Standard Designs for Hardwood Glued-Laminated Highway Bridges

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Abstract

Standard plans and specifications for hardwood glued laminated highway bridges have been developed and published. The plans are based on recent research to identify laminating processes, resin systems, structural properties of efficient beam cross sections, and preservative treatment processes. The results of these efforts are summarized. The standard plans are based upon nationally recognized allowable strength design methodologies and are for HS-25 or IML-80 loads. The standard plans, which are for northern red oak, red maple, and yellow poplar bridges, include details for design and construction of highway bridge superstructures and substructures The standard plans and specifications are being revised. The status of efforts to incorporate design efficiencies suggested by ongoing research and to convert the standard plans to a load resistance factor design basis are also presented.

Keywords: Bridges, timber, wood, highway, hardwood, glued-laminated, glulam, standard plans, specifications, allowable strength design(ASD), load resistance factor design (LRFD),

Introduction and Background

Overview

Researchers at The Pennsylvania State University (PSU), in cooperation with the Pennsylvania Department of Transportation (PennDOT) and the engineering firm of Gwin, Dobson and Foreman (GDF) have developed standard plans for hardwood glued-laminated (glulam) highway bridges. The overall goals of the project were to develop or identify the technology for more efficient use of temperate hardwoods in glulam bridge construction and to produce a user friendly set of standard plans for use by consulting engineers, highway and transportation department engineers, and local governmental engineers. Also the standard designs were to specify bridges suitable for normal highway use with no special load restrictions.

Before bridge standards could be developed, it was necessary to develop the technology base for manufacture and use of efficient hardwood glulam beams and deck panels. This required identification of candidate species for the bridges, identification of the resin systems to be used, identification and refinement of the preservative treatment for the bridge members, and development or verification of the structural properties of efficient hardwood glulam beams. Only with these technologies in place could the first generation standard plans be developed.

A second generation of the standard plans is currentlv under development. These plans ‘will include the results of more recent research to improve the efficiencies of the girder and deck panel designs. The second generation plans will also convert the designs from an allowable strength design (ASD) basis to a load resistance factor design (LRFD) basis, as well as convert the standards to metric units.

The goals of this paper are to summarize the technological developments leading to completion of the standard plans and to outline the features of the ASD-based Standard Plans for Hardwood Glued Laminated Timber Highway Bridges (BLC-560). A further goal is to summarize the progress to date on the conversion to LRFD-based standard plans and to present preliminary comparisons of the outcomes of ASD- versus LRFD-based outcomes for hardwood glulam bridge girders.

Accomplishments To-Date

Species Selection -- Results of a literature survey of the availability, strength, bondability and treatability of temperate zone hardwoods common to Pennsylvania and the Northeastern United States were reported by Manbeck et al. (1994a). Red oak and red maple were identified as the most abundant species in Pennsylvania forests (18% and 12%, respectively) and yellow poplar ranked tenth (4%). However, yellow poplar is an abundant species in states just south of Pennsylvania. They reported that, of the ten most abundant hardwoods in the region and based on treatability and bondability, red oak and red maple were the least difficult to process as glulam timbers. Yellow poplar ranked one category lower in overall processing difficulty due in large part to relative difficulty in treatability. In addition, red oak and red maple both have very good to excellent structural characteristics (Wood Handbook, 1987; NDS, 1991). Thus, red oak, red maple, and yellow poplar were selected as the species for development of hardwood glulam bridge standards.

Resin System - Room temperature cured phenol-resorcinol formaldehyde (PRF) and resorcinolic formaldehyde resin systems were identified as suitable resin systems for bonding the candidate hardwood species. Glueline shear strengths and delamination characteristics for several room temperature cure and elevated temperature cure resorcinolic and PRF adhesives were found adequate for structural bonding (Manbeck et al., 1994a; Labosky et al., 1993). Several PRF resins were recommended for hardwood glulam manufacture because of their superior performance with hardwoods. Also, melamine was found satisfactory for finger joint fabrication of the three species (Janowiak et al., 1993).

Laminating Procedures - Manbeck et al. (1994a) presented the results of a study to identify the laminating procedures (e.g., surface quality and clamping pressures) necessary for satisfactory glueline performance for the three hardwood species. They concluded: (1) All three species can be end-jointed using any type of structural finger-joint assembly or other end connection provided minimum end joint criteria are met; (2) Planar feed rates should be reduced to assure 14 or greater knife cuts per inch surface dressing-it is preferable that red oak be processed with 16 to 20 knife cuts per inch; (3) Girders should be manufactured with plainsawn lumber; (4) Clamping pressures should be between 175 and 225 psi for red oak, 150 to 200 psi for red maple and 125 to 150 psi for yellow poplar, depending upon surface quality; and (5) Clamping pressure should be maintained for at least ten hours at 70°F.

Preservative Treatment- Oil borne preservatives are recommended for hardwoods by AWPA C14 and AWPA C28 for bridge applications (AWPA, 1995). Creosote was selected as the preservative treatment for the bridge standards because it is readily available commercially and is cost effective (Manbeck et al., 1994a). Blankenhorn et al. (1996) and Baileys et al. (1994) reported that required retention levels of creosote (12 pcf) and penetrations can be achieved in red oak, red maple and yellow poplar by modifying standard treatment processes. Blankenhorn, et al. (1996) identified the treating processes and also demonstrated that the treating process does not negatively affect the glueline strength. They also identified post treatment processes which reduce bleeding from the treated elements. Manbeck et al. (1995) reported that post fabrication creosote treatment of glulam beams from the three hardwood species to approximately 12 pcf retention did not significantly reduce flexural strength or stiffness.

Structural Characteristics- The structural characteristics of hardwood glulam beams manufactured from red maple were reported by Manbeck et al. (1993) Manbeck et al. (1994b) and Janowiak et al. (1995). Compression strengths for red maple and red oak were reported by Janowiak et al. (1994). The flexural performance of yellow poplar and red oak were reported by Moody et al. (1993) and Shedlauskas et al. (1996), respectively. The outcome of these studies was that it is feasible to manufacture glulam sections in each of the three species which: (1) Are efficient in that lower quality lumber (No. 2 or No. 3 VSR) can be used in the inner 50% of the cross section with higher quality lumber used only in the outer laminations; and (2) Have flexural strength of 2400 psi and flexural stiffness of 1,800,000 psi. These studies also demonstrated that ASTM D3737 (ASTM, 1993) calculation procedures satisfactorily predict the flexural properties of hardwood glulam and that the volume effect on the strength properties of the hardwood glulam beams was similar to that for specified in the NDS (1991) for softwood glulam beams. Consequently, the American Institute of Timber Construction (AITC) has recently revised AITC 119-96 (AITC, 1996) to include hardwood glulam beam design and manufacture with cross sections which have laminations of different grades. Prior editions of AITC 119 (AITC, 119-85) required uniform quality laminations throughout the cross section. The study by Janowiak et al. (1995) also demonstrated the suitability of fabricating red maple glulam beams with non-edge glued combination laminations. This development is significant because it allows beams wider than 5.125 inches to be manufactured from nominal 2 x 4 and nominal 2 x 6 material. Limiting required laminating stock to maximum widths of 6 inches (nominal) is a key economical factor for hardwood glulam fabrication.

Connector Performance- The static and cyclic (up to 1 million cycles) load-slip behavior of lag bolts for hardwood glulam bridge applications were determined and reported by Thomforde (1995), Janowiak et al. (1996) and Witmer (1996). The load slip behaviors were subsequently used to model and predict the live load distribution for the hardwood decks and to predict the degree of partial composite action between the deck and the glulam girders. Witmer (1996) reported between 8% and 12% increase in the flexural stiffness of hardwood glulam girders when lag bolt- or Weyco clip-connected to the glulam deck panels. Load-slip behavior of lag-connectors degraded with number of load cycles. However, the residual strength of 75% of the specimens after 1 million cycles still exceeded 1.6 times NDS design values for lag-screws.

Standard Plans- The standard plans for hardwood glulam timber bridges for clear spans between 18 and 90 ft. were completed and published in September, 1994 (PennDOT, 1994). Also, standard specifications for hardwood glulam bridge applications were developed and published in PennDOT Publication 408 (PennDOT, 1994). The standard plans have been used to design and fabricate several bridge projects in Pennsylvania (Manbeck et al., 1996). The standard plans are ASD-based and are described in more detail in the next section of this paper. The standard plans are currently being revised to incorporate results of research completed since publication of the first edition and to convert them to an LRFD-basis and to metric units. The ASD-versus LRFD-outcome comparisons will be completed by December, 1996 and the LRFD and metric version of the standard plans will be published in July, 1997. A summary of the ASD vs. LRFD comparisons is presented in a subsequent section of this paper.

ASD-Based Standard Designs (BLC-560 Series)

Overview

The BLC-560 Series “Standards for Hardwood Glulam Bridge Design” (PennDOT, 1994) were developed cooperatively for PennDOT by researchers at The Pennsylvania State University and by personnel at the engineering firm of Gwin, Dobson and Foreman af Altoona, Pa. Design standards are included for glulam bridge clear spans ranging from 18 to 90 ft. Plans and specifications for longitudinal glulam panel bridges are provided for simple spans from 18 to 22 ft in each of the three hardwood species (red oak, red maple, and yellow poplar) and for longitudinal glulam girder-transverse glulam deck bridges for simple spans from 18 to 90 ft for each of the three hardwood species. The standard plans include general design criteria, superstructure designs for both bridge types, substructure designs for both bridge types and a design example. The 38 sheets contain all the details necessary for design, specification and assembly of the bridge substructure and superstructure.

Design Conditions

Wood- The BLC-560 Series standard plans are based solely on ASD methodologies and were developed in accordance with the AASHTO Bridge Design Specifications (AASHTO, 1992). Allowable design stresses for the three hardwood species were estimated from the results of the Penn State research efforts cited earlier in this paper. Allowable values are summarized in Table 1. Allowable flexural strength and stiffnesses for red maple and yellow poplar are based directly upon the research results of Manbeck et al., (1993), Manbeck et al. (1994) and Moody et al. (1993). Allowable flexural stress and stiffness for northern red oak are for combination A layups as specified in AITC 119-85 and in the NDS (1991). The research by Shedlauskas et al. (1995) which defined more efficient layups was incomplete at the time of publication of the BLC-560 plans. The shear strengths and the bearing strengths are estimates based on values published in the NDS (1991) for similar species and lamination layups. Estimation was necessary for these parameters because AITC had not yet revised AITC 119-85 at the time the plans were published. The design values in the revised version of AITC 119-96 are summarized in Table 2. The allowable design values for girders ate for the cross section layups described in Tables 3 and 4 for the first generation BLC-560 plans and for AITC 119-96, respectively. The allowable design values for the transverse deck and longitudinal deck panels are for lamination layups using No. 2 VSR lumber. Wet use conditions were used for design of all wood components in the bridge. Live load deflections are limited to span/500 for the superstructure. All wood is specified as creosote treated to retention levels of 12 pcf.

Table 1-Allowable design values for glulam elements in the BLC-560 ASD bridge designs (PennDOT, 1994)[a]

Bridge Element	Design Value (psi)	Northern Red Oak (NRO)	Red Maple (RM)	Yellow Poplar (YP)	Moisture Coefficient (Cm)
Transverse[b] Deck Panels	Bending Stress (F_{by})	1800	1800	1400	0.800
	Shear Stress (F_v)	215	205	145	0.875
	Modulus of Elasticity (E_y)	1.6e6	1.8e6	1.4e6	0.833
Longitudinal[c] Deck Panels	Bending Stress (F_{by})	1700	1700	1300	0.800
	Shear Stress (F_{vy})	215	205	145	0.875
	Bearing Stress (F_{cp})	885	615	420	0.530
	Modulus of Elasticity (E_y)	1.6e6	1.8e6	1.4e6	0.833
Longitudinal[d] Girders	Bending Stress (F_{bx})	2400	2400	2400	0.800
	Shear Stress (F_v)	215	205	145	0.875
	Bearing Stress (F_{cp})	885	615	420	0.530
	Modulus of Elasticity (E_x)	1.8e6	1.8e6	1.8e6	0.833

[a]Dry use, normal load duration and single piece laminations.
[b]Values for uniformly distributed No. 2 or better VSR laminations and for decks up to 5.125 in.
[c]Values for uniformly distributed No. 2 or better VSR laminations and for depths up to 12 in.
[d]Values for red oak are for Combination A lamination layups for Northern red oak (AITC, 119-85); values for red maple and yellow poplar are for 2400f-1.8E lamination layups. Lamination layups details given in Table 3.

Table 2-Allowable design values published in AITC 119-96 (AITC, 1996).[a]

Bridge Element	Design Value (psi)	Northern Red Oak (NRO)	Red Maple (RM)	Yellow Poplar (YP)	Moisture Coefficient
Transverse and Longitudinal Deck Panels[b]	Bending Stress (F_{by})[c]	1700	1450	1200	0.800
	Shear Stress (F_{vy})	175	160	135	0.875
	Bearing Stress (F_{cp})	835	590	405	0.530
	Modulus of Elasticity (E_y)	1.5e6	1.3e6	1.2e6	0.833
Longitudinal[d] Girders	Bending Stress (F_{bx})	2400	2400	2400	0.800
	Shear Stress (F_v)	235	220	155	0.875
	Bearing Stress (F_{cp})	1075	895	590	0.530
	Modulus of Elasticity (E_x)	1.8e6	1.8e6	1.4e6	0.833

[a]Values for dry use, normal load duration and single piece laminations.
[b]Panels fabricated with N2 laminations; combination symbol H2, H6, and H10 for NRO, RM and YP, respectively (AITC 119-96).
[c]F_{by} values for panels 12 in. deep; multiply by 1 .10 for 5.125 in deep transverse deck panel application; reduce by $(12/d)^{1/9}$ for longitudinal deck panels deeper than 12 inches.
[d]Values for red oak for 24F-E3 lamination layup; values for red maple for 24F-E4 lamination layup; values for yellow poplar for 24F-E3 lamination layup. Lamination layups shown in Table 4. (AITC 119-96).

Table 3-Lamination layups for girders used in BLC-560 standard plans[a]

Species	Combination Symbol	Outer[b] Tension Zone	Inner[b] Tension Zone	Core[b]	Inner Compression[b] Zone	Outer Compression[b] Zone
Northern Red Oak	A[c]	5% 302-24[d]	-	90% SG<1:8; KS<0.1W	–	5% 302-24
Red Maple	24F-1.8E	10% 2.0E6	15% 1.8E5	50% No. 2 VSR	15% 1.8E3	10% 2.0E3
Yellow Poplar	24F-1.8E	10% 2.0E6	15% 1.8E3	50% No. 2 VSR	15% 1.8E3	10% 2.0E3

[a]For dry use and normal load duration.
[b]Percent of cross section depth and lamination grade; 2.0E6 indicates MOE=1.8e6 and edge knot limited to 1/6 lamination face width.
[c]Combination A as defined by AITC 119-85. Maximum slope of grain (SG) = 1:8; maximum knot size (KS) ≤ 0.1 x lamination width (W).
[d]AITC tension lamination grade.

Table 4-Lamination layups for 24F-E1.8 girders in AITC 119-96 (AITC, 1996).[a]

Species	Combination Symbol	Tension[b] Lamination	Outer[c] Tension Zone	Inner[c] Tension Zone	Core[c] Zone	Inner[c] Compression Zone	Outer[c] Compession Zone
Red Oak[d]	24f-ES	302-24	10% 10E6	15% 1.8E3	No. 2 VSR	15% 1.8E3	10% 2.0E3
Red Maple[d]	24f-E4	302-24	10% 20E6	15% 1.8E3	No. 2 VSR	15% 1.8E3	10% 2.0E3
Yellow Poplar	24f-E3	302-26	10% 2.0E6	15% 1.8E3	No. 2 VSR	15% 1.8E3	10% 2.0E3

[a]For dry use and normal load duration.
[b]AITC tension lamination grade.
[c]Percent of cross section depth and lamination grade; 2.0E6 indicates MOE=1.06e6 and edge knot limited to 116 the face width of lamination.
[d]For members greater than 15 in. deep.
[e]For members greater than 22.5 in deep.

Loads- Design loads were the controlling one of HS-25 or IML-80 for all bridge components. Wheel load distribution factors for girder moments were s/4 for 4 in (nominal decks) and s/5 for 6 in (nominal) decks. Live load distribution factors for deflections were computed as the ratio of the number of lanes to the number of beams. Wheel live load distribution width for longitudinal panel superstructures comply with Section 3.25.3 of AASHTO (1992).

Deck panels- Deck panels for transverse decks were restricted to 5.125 in thickness. Transverse deck panels were all 3 or 4 ft wide and the full width of the bridge. All deck panels are dowel connected with 1.25 in. diameter steel dowels equally spaced at a maximum of 10 in. on center. A 0.25 to 0.375 in. spacing is specified between panels. Longitudinal deck glulam bridges are constructed with 12 to 15 in. thick panels approximately 44 to 48 in. wide. Panels are interconnected with 1 in. diameter A307 steel dowels

Table 5-Typical longitudinal girder sizes in BLC-560 standard plan for hardwood glued-laminated highway bridges (PennDOT 1994).

Species	Simple Span (ft)	Girder Spacing (in)	Curb to Curb Width (ft)	Girder Width (in)	Girder Depth (in)	Bearing Extension (--)
Northern Red Oak[a]	18	29	32	5.125	28.5	N
	30	29	32	5.125	34.5	N
	40	29	32	5.125	42.0	N
	60	29	32	6.750	55.5	N
	80	29	32	6.750	73.5	N
	90	29	32	6.750	84.0	N
Red Maple[b]	18	29	32	5.125	30.0	N
	30	29	32	5.125	34.5	N
	40	29	32	5.125	43.5	Y
	60	29	32	6.750	55.5	N
	80	29	32	6.750	73.5	N
	90	29	32	6.750	82.5	N
Yellow Poplar[b]	18	29	32	6.750	31.5	N
	20	29	32	6.750	33.0	N
	30	29	32	6.750	37.5	Y
	40	29	32	6.750	43.5	Y

[a]Combination A lamination layup (Table 3)
[b]24F-1.8E lamination layups (Table 2)

spaced 2 ft. on center in red oak and red maple and spaced 1 ft on center in yellow poplar. A6x6 transverse glulam spreader beam is also specified at the bridge centerline of longitudinal panel bridges.

Girders- Typical longitudinal girder sizes for each of the three species for selected spans are summarized in Table 5. Girder sizes are specified for clear spans up to 90 ft. for both red oak and red maple. Girder sizes are specified only up to 40 ft. clear spans for yellow poplar because depths became excessive and because beam widths greater than 5.125 in. are not economically feasible for hardwood glulam fabricated with one piece laminations. Yellow poplar required 8.75 in. wide girders to yield reasonable beam depths at spans greater than 40 ft. Therefore, spans greater than 40 ft. were excluded from the design tables. Beam layups are restricted to laminations consisting of one piece of lumber in standard beam widths of 5.125 in. and 6.750 in. Beam widths were restricted to 5.125 in. in all cases except where beam depths were large enough to present potential hydraulic opening difficulties at typical bridge sites. All girder spacings were approximately 2.5 ft. Girder depths ranged from 30 in. to 43.5 in. to 82.5 in. for red maple spans of 18 ft., 40 ft. and 90 ft., respectively. Corresponding depths for red oak girders were 28.5 in., 42 in. and 84 in., respectively. Girder depths for yellow poplar ranged from 31.5 in. to 43.5 in. for clear spans of 18 ft. and 40 ft, respectively. Significantly, all the yellow poplar girders were 6.75 in. wide. All the girders were designed assuming no composite action between the girder and connected deck. Most girder designs were controlled by beam moment; a few at the higher spans were controlled by the deflection criterion. The narrow width of the girders also created some bearing area difficulties at longer yellow poplar spans (6.75 in. wide beams) and at intermediate red maple spans (Between 35 ft and 50 ft.). Special bearing area extensions are specified for these cases. The standard plans also include H-, HS, and IML-80 load ratings for each girder in the design tables.

Connections- All deck to girder connections are galvanized 0.75 in. diameter lag bolts spaced 12 inches on center along each girder. The girders are fastened to the bearing pads with 0.875 in. diameter A325 galvanized steel bolts.

Geometry- The standard plans are detailed for right skews between 45 and 90 degrees and for left skews between 90 and 135 degrees. Details are provided for curb to curb widths of 24 ft., 28 ft. and 32 ft.

Substructure and other details- The standard plans include details for substructures. Details for attachment of the superstructure to existing concrete abutments are included as are details for wood pile and steel pile abutments. Bridge rail details are also included. The rail details are based upon static analysis of the components. Details for cribbing wingwalls and gabion wingwalls are also included in the BLC-560 plans.

Reviews and approvals- The BLC-560 standard designs were reviewed by engineers at PennDOT, FHWA, and USDA Forest Products Laboratory (FPL). Final approval for the standard plans was given during in September, 1994 and have been available for distribution from PennDOT since January 1, 1995.

LRFD-Based Standard Designs (BLC-560M Series)

Major Changes

The major refinements incorporated into the second edition of the BLC-560 Series standard plans are: (1) Incorporation of composite action in the design of longitudinal girders for the girder-transverse deck bridges. Girder stiffness (EI) is increased by approximately 10% when the transverse deck panels are properly connected to the girders (Witmer, 1996); (2) Inclusion of wider (greater than 5.125 in.) multiple piece laminations (e.g., non edge glued 2 x 4's and 2x6's) in girders and longitudinal deck panels. Janowiak et al. (1995) demonstrated the satisfactory performance of such layups for hardwoods. Multiple piece laminations provide more design flexibility (i.e., wider beams and wider girder spacings and deeper longitudinal deck panels can now be specified with economically sized hardwood lumber (less than 5.125 in.) in the overall bridge design; (3) The base strength and stiffness values for the hardwood glulam members are updated to the values recently published in AITC 119-96 (AITC, 1996) (Table 2); (4) Clip connections are added as an alternative to lag bolts for connecting the transverse deck to the girders. Witmer (1996) and Janowiak et al. (1996) verified the adequacy of the clip connectors for hardwoods; (5) Inclusion of transverse deck panel designs for installation on new or existing longitudinal steel girders; (6) Conversion of the standard plans to metric units; and (7) Conversion of the design basis from an ASD to an LRFD format. Comparisons of design outcomes of hardwood bridge girder designs using ASD and LRFD formats are the focus of the remainder of this paper.

LRFD vs. ASD Comparisons

Four factors may contribute to differences between hardwood bridge girder designs using ASD methods prescribed in AASHTO (1992) and the LRFD methods prescribed by AASHTO (1994) and PennDOT. They are: (1) Addition of a live load impact factor (IM); (2) Small changes in the live load distribution factor (LLDF) from s/3048 to s/3000 (s = girder spacing in mm); (3) Changes in the live load specification from HS-25 or IML-80 to PHL-93 (PennDOT, 1995); and (4) The change in format from ASD to LRFD. The first step in our comparison was to evaluate the overall effect of the four factors.

The longitudinal girder designs published in the first edition of the hardwood glulam bridge designs (BLC-560) were the basis for comparison of the ASD and LRFD outcomes. Using ASD, the ratio of maximum design moment (based on service loads) to the allowable moment (M_{max}/M_{all}) was calculated (These were all near 1.0 since moment controlled most of the girder sizes in the BLC-560 designs.). The maximum design moment was based on the controlling one of HS-25 or IML-80 loads, no impact factor, and a LLDF of s/3048. For the same beam size, span and spacing, the ratio of the maximum factored moment to the ultimate moment resistance of the cross section (M_u/M_r) was calculated. The maximum factored moment for LRFD included an impact factor (IM) of 1.165, a LLDF of s/3000, a live load factor of 1.75, a component dead load factor of 1.25, and a wearing surface dead load factor of 1.50. Finally, the %-difference between the ratios for the LRFD and ASD analysis was computed using equation 1.

$$\%\text{-Diff.} = \frac{(M_u / M_r - M_{max} / M_{all})}{(M_{max} / M_{all})}\ 100 \qquad (1)$$

Similar computations were completed for both shear and bearing stresses. The only exception for shear calculations was substitution of shear capacity for moment capacity in equation 1. The only exceptions for the bearing calculations was substitution of end bearing loads for moments in equation 1.

The results of the preliminary comparisons for red oak girders for spans ranging from 18 to 90 ft. (5.5 to 27.4 m) are summarized in Table 6. The percentage difference between the ratio of LRFD to ASD moment capacities decreased from -4 % to -22 % as span increased from 18 to 90 ft. Thus, overall effect of the LRFD specification and PHL-93 loads is to reduce the girder section requirement. Furthermore, the reduction increases with girder span. Similarly, the percentage difference between the ratio of LRFD to ASD shear capacities decreased from -20 to -39 % as span increased from 18 to 90 ft. Thus, the overall effect of the LRFD specification and the PHL-93 loads is to reduce the girder section requirement. The reduction also increases with girder span. The percentage difference between the ratio of LRFD to ASD bearing load capacities ranged from -10.2 to -22.1% as span increased from 18 to 90 ft. This indicates that the overall effect of the LRFD specification and the PHL-93 loads is to reduce bearing requirements. The percentage reduction appears to increase with girder span.

Table 6. Comparison of design outcomes for northern oak glulam bridge girders using AASHTO (1992) ASD and HS-25 or IML-80 loads vs. AASHTO (1994) LRFD and PennDOT PHL-93 loads[a,b]

		Moment Capacity[c]			Shear Capacity[d]			Bearing Capacity[c]		
Span (ft)[e]	Beam Depth (in)	M_u/M_r	M_{max}/M_{all}	% Diff. (LRFD/ ASD)	V_u/V_r	V_{max}/V_{all}	% Diff. (LRFD/ ASD)	P_u/P_r	P_{max}/P_{all} DK	% Diff. (LRFD/ ASD)
18	22.5	0.92	0.95	-3.9	0.68	0.86	-20.2	0.51	0.57	-10.2
20	24.0	0.94	0.99	-5.0	0.66	0.83	-20.6	0.52	0.59	-12.1
25	28.5	0.94	1.00	-5.6	0.59	0.75	-21.6	0.55	0.64	-15.1
30	33.0	0.93	0.99	-6.0	0.54	0.70	-23.2	0.57	0.69	-17.2
35	37.5	0.93	0.98	-5.7	0.50	0.68	-26.7	0.59	0.73	-19.0
40	42.0	0.92	1.02	-10.1	0.46	0.63	-26.8	0.62	0.77	-20.0
45	48.0	0.86	0.97	-11.9	0.42	0.61	-31.3	0.64	0.81	-21. 0
50	52.5	0.86	0.99	-13.9	0.40	0.60	-32.8	0.67	0.84	-21.5
60[f]	55.5	0.82	1.00	-17.7	0.33	0.50	-35.2	0.55	0.70	-22.1
70[f]	64.5	0.80	1.00	-19.7	0.30	0.48	-36.9	0.59	0.76	-22.0
80[f]	73.5	0.80	1.01	-20.9	0.29	0.47	-37.8	0.64	0.82	-22.0
90[f]	84.0	0.77	0.99	-22.0	0.27	0.45	-39.2	0.69	0.88	-22.1

[a]All ASD Calculations are for HS-25 or IML-80 loads and impact factor = 1.0

[b]All LRFD calculations are for PHL-93 loads, impact factor = 1.165, LLDF=s/3000, live load factor=1.75, component dead load factor = 1.25 and wearing surface dead load factor=1.50

[c]LLDF=s/3048 for ASD calculations

[d]LLDF by section 13.6.5.2 of AASHTO (1992).

[e]All girder widths are 5.125 in. unless noted otherwise; all girders spaced 2.42 ft. o.c.

[f]Girder width-6.75 in.

Table 7. Change in unfactored live load moment, shear and bearing capacities of the BLC-560 Series northern red oak girders due to change in load specification from HS-25 or IML-80 to PHL-93.[a]

		%-Change[b,c]		
Span (ft)	Span (m)	Moment	Shear	Bearing
18	5.49	36.8	18.6	22.3
20	6.10	37.8	19.0	19.8
25	7.62	40.3	21 .o	15.8
30	9.14	42.8	22.3	12.4
35	10.67	45.5	17.9	11.6
40	12.19	43.2	15.0	10.6
45	13.72	38.4	13.9	10.0
50	15.24	35.5	13.6	10.1
60	18.29	32.9	13.7	11.5
70	21.34	32.5	14.9	13.9
80	24.38	33.3	17.8	16.6
90	27.43	35.2	20.8	18.9

[a]Impact factor of 1.165 included in PHL-93.

[b]LLDF=s/3000 for all cases

[c]Values represent the percentage change (+ means increase) in unfactored live load capacity due only to change in load specifications.

Work is currently underway to separate the effects of load specification changes and impact factors from the effects of the differences in ASD and LRFD calculation methods. The first step in the process is to separate the effect of the load specification (HS-25 or IML-80 vs. PHL-93) on the unfactored live load moment, shear and bearing capacities for the girders in the BLC-560 standard plans. The results of these calculations ate shown in Table 7. The results indicate that unfactored live load moment capacities of the northern red oak girders in the BLC-560 standards increase by 33 to 45 % when the load specification changes to PHL-93. Unfactored live load shear capacities also increased when the PHL-93 load specification was used (by 14 to 22 %). The unfactored live load bearing capacities increased by 10.0 to 22.3% when the PHL-93 load specification was used. The trends in the data in Table 7 indicate that much of the difference in moment, shear and bearing capacity ratios in Table 6 are load specification change related.

Work is continuing to separate and to interpret the effects of the several interacting factors on the girder design outcomes. After the effects are separated, then it will be clearer why the design outcomes are different and the basis for converting the BLC-560 ASD standard plans to an LRFD format will be identified. These analyses are ongoing. Final outcomes are anticipated by the end of 1996.

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