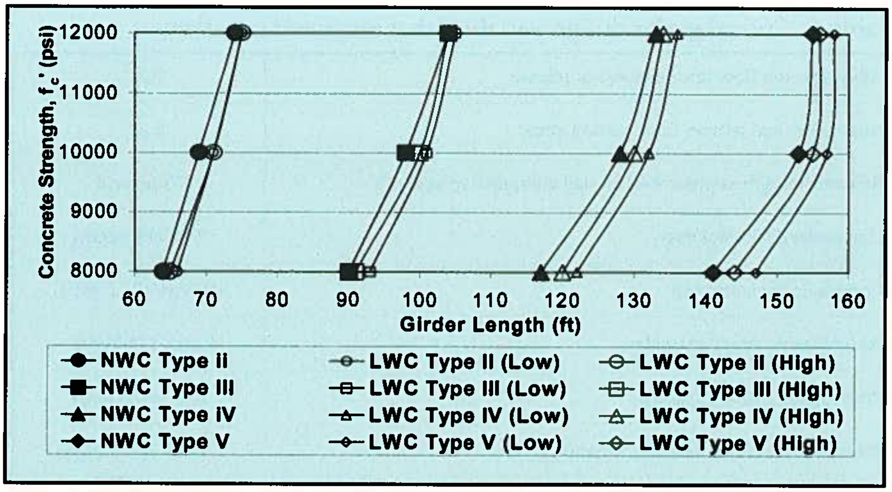

Fig. 6. Concrete strength versus maximum girder length for standard AASHTO I-girders using normal weight concrete (NWC) and low- to high-density lightweight concrete (LWC).

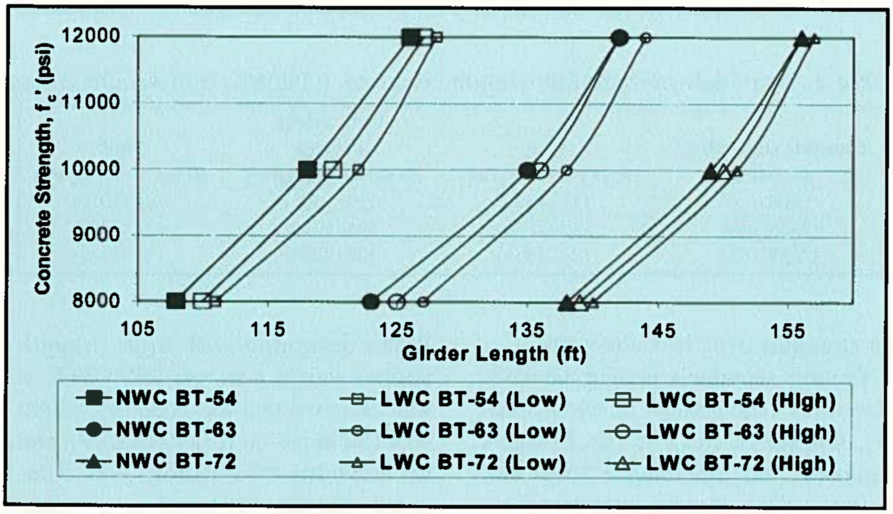

Fig. 7. Concrete strength versus maximum girder length for standard AASHTO-PCI bulb-tee girders using normal weight concrete (NWC) and low- to high-density lightweight concrete (LWC).

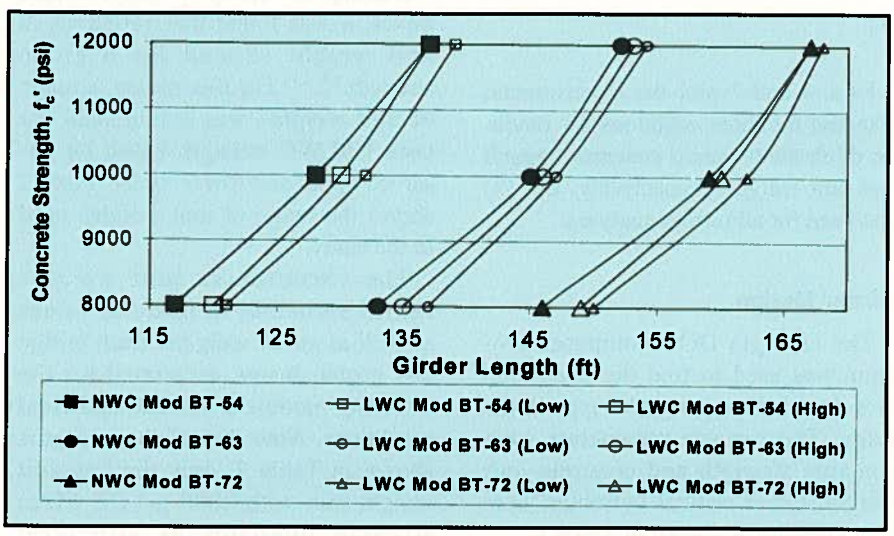

Fig. 8. Concrete strength versus maximum girder length for modified AASHTO-PCI bulb-tee girders using normal weight concrete (NWC) and low- to high-density lightweight concrete (LWC).

RESULTS AND DISCUSSION

Figs. 6 through 8 present the composite girder maximum simple-span length versus the concrete compressive strength for standard AASHTO Type II through V sections, AASHTO-PCI bulb tee BT-54, BT-63, and BT-72 sections, and modified BT-54M, BT-63M, and BT-72M sections. Both normal weight (NWC) and lightweight (LWC) concretes are shown. "Low" and "high" values for LWC provide the range of results of concrete unit weight for each concrete strength.

Effect of Concrete Strength and Unit Weight on Girder Span Length

AASHTO Type II through V sections – Span lengths of girders using 8 ksi (55 MPa) HSLWC could be extended by up to about 4 percent [7 ft (2.13 m) for 140 ft (42.7 m) spans] as shown in Fig. 6. The most significant length increases resulted from the use of the lightest concrete unit weight. The increase in length for Type II and III sections was less than for Type IV and V sections, implying that the use of HSLWC provided the most significant benefit for girders with lengths over approximately 105 ft (32 m).

Fig. 6 also indicates that, for Type V sections, there is little benefit to using concrete with a strength greater than 10 ksi (69 MPa). Length increase is proportional to the amount of reinforcement that can be located in the bottom flange.[1] At maximum lengths for Type V sections, the total strands possible were used in the bottom flange with 10 ksi (69 MPa) concrete.

The HSLWC girder maximum live load deflections resulting from HS 20-44 loading increased 15 to 22 percent on average over HSNWC girders. The lighter concrete unit weights had the greatest deflections; however, deflections were, at most, 85 percent of the AASHTO[7] maximum allowable $L/800$ requirement, compared to 57 percent for HSNWC. Furthermore, the natural period of vibration increased by less than 19 percent when HSLWC was used compared to HSNWC for the 10 ksi (69 MPa) Type IV girders.

Bulb-tee sections – Fig. 7 shows

trends for the AASHTO-PCI bulb-tee sections similar to those for the AASHTO I-girder sections. HSLWC with a strength of 8 ksi (55 MPa) provided a length increase up to about 3 percent [3 ft (0.91 m) for 110 ft (33.5 m) girders]. The major difference from I-girder sections was that bulb-tee sections exhibited a consistent benefit from using concrete strengths up to 12 ksi (83 MPa). Based on the improved efficiency of the bulb-tee sections, there was not an observed plateau within the strength range investigated for the constant 7 ft (2.13 m) girder spacing. Live load deflections of bulb-tee sections averaged, at most, 70 percent of the AASHTO maximum allowable of *L*/800. The natural period of vibration for a 135 ft (41.4 m) long, 63 in. (1.60 m) deep HSLWC bulb-tee girder increased by less than 17 percent compared to that of a HSNWC girder.

Modified bulb-tee sections – Fig. 8 shows that the modified bulb-tee sections behaved in a similar manner to the standard bulb-tee sections. The largest percent increase in length was gained with 8 ksi (55 MPa) HSLWC at about 3 percent [4 ft (1.22 m) for 146 ft (44.5 m) girders]. The addition of a row of pretensioning strands in the bottom flange allowed an increase in length of about 10 ft (3 m) over that of standard bulb-tee sections at all strengths of concrete and for all sections. Live load deflections were, again, well within the maximum allowable *L*/800 limit, averaging 64 percent of the allowable.

The additional strands and 2 in. (50.8 mm) increase in depth would allow designers to use a shallower section to reach lengths previously achievable only with the next larger size bulb-tee section. For the 156 ft (47.5 m) long, 65 in. (1.65 m) deep HSLWC modified bulb-tee girders, the natural period of vibration increased by less than 15 percent compared to that of a HSNWC girder.

Weight Reduction Using HSLWC

The most significant advantage gained through the use of HSLWC was girder weight reduction. In this study, it was possible to lower the GVW below the 150 kip (68,200 kg) limit for the target 150 ft (45.7 m) girder. Table 3 shows information related to Georgia permitting requirements for overweight cargo as provided by the Georgia DOT.

Table 3. Permitting requirements for overweight cargo in Georgia.

Load category	Gross vehicle weight range (kips)	Estimated vehicle weight (kips)	Maximum girder weight (kips)	Permit type	No. of axles required
1	0 < GVW ≤ 80	40	40	None	NA
2	80 < GVW ≤ 125	40	85	Regular	6
3	125 < GVW ≤ 150	45	105	Regular	7
4	150 < GVW ≤ 160	50	110	Superload	8
5	160 < GVW ≤ 175	52.5	122.5	Superload	9
6	175 < GVW ≤ 180	55	125	Superload	10
7	180 < GVW	55+	125+	Superload	>10

Note: 1 kip = 454.5 kg.

It is difficult to provide exact data on weights and permitting requirements due to variations in tractor-trailer rig configurations and capabilities, but Table 3 does provide some average values that would be expected. Fig. 9 illustrates the GVW for each of the maximum length girders for the ten sections studied using HSNWC at 8 ksi (55 MPa) strength.

Similar graphs were developed for 10 and 12 ksi (69 and 88.2 MPa) HSNWC and HSLWC. The collective results of GVW, maximum girder length, and section type are shown in Fig. 10. Because AASHTO Type II and III girders showed little benefit from HSLWC, they were not included in the graph.

Within Fig. 10, there are three data points for each section listed. The three points correspond with the three strengths of concrete. In each case, the leftmost of the three points represents 8 ksi (55 MPa), the center point represents 10 ksi (69 MPa), and the rightmost point represents 12 ksi (88.2 MPa). The average unit weight given in Table 2 was used for the lightweight (L) sections, whereas a 150 lb per cu ft (2403 kg^3) unit weight was used for each of the normal weight (N) sections.

The most important finding was that through the use of HSLWC, it was possible to reach the target span of 150 ft (45.7 m) without exceeding the 150 kip (68,200 kg) GVW limit. There were three HSLWC girders that satisfied the requirement: BT-63M (12 ksi), BT-72 (10 ksi), and BT-72M (8 ksi). Further economic analysis and a review of site constraints would be required to select the best possible alternative. For spans between 125 and 135 ft (38.1 and 41.1 m), Type IV lightweight girders satisfied the 150 kip (68,200

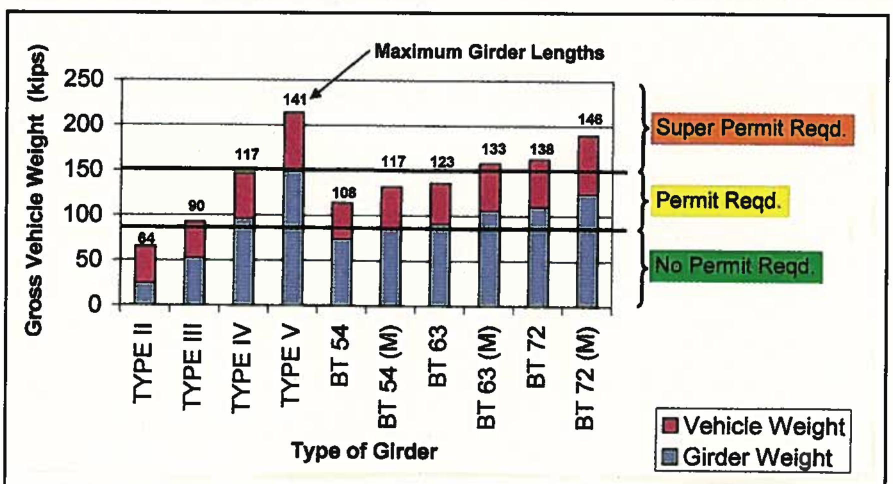

Fig. 9. Gross vehicle weight (GVW) by girder type, the maximum length labeled, based on 8 ksi (55 MPa) strength normal weight concrete (NWC).

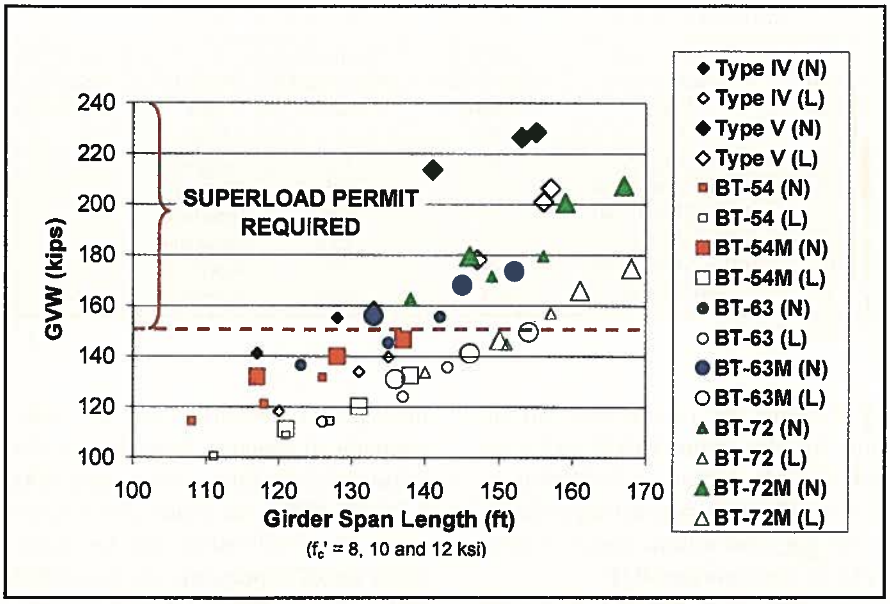

Fig. 10. Gross vehicle weight (GVW) versus maximum girder length.

kg) GVW limit, whereas Type IV normal weight girders did not.

Based on the parameters and results found from this study, both standard and modified AASHTO-PCI bulb-tee sections are more efficient than AASHTO Type IV and V sections of similar height.

When the AASHTO Type IV and V sections are removed from the chart in Fig. 10, there are noticeable trend lines for the remaining normal weight and lightweight bulb-tee girders. The trend lines demonstrate the benefit in terms of GVW that result from the use of HSLWC. The trend lines converge for girder lengths less than 105 ft (32 m) and diverge as girder lengths increase from that point.

CONCLUSIONS

Based on the results of this analytical investigation, the following conclusions are drawn:

1. The use of high strength, lightweight aggregate concrete (HSLWC) has the potential to increase the length of simple span AASHTO I-girders by up to 4 percent and the length of AASHTO-PCI bulb-tee girders by up to 3 percent.

2. For spans between 125 and 155 ft (38.1 and 47.2 m), the use of lightweight concrete can reduce the gross vehicle weight to less than 150 kips (68,200 kg) so that a superload permit would not be required to transport long span girders. For the same span range, a superload permit would be needed when normal weight concrete is used.

3. The use of HSLWC provides no appreciable benefit to AASHTO Type II and III sections.

4. The modified bulb-tee section can be extended by 10 ft (3.1 m) over a standard bulb-tee using either HSLWC or HSNWC at strengths of 8, 10, or 12 ksi (55, 69, or 83 MPa).

5. For girders over 105 ft (32 m) in length, AASHTO-PCI bulb-tee sections, both standard and modified, provide longer spans at less weight than standard AASHTO I-girder sections.

RECOMMENDATIONS

The authors recommend the following:

1. HSLWC should be evaluated experimentally to determine if its perceived lengths can be developed in actual bridge construction.

2. All types of lightweight aggregate should be investigated for their potential use in high strength concrete.

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