C5.5 Steel Girders and Beams

C5.5.1.1.1 Policy overview

1 July 2015 ~ End Span Policy
The Bureau generally has followed a policy that the end span should not exceed 54% of the adjacent interior span. Origin of the policy is uncertain but apparently is the result of some study during the design of the first continuous welded plate girder bridge many years ago.

Although not stated in the BDM, Bureau policy is that new and replacement bridges should not require positive connections at the abutment or counterweights in order to prevent uplift. Recent study (2015) based on the LRFD code indicates that the end span should be at least 60% of the adjacent interior span in order to prevent uplift in the fully-constructed condition. In light of this study the end span shall not be less than 60% of the adjacent interior span. Additionally, the end span should generally not be greater than 60% of the adjacent interior span unless there are uplift concerns or other factors present. CWPG bridges with integral or semi-integral abutments are subject to the 60% end span requirements, and should not require end span lengths exceeding 60% due to uplift in the fully-constructed condition since the concrete diaphragm acts as a built-in counterweight and, in the case of integral abutments, have a positive connection. CWPG bridges with stub abutments having large skews and/or horizontal curvature may need end span lengths that exceed 60% in the fully-constructed condition, however, it is generally thought that the number of cases will be relatively few. If uplift concerns are present in the construction phase, contact the Chief Structural Engineer about providing temporary tie-downs or temporary counterweights.

2010 ~ Interior/Exterior Girder Design
During design of the I-74 approach spans the engineering consultant performed a detailed comparison of three options: Case 1, separate interior and exterior design sections, Case 2, same section all girders based on Iowa DOT criteria, and Case 3, same section all girders based on Illinois DOT design criteria. Case 1 with the different design sections clearly had an initial cost savings over either of the single section options. Based on this case study it is desirable to consider the option of using different design sections for interior and exterior girders for major bridges.

C5.5.1.2.3 Fatigue

1 July 2014
Previous to this update, $ADTT_{SL}$ was determined based on the frequency of truck traffic at the center girder of a multi-girder bridge with one lane of traffic in each direction. This same $ADTT_{SL}$ was then applied to each girder in the cross-section. Under the previous policy, $ADTT_{SL}$ was set equal to ADTT*0.85. ADTT is the total truck traffic in both directions and 0.85 is the p-value from AASHTO-LRFD Table 3.6.1.4.2-1. The full ADTT was used because both lanes of traffic are adjacent to the center girder and therefore affect it. The p-value of 0.85 was used, but is somewhat inconsistent with the philosophy that the center girder would experience the full frequency of both lanes of traffic. Perhaps a p-value of 1.00 was assumed to be too conservative. Ultimately applying this $ADTT_{SL}$ to the exterior girder which typically has a larger live load distribution factor and smaller effective slab width was determined to be overly conservative.

After some investigation, it appears an exterior girder design will typically be controlled by fatigue when $ADTT_{SL}$ = 0.60*ADTT*1.00 for the exterior girder versus a center girder design using the $ADTT_{SL}$ in the paragraph above. In this case, ADTT is still the total truck traffic in both directions, but 0.60*ADTT is the number of trucks travelling in one direction and 1.00 is the p-value for only one lane of traffic being available to trucks travelling in one direction.

As a result, the updated BDM policy will follow the AASHTO LRFD code when determining $ADTT_{SL}$. The only difference is Iowa's use of 0.60 rather than 0.55 as found in AASHTO LRFD C3.6.1.4.2 for determining the number of trucks travelling in one direction.

C5.5.1.3.2 Limit states

2011 ~ Strength V Limit State During Construction and Other Revisions

Based on the description in the AASHTO LRFD Specifications of the Strength V limit state it seemed that it was not intended to be checked during construction. However, a steel plate girder example by M.A. Grubb and R.E. Schmidt distributed nationally by the U.S. Department of Transportation (USDOT) and National Steel Bridge Alliance (NSBA) includes Strength V during construction. The description of Strength V notes: "…plus 1.35 times the design live load (or any temporary live loads acting on the structure when evaluating the construction condition), plus 0.4 times the wind load on the structure, plus 1.0 times the wind on the live load. For evaluating the construction condition under the STRENGTH V load combination, the load factor for temporary dead loads that act on the structure during construction is not to be taken less than 1.25 and the load factor for any non-integral wearing surface and utility loads may be reduced from 1.5 to 1.25." Based on the example and other sources it is clear that Strength V should be checked during construction when appropriate, and articles in the design manual have been revised with respect to construction limit states. (There are several other changes, also.) The steel example is available at the following URL:

http://www.virginiadot.org/business/resources/SteelDesignExample.pdf

C5.5.1.4.1.6 Flanges

From 1977 until 2002 the Bureau followed the policy of recommending a flange thickness limit of 2 inches. The limit was stated in FHWA Notices N 5040.23 dated 16 February 1977 and N 5040.27 dated 17 August 1977. The Bureau now uses a larger recommended flange thickness limit of 2.5 inches based on Table 4.4 in *Bridge Welding Code, AASHTO/AWS D 1.5M/D1.5: 2002.* The minimum preheat and interpass temperature generally is the same for plates 1 ½ to 2 ½ inches thick.

In the 1970s the Bureau followed a rule that the top flange area should be at least 45% of the bottom flange area. Although no explanation for the rule is available, the rule probably promoted constructibility by ensuring a certain amount of lateral stiffness for a welded plate girder. Because of the constructibility article in the LRFD specifications [AASHTO-LRFD 6.10.3], the Bureau has rescinded the 45% rule and requires that the designer meet the constructibility provisions in the AASHTO LRFD specifications.

C5.5.1.4.1.12 Deflection and camber

2010 ~ CWPG Camber

Large utility pipes were added to the list of dead loads to consider in camber computations.

C5.5.1.4.1.16 Diaphragms and cross frames

At the time of the January 2006 Bridge Design Manual update the standard cross frames used by the Bureau were redesigned to meet AASHTO LRFD specifications (even though the manual still was based on the AASHTO standard specifications). The following is a summary of the AASHTO and AISC changes that affected the redesign.

- Single angles connected with bolts and welds no longer are permitted to use K = 0.75; K must be 1.0 [AASHTO-LRFD 4.6.2.5]. This 2005 AASHTO LRFD change is in the direction of conservatism and makes published cross frame examples obsolete. Because many cross frame members are at the KL/r<=140 limit for compression members, the change has a significant effect on member size.
- Webs of rolled shapes (and presumably stems of tees) no longer are permitted to be 0.23 inches thick; they now must be 0.25 inches thick [AASHTO-LRFD 6.7.3]. This 1998 or earlier AASHTO LRFD change from the standard specifications is in the direction of conservatism and makes our use of WT 4x9 (with a stem thickness of 0.230 inches) obsolete. We need to use at least a WT 4x10.5.
- Outstanding legs of angles no longer are permitted to have a maximum b/t ratio of 16; they now must have a ratio of 15.89 for A36 steel or less for higher grades of steel or single angles [AASHTO-LRFD 6.9.4.2]. This 1998 or earlier AASHTO LRFD change from the standard specifications is in the direction of conservatism and requires thicker angle legs in some cases.
- For relatively thick angle legs, AISC permits an increase in flexural capacity from $1.25M_y$ to $1.50M_y$. This change in the 2000 AISC single angle specification reduces conservatism in angle capacity for angles with relatively thick legs.

2010 ~ Cross Frame and K-Frame Member Design

Rules added to the 5th Edition of the AASHTO LRFD Specifications in 2010 cover all compression buckling modes and flexure for typical diaphragm and cross frame members, and it is no longer necessary to consult the AISC LRFD Specifications for design of angles and tees.

Where diagonals cross, there is the question of whether the crossing connection can be considered a brace point for out-of-plane buckling. Two papers in AISC's *Engineering Journal* give justification for a brace point if one of the two diagonals is in tension. If both diagonals are in compression, however, the crossing connection is not a brace point for out-of-plane buckling.

Critical cases for design of the cross frames were the following:

- Pier frame: diagonal in completed structure, Strength III for wind, with center brace point
- Intermediate frame: diagonal during construction, Strength III for wind, without center brace point; diagonal during construction, Strength I for deck pour, without center brace point
- Intermediate frame: strut during construction, Strength I for deck pour

C5.5.1.4.1.18 Additional considerations

Below is an example of a moment table and reaction table. The tables may be modified to address particular project needs. When live load distribution factors are determined based on methods other than AASHTO's approximate methods of analysis a note below the table should be added indicating the methodology. Live load distribution factors for curved steel girder projects may be omitted when distribution is based on a more complex analysis such as a 2D grid analysis or 3D finite element analysis, however, a note indicating the analysis methodology should be included.

MOMENT TABLE (ft/kips)												
LOAD NAME	LOAD – kips/ft		POSITIVE MOMENT						NEGATIVE MOMENT			
			SPAN 1		SPAN 2		SPAN 3		PIER 1		PIER 2	
	INT.	EXT.	INT.	EXT.	INT.	EXT.	INT.	EXT.	INT.	EXT.	INT.	EXT.
DC1	1.038	0.880	570	509	570	509	603	535	-3350	-2927	-4865	-4250
DC2	0.213	0.213	135	130	135	130	123	120	-448	-455	-642	-651
DW	0.176	0.176	111	108	111	108	120	101	-440	-385	-633	-550
HL-93 LIVE LOAD + IMPACT			2495	2677	2495	2677	2399	2535	-2480	-2763	-3376	-3735
LIVE LOAD DISTR. FACTOR			0.800	0.829	0.758	0.829	0.785	0.829	0.775	0.829	0.791	0.829

REACTION TABLE (kips)										
LOAD NAME	LOAD – kips/ft		REACTION AT SOUTH ABUT.		REACTION AT PIER 1		REACTION AT PIER 2		REACTION AT NORTH ABUT.	
	INT.	EXT.	INT.	EXT.	INT.	EXT.	INT.	EXT.	INT.	EXT.
DC1	1.038	0.880	36	32	233	196	222	194	40	35
DC2	0.213	0.213	7	7	34	34	34	34	7	7
DW	0.176	0.176	7	6	33	29	33	29	7	6
HL-93 LIVE LOAD + IMPACT			113	97	239	205	246	211	119	102
LIVE LOAD DISTR. FACTOR			0.968	0.829	0.968	0.829	0.968	0.829	0.968	0.829

MOMENT AND REACTION TABLE NOTES:

MOMENTS AND REACTIONS ARE UNFACTORED.

DC1 LOAD VALUE (kips/ft) ONLY INCLUDES CONCRETE DECK WEIGHT AND CONCRETE HAUNCH WEIGHT AND EXCLUDES STRUCTURAL STEEL WEIGHT (GIRDERS, DIAPHRAGMS, CROSS FRAMES, AND LATERAL BRACING).

DC1 MOMENTS (ft/kips) AND REACTIONS (kips) INCLUDE DC1 CONCRETE AND STRUCTURAL STEEL LOADS.

DC2 CORRELATES WITH BARRIER RAIL LOAD WHICH IS DISTRIBUTED EQUALLY TO ALL GIRDERS.

DW CORRELATES WITH FUTURE WEARING SURFACE LOAD WHICH IS DISTRIBUTED EQUALLY TO ALL GIRDERS.