Strength of Beams in Beam-to-Column Connections with Holes in the Tension Flange

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ABSTRACT

A discussion of several approaches to predicting the flexural strength of beams with holes in the tension flanges is presented. Experimental data pertinent to the discussion is then presented, summarized and analyzed. It is observed that although provisions in the AISC 360-16 specification predict a net section fracture of the tension flanges in a number of the experiments, many of those beams were able to achieve their full plastic moment prior to failure. Two alternative models are proposed as improvements to the provisions in AISC 360-16; the merits and limitations of those models are discussed.

Keywords: beam-to-column connection, moment connection, bolted connection, net section fracture, flange plate connection, T-stub connection, double-tee connection.

INTRODUCTION

When a bolted connection is made to the flange of a beam, material is invariably removed, which can require either reinforcing of the flange or a reduction in available strength of the beam. The issue of reduced strength or net sections in flexural members dates back as early as 1891 (Lilly and Carpenter, 1939). The issue was first encountered in the context of plate girders—before the acceptance of modern welding procedures, sections were invariably built up from plates and angles that were riveted together resulting in girders with numerous flange perforations. Before experimental data were available to indicate otherwise, strength calculations were based on the net-section properties of the girders, even though deflections were often computed using gross-section properties.

The 1939 paper by Lilly and Carpenter includes discussion by W. R. Osgood, which refers to an experiment conducted by Friedrich Hartmann, wherein Hartmann concluded that "in the absence of considerations of fatigue, girders of symmetrical cross sections may be designed safely on the basis of gross moment of inertia." Lilly and Carpenter used two experiments to investigate the elastic stiffness and flange stresses of flexural members built up from plates that were riveted together using angles. Based on the findings of their study, they suggested that the design of girders could be based on the gross-section girder properties but proposed an equation resulting in a slightly reduced "effective" moment of inertia. The two girders tested by Lilly and Carpenter, however, were not tested to failure. The extensive discussion that followed the publication of the paper is evidence of just how contentious the issue was. At least 18 different engineers and scientists wrote to express their opinions of Lilly and Carpenter's findings—most expressed their discomfort with the idea of using the gross-section properties for the design of girders, even when designs were limited by elastic allowable stresses.

Today, girders built up from plates and angles that are riveted together are a thing of past, but the idea of using effective section properties or imposing reductions to the available strength of beams and girders persists. One of the most direct applications on these provisions is in the design of bolted moment connections in lateral force resisting systems.

BACKGROUND

AISC General Provisions

The provisions in Section B10 of the 3rd Edition LRFD *Specification* (AISC, 1999) covered the determination of flexural strength of rolled or built-up members with holes in their tension flanges. The provisions consisted of a design check, shown as Equation 1, wherein an engineer investigated the ratio of the net-section-fracture strength to the gross-section-yielding strength, both evaluated using corresponding resistance factors. In cases where the available gross-section-yielding strength governed the behavior of the tension flange, the full plastic moment of the section was available and could be used. When the net-section-fracture

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Paper No. 2015-32

strength governed, however, a reduction in strength was required. This reduction was made by computing an effective area for the tension flange using Equation 2, which included the ratio of resistance factors of 0.75 to 0.90, or ⅚. Using the effective flange area, modified flexural properties were computed and the moment capacity was based on the modified section modulus.

$$0.75F_u A_{fn} \geq 0.90F_y A_{fg} \tag{1}$$

$$A_{fe} = \frac{5}{6}\frac{F_u}{F_y}A_{fn} \tag{2}$$

where

F_y = yield strength of beam material, ksi

F_u = ultimate strength of beam material, ksi

A_{fg} = gross tension flange area, in.^{2}

A_{fn} = net tension flange area, in.^{2}

A_{fe} = effective net tension flange area, in.^{2}

An unfortunate reality of these provisions, however, was that almost none of the members tabulated in the AISC *Steel Construction Manual* (AISC, 2001) would pass the initial design check when two typically sized bolt holes are placed in the flange, as would be typical for a bolted beam-to-column connection. If A992 material is assumed for the beam and it is assumed that two ⅞-in.-diameter bolts are placed in the beam flanges, not a single W-section in the manual would satisfy the criterion shown as Equation 1. As a result, using the 1999 provisions, it is impossible to design to a fully bolted moment connection capable of reaching the plastic moment of the beam without providing reinforcement to the flanges.

The revised provisions appearing in Section F13 of AISC 360-05 (AISC, 2005), which remain unchanged in AISC 360-10 (AISC, 2010b) and AISC 360-16 (AISC, 2016b), are largely the same as those in AISC 1999 except for three subtle, but important changes. The check of net-section-fracture strength versus gross-section-yielding strength now appears as shown in Equation 3. Like before, when this design check is satisfied, no reduction in moment capacity is needed, and the full plastic moment of the beam can be used. The first difference is that the resistance factors of 0.75 and 0.90 have been removed from the design check and from the reduction formula. The second difference is that a factor, Y_T, was added in the design check to guard against the detrimental effect of a high-yield-strength to ultimate-strength ratio. The last difference is in the way that the strength reduction is made for beams failing the design check. Previously, if a section failed the design check shown in Equation 3, an engineer had to recompute the section properties of the beam based on an effective flange area and then base the moment capacity on yielding at that location. This approach has been streamlined by using the ratio of A_{fn} to A_{fg} to modify the nominal elastic section modulus, S_x. This reduced modulus is then multiplied by the ultimate strength, as shown in Equation 4, to arrive at the moment capacity at the net section.

$$F_u A_{fn} \geq Y_T F_y A_{fg} \tag{3}$$

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \tag{4}$$

where

Y_T = 1.0 when $F_y/F_u \leq 0.80$

Y_T = 1.1 when $F_y/F_u > 0.80$

S_x = section modulus for the beam's gross section, in.^{3}

The basis for the changes from the 1999 provisions to the 2005 provisions is discussed by Geschwindner (2010). An important ramification of the changes is that a larger number of W-sections pass the design check, permitting them to be designed for their full plastic moment capacity without flange reinforcement. As before, if A992 material is assumed for the beam and it is assumed that two ⅞-in.-diameter bolts are placed in the beam flanges, more than two-thirds of the tabulated W-sections pass the AISC 360-16 design check shown as Equation 3.

AASHTO Provisions

The American Association of State Highway Transportation Officials (AASHTO) specification (AASHTO, 2016) includes a provision to limit stress at net sections of girders. The provision, shown as Equation 5, limits the stress in the tension flange, f_l, under strength limit loading and during construction. Rewriting the equation, it can be shown that the moment capacity of the member at the net section is limited by the smaller of the yield moment, $F_y S_x$, or the moment corresponding to fracture, as shown in Equation 6. The value of 0.84 found in both equations is the ratio of the AASHTO resistance factor for fracture, 0.80, to the factor for yielding, 0.95. It should be noted that a comparison of the 0.84 factor in Equation 6 to the factor of ⅚ in Equation 2 must include the understanding that the AASHTO specification is based on a different probabilistic model for loading than the AISC specifications, which refers to ASCE-7-10 (2010):

$$f_l \leq 0.84\left(\frac{A_{fn}}{A_{fg}}\right)F_u \leq F_y \tag{5}$$

$$M_n = 0.84\left(\frac{A_{fn}}{A_{fg}}\right)F_u S_x \leq F_y S_x \tag{6}$$

Because the focus of this paper is beams and beam-to-column connections in buildings, wherein the sections, connections, and splices are often proportioned differently than in bridges, the AASHTO provisions are included for sake of completeness but will not be discussed further.

AISC Seismic Provisions

When beam-to-column connections are designed for special moment frames and intermediate moment frames, one option for satisfying the conformance demonstration required in the AISC *Seismic Provisions* (AISC, 2010a) is to select a connection prequalified by the AISC Connection Prequalification Review Panel (CPRP) (AISC, 2011). Two connections in the current AISC 358 specification, the bolted flange plate (BFP) and Kaiser bolted bracket (KBB) connections, and one more to appear in the next edition of the specification (AISC, 2016b), the double-tee (DT) connection, employ connections whereby the beam is bolted through its flanges.

The design procedure for the BFP connection includes a check of the beam net section to make sure that the beam can reach its full plastic moment. The check is written such that the maximum bolt diameter is determined, assuming that standard holes are used in the beam flange. Although this check is expressed differently, it is algebraically the same as the provisions in Section F13 of AISC 360-16 as shown in Equation 3.

The KBB provisions make no mention of a check for the net section of the connected beam but do employ "clamp plates" on the inside faces of the beam flanges to limit distortion and strains at the net section due to local buckling and necking (AISC, 2011; Kasai and Bleiman, 1996). The commentary to the KBB provisions notes that in similar experiments performed with and without clamp plates, those without clamp plates failed by necking and then fracture through the net section of beam flanges, whereas those experiments with the clamp plates failed via yielding and fracture through the gross section of the beam flange outside of the clamped region. The provisions also mention the reduced strength of the column resulting from drilling holes. In step 7 of the design procedure, a check of the column net section is performed that is consistent with Section F13 of AISC 360-16 shown in Equation 3. The commentary cites work by Masuda et al. (1998) that showed that removing as much as 30 to 40% of the column flange resulted in only a 10% reduction in yield moment.

The 2016 AISC 358 specification (AISC, 2016a) will include double-tee standards that require the strength corresponding to fracture on the net plastic section to be greater than the plastic moment of the gross section. Written in terms of the expected strengths, $R_y F_y$ and $R_t F_u$, this requirement can be written mathematically as shown in Equation 7. Restricting the discussion to A992 steel where $R_y = R_t$, it can be shown that the full plastic moment capacity of the section is available provided that $Z_{x,net}/Z_x$ is not less than $F_y/F_u = 0.7692$.

$$Z_{x,net} R_t F_u \geq Z_x R_y F_y \tag{7}$$

The introduction of Equation 7 was born from the idea that the provisions in AISC 360 were too conservative, as was evidenced by experimentation, and that the comparison of net fracture and gross yielding limit states should be made at the plastic moment as opposed to the yield moment. Because the context within which this equation is presented is that of a full-strength moment connection, no mention is made of a reduced moment capacity of the beam in the case that Equation 7 is not satisfied.

SUMMARY OF RECENTLY ADDED EXPERIMENTAL DATA

During an investigation of the moment capacity of T-stub connections at the Georgia Institute of Technology (Smallidge, 1999; Swanson, 1999; Schrauben, 1999; Swanson and Leon, 2000), several fully bolted moment connections were able to develop moments in the beams that exceeded the plastic moment of the sections. Further, in studies on bolted flange plate connections at the University of Texas, University of Illinois, and the University of California–San Diego (Larson, 1996; Schneider and Teeraparbwong, 2002; Sato et al., 2007), several more moment connections with bolted flanges were also able to develop moments in the beams that exceeded the plastic moment of the sections. Finally, in experiments on steel girders with holes in the tension flanges conducted at the University of Minnesota (Altstadt, 2004), three of four specimens were able to develop bending moments that exceeded the plastic moment of the sections. In all of these cases, the beams failed to satisfy the net-section-facture design check in Section B10 of the 1999 provisions, and in several cases, the beams failed to satisfy the net-section-facture design check in Section F13 of AISC 360-16.

The following sections summarize recent research data that are applicable to the current discussion. The first three programs—conducted at Georgia Tech, the University of Texas, and the University of Illinois—were completed as part of the SAC Steel Project in the aftermath of the 1994 Northridge, California, earthquake; the fourth program, conducted at UC San Diego, focused on bolted flange plate connection performance in special moment frames; and the last program was conducted at the University of Minnesota in 2003 to investigate the rotational capacity of girders constructed from high-performance steel (50W and 70W). An additional test program, conducted in 2004 at the University of Cincinnati to provide additional data to answer questions about the influence of fabrication methods on the strength and ductility of net sections (Yuan, 2005), is not included in this work because the experiments were of specimens loaded axially instead of flexurally.

Georgia Tech T-Stub Tests

As part of the SAC Steel Project conducted in the aftermath of the Northridge earthquake, a series of eight full-scale beam-column subassemblies was tested at the Georgia Institute of Technology in 1998 (Schrauben, 1999; Smallidge, 1999; Swanson 1999). These assemblies, summarized in Table 1, consisted of cantilever beams connected to pinned-pinned columns using fully bolted T-stub connections to resist moment and a shear tab connections to resist shear. Values reported for F_y and F_u in Table 1 were measured during material testing performed at Georgia Tech. The critical section noted in the table is the net section in the beam at the row of bolts farthest from the face of the column. The connections were loaded by applying a displacement to the ends of the cantilever beams.

Connections FS-01 and FS-02 were designed as partial-strength top-and-seat angle connections and are not reported herein. Connection FS-03 failed when the stem of one of the T-stubs connecting the beam to the column sustained a net-section fracture after a limited amount of flange yielding was noted in the beam. Testing of connection FS-04 was stopped after the beam developed a plastic hinge and the flanges sustained significant inelastic local buckling. Testing of connections FS-05, FS-06 and FS-08 was stopped after the beam developed plastic hinges and the flanges and webs sustained significant inelastic local buckling. Connection FS-07 failed by a net-section fracture of the beam flange, but only after significant yielding and localized flange buckling had occurred. The net-section beam-flange fracture began when the material between the bolt hole and the edge of flange fractured, as shown in Figure 1. The fracture then progressed across the remainder of the flange and into the web, as shown in Figures 2 and 3, before the experiment was stopped. Connection FS-09 failed when the bolts connecting the T-stub to the column flange fractured, though Schrauben (1999) reported that yielding was observed in the beam flanges and web prior to bolt failure. Testing of

Fig. 1. Initial fracture in the beam flange of connection test FS-07.

Fig. 2. Beam from connection FS-07 after disassembling the connection.

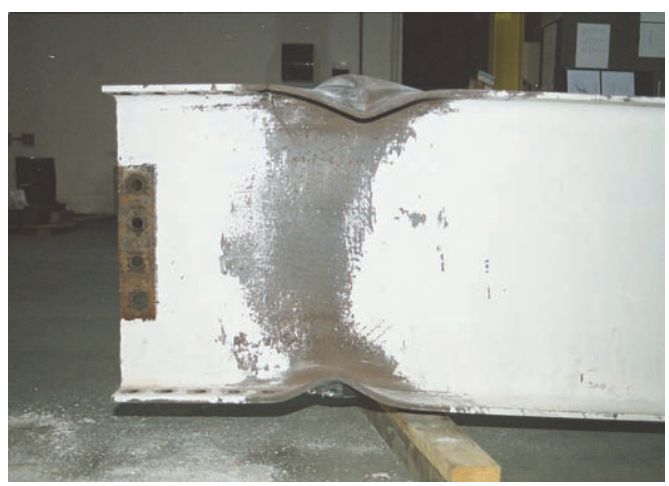


Fig. 3. Beam from connection FS-07. White-wash spalling indicates the formation of a plastic hinge.

Table 1. Details of Georgia Tech Connection Tests												
					Measured				@ Column Face		@ Crit Net Section	
Test ID	Beam Section	Bolt Diameter (in.)	Hole Diameter (in.)	Beam Grade	F_y (ksi)	F_u (ksi)	F_yZ_x (k-in.)	Expected Failure	$M_{Failure}$ (k-in.)	% Above	$M_{Failure}$ (k-in.)	% Above
FS-03	W21×44	⅞	0.938	A572-50	58.0	71.0	5,533	T-stub NSF	5,773	4%	5,405	–2%
FS-04	W21×44	1	1.063	A572-50	58.0	71.0	5,533	Beam FLB	5,949	8%	5,526	0%
FS-05	W24×55	⅞	0.938	A572-50	61.0	76.0	8,235	Beam WLB & FLB	9,363	14%	8,629	5%
FS-06	W24×55	1	1.063	A572-50	61.0	76.0	8,235	Beam WLB & FLB	8,642	5%	8,028	–3%
FS-07	W24×55	⅞	0.938	A572-50	61.0	76.0	8,235	Beam NSF	9,205	12%	8,483	3%
FS-08	W24×55	1	1.063	A36	53.8	70.7	7,263	Beam WLB & FLB	8,254	14%	7,527	4%
FS-09	W27×84	⅞	0.938	A572-50	59.1	78.3	14,420	T-stub T-Bolts	17,794	23%	16,105	12%
FS-10	W27×84	1	1.063	A572-50	59.1	78.3	14,420	Beam FLB/ Fixture	18,269	27%	16,660	16%
WLB - Web Local Buckling FLB - Flange Local Buckling				NSF - Net Section Fracture						Ave: 13% StDev: 8%		Ave: 4% StDev: 6%

connection FS-10 was discontinued when the connection at one end of the column failed. Schrauben reported yielding and local buckling in the beam flanges and web along with extensive yielding in the T-stubs prior to stopping the test.

In all cases—even FS-03, which was stopped after a T-stub failure—the maximum moment developed at the face of the column during the experiment exceeded the expected plastic moment[1] of the beam. The experimental moment at the column was, on average, 13% higher than the expected plastic moment of the beam. Alternatively, the experimental moment at the critical beam section was, on average, 4.2% higher than the expected plastic moment of the beam.

University of Texas Tests

A series of five full-scale beam-column subassemblies was tested at the University of Texas at Austin in 1996 (Barbaran, 1996; Larson, 1996). These assemblies consisted of cantilever beams connected to pinned-pinned columns using a shear tab to resist shear and, in most cases, fully bolted T-stubs to resist moment. The first specimen was designed with a shear-only connection—with a shear tab but without T-stubs—so as to investigate the contribution of the shear tab and beam web to the moment strength of the connection. The second and third specimens were designed both with a shear tab and T-stubs, but the T-stubs were configured to provide only a partial-strength beam connection. The fourth and fifth specimens, indicated in Table 2 as DT04 and DT05, were designed with both a shear tab and T-stubs, with the tees in DT-04 proportioned to transmit 100% of the beam plastic moment to the column and the tees in DT05 proportioned to transmit approximately 125% of the beam's plastic moment (Barbaran, 1996). Values reported for F_y and F_u in Table 2 were measured during material testing performed at the University of Texas at Austin as reported in Larson (1996) for static loading of flat coupons cut from the beam flange parallel to the direction of rolling. The critical section noted in the table is the net section in the beam at the row of bolts farthest from the face of the column, though it should be noted that Larson and Barbaran indicated the presence of vacant holes farther from the column face in the beams of some specimens. The connections were loaded by applying a displacement to the ends of the cantilever beams.

Connection DT-04 failed when the bolts connecting the T-stubs to the column flange fractured. After disassembling the connection, small fractures near the bolt holes in the flange of the beam were noted as is shown in Figure 4. Testing of connection DT-05 was stopped when a fracture of the beam flange, shown in Figure 4, was noticed. Comparing the maximum moment observed at the critical section to the expected plastic moment of the section shows that the beams exceeded the expected plastic moment by margins of 3% and 19%.

1 When the yield strength was measured, either through mill certification or independent testing, the expected plastic moment is defined as $M_{pe} = Z_x F_{y,measured}$. In other cases, the expected plastic moment is defined as $M_{pe} = Z_x R_y F_y$.

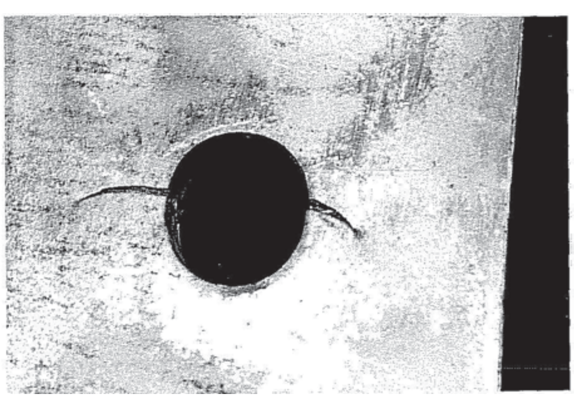
Table 2. Details of University of Texas at Austin Connection Tests												
		Bolt	Hole		Measured				@ Column Face		@ Crit Net Section	
Test ID	Beam Section	Diameter (in.)	Diameter (in.)	Beam Grade	F_y (ksi)	F_u (ksi)	F_yZ_x (k-in.)	Expected Failure	$M_{Failure}$ (k-in.)	% Above	$M_{Failure}$ (k-in.)	% Above
DT-04	W36×150	1¼	1.313	A36	37.7	57.2	21,904	T-Bolts/ NSF	25,862	18%	22,533	3%
DT-05	W36×150	1¼	1.313	A36	37.7	57.2	21,904	Beam NSF	30,016	37%	26,152	19%
NSF - Net Section Fracture									Ave: 28% StDev: 13%		Ave: 11% StDev: 12%	

University of Illinois Tests

Again, as part of the SAC Steel Project, a series of eight full-scale beam-column subassemblies was tested at the University of Illinois (Schneider and Teeraparbwong, 2002). These assemblies, summarized in Table 3, consisted of cantilever beams connected to columns using bolted flange-plate connections to resist moment and shear tab connections to resist shear. Six of the tests incorporated W24×68 beam sections, and two incorporated W30×99 beam sections. Column sections were W14×120, W14×145 or W14×211 sections. Values reported for F_y and F_u in Table 3 are those reported on manufacturer's mill reports. The critical section noted in the table is the net section in the beam at the row of bolts farthest from the face of the column. The connections were loaded by applying a displacement to the free ends of the cantilever beams.

In one case (BFP-01), the failure was described as a heat-affected-zone fracture of the weld connecting the flange plate to the column flange. In a second case (BFP-08), the failure was described as a net-section fracture of the flange plate connecting the beam flange to the column. In all six remaining cases, the failure was described as a net-section fracture of the beam section. Oversized holes were used in the beam flanges in specimens BFP-01 and BFP-06. In specimens BFP-05 and BFP-07, "clamp plates" were included in the grip of the bolts at the critical section in an effort to push flange local buckling away from the net section of the beam and mitigate its effect on the net-section strength. Because these plates were not welded to the beam flange, they were not treated as flange reinforcement herein.

In all eight experiments—even BFP-01, which failed with a HAZ fracture, and BFP-08, which failed with a flange-plate fracture—the moment developed at the column face exceeded the expected plastic moment of the beam. The experimental moment at the column face was, on average, 23% greater than the expected plastic moment of the beam. Alternatively, the experimental moment at the critical



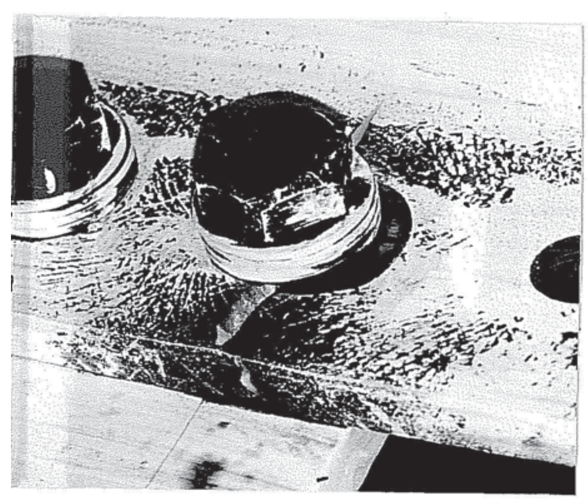


Fig. 4. Beam flange fractures in the University of Texas at Austin tests (Larson, 1996:) specimen DT-04 (left) and specimen DT-05 (right).

Table 3. Details of the University of Illinois Tests

Test ID	Beam Section	Bolt Diameter (in.)	Hole Diameter (in.)	Beam Grade	Mill Certified F_y (ksi)	Mill Certified F_u (ksi)	Mill Certified F_yZ_x (k-in.)	Expected Failure	@ Column Face $M_{Failure}$ (k-in.)	@ Column Face % Above	@ Crit Net Section $M_{Failure}$ (k-in.)	@ Crit Net Section % Above
BFP-01	W24×68	1	1.250	A572-50	56.0	74.0	9,912	HAZ Fracture	11,606	17%	9,377	–5%
BFP-02	W24×68	1	1.063	A572-50	56.0	74.0	9,912	Beam NSF	12,144	23%	10,868	10%
BFP-03	W30×99	1	1.063	A572-50	53.5	67.5	16,692	Beam NSF	21,362	28%	17,725	6%
BFP-04	W24×68	1	1.063	A572-50	56.0	74.0	9,912	Beam NSF	12,337	24%	10,505	6%
BFP-05	W30×99	1	1.063	A572-50	53.5	67.5	16,692	Beam NSF	21,390	28%	17,748	6%
BFP-06	W24×68	1	1.250	A572-50	56.0	74.0	9,912	Beam NSF	12,158	23%	10,616	7%
BFP-07	W24×68	1	1.063	A572-50	56.0	74.0	9,912	Beam NSF	12,379	25%	11,078	12%
BFP-08	W24×68	1	1.250	A572-50	56.0	74.0	9,912	FP Fracture	11,702	18%	9,582	–3%

HAZ - Heat Affected Zone FP - Flange Plate Ave: 23% Ave: 5%
NSF - Net Section Fracture StDev: 4% StDev: 6%

BFP-01: Oversize Holes, Slip Critical Design BFP-05: Used Clamp Plates Under Last Two Rows of Bolts
BFP-06: Oversized Holes BFP-07: Used Clamp Plates Under Last Two Rows of Bolts

Table 4. Details of the UC San Diego Tests

Test ID	Beam Section	Bolt Diameter (in.)	Hole Diameter (in.)	Beam Grade	Measured F_y (ksi)	Measured F_u (ksi)	Measured F_yZ_x (k-in.)	Expected Failure	@ Column Face $M_{Failure}$ (k-in.)	@ Column Face % Above	@ Crit Net Section $M_{Failure}$ (k-in.)	@ Crit Net Section % Above
BFP-11	W30x108	1	1.063	A992	52.0	77.5	17,992	Beam FLB & NSF	23,643	31%	20,985	17%
BFP-12	W30x148	1	1.063	A992	58.5	80.0	29,250	Beam LTB	40,381	38%	33,350	14%
BFP-13	W36x150	1	1.063	A992	58.0	75.0	33,698	Beam FLB & NSF	47,375	41%	40,937	21%

FLB - Flange Local Buckling F_y and F_u for BFP-13 taken from Mill Certificates Ave: 37% Ave: 17%
NSF - Net Section Fracture StDev: 5% StDev: 4%

section of the beam was, on average, 4.8% higher than the expected plastic moment of the beam.

University of California, San Diego Tests

A series of three full-scale bolted flange plate connections were tested at the University of California–San Diego to examine performance in special moment frames (Sato et al., 2007).[2] These assemblies, summarized in Table 4, consisted of cantilever beams connected to columns using bolted flange-plate connections to resist moment and shear tab connections to resist shear. Values reported for F_y and F_u were determined by coupon testing except for BFP-13, which are those reported on manufacturer's mill reports. The critical section noted in the table is the net section in the beam at the row of bolts farthest from the face of the column. The connections were loaded by applying a displacement to the free ends of the cantilever beams.

In BFP-11 and BFP-13, yielding and local buckling of the beam flanges and webs were noted prior to a fracture of the beam flange at the critical section. In both cases, some lateral-torsional buckling of the beam was observed. In BFP-12, lateral-torsional buckling of the beam led to the discontinuation of the experiment prior to failure of the beam, column or connection. In all three experiments—even BFP-12—the

2 Experiments BFP-1, BFP-2 and BFP-3, as reported in Sato et al., have been renumbered herein as BFP-11, BFP-12 and BFP-13, respectively, for ease of reference and to avoid confusion with the Illinois tests.

moment developed at the column face exceeded the expected plastic moment of the beam. The experimental moment at the column face was, on average, 37% greater than the expected plastic moment of the beam. Alternatively, the experimental moment at the critical section of the beam was, on average, 17% higher than the expected plastic moment of the beam.

University of Minnesota Tests

A series of eight experiments on high-performance steel plate girders was carried out at the University of Minnesota in an effort to quantify the rotational capacity of these sections (Altstadt, 2004). The girders were detailed with extremely thick top flanges to simulate the distribution of strain present in a fully composite bridge design. Specifically, the researchers were interested in determining if the newer high-performance steel possessed sufficient ductility—and thus, rotational capacity—to reach a full plastic moment, even under the severe strain distributions found in composite sections. The girders were tested by applying a point load at the mid-span of the girders. Four of these eight girders included a pair of bolted splices situated symmetrically about the mid-span of the girders. All of the bolt holes for these splices were drilled. The girders were built up of 8-in. × 3½-in. top flanges, 29-in. × ¾-in. web plates and 8-in. × ¾-in. bottom flanges. Additional details are provided in Table 5.

Three of the four girders failed with the development of a net-section fracture in the tension flange, with the fourth girder failing via lateral-torsional buckling prior to fracture of the tension flange (necking of the tension flange was observed, however). In three of four cases, the moment developed at the critical section exceeded the expected plastic moment of the section. The moment on the critical section was, on average, 4.3% larger than the expected plastic moment.

DISCUSSION

Considering all of the experiments described in the previous sections, there are 25 relevant data points. Considering all 25 experiments, the strength of the beams at the critical net sections was, on average, 5.7% larger than the expected plastic moment.

Because the strength of connection FS03 was limited by a failure mode not associated with net fracture of the beam flange, that experimental data point represents a lower bound to the net-section fracture strength of the beam. The same argument can be made for connections BFP01, BFP08 and GIR-7. If these experiments are excluded, the strength of the remaining beams at their critical net sections was, on average, 7.0% larger than the expected plastic moment. Going a bit further, if the scope of this paper is limited to sections with proportions similar to those expected in moment frames, GIR-6, GIR-8 and GIR-10 could also be excluded. In that case, the strength of the 18 remaining beams was, on average, 8.7% larger than the expected plastic moment with a corresponding standard deviation of 6.7%. It can also be noted that in only one of the remaining cases, FS-06, was the maximum moment at the critical net section lower than the expected plastic moment of the beam.

The remaining 18 admissible data points are shown as bold diamonds in Figure 5,[3] where the independent parameter is presented as the ratio of A_{fn} to A_{fg} and the dependent parameter is the measured moment capacity normalized by the expected plastic moment, or M_n/M_{pe}. A regression analysis of these 18 data points, shown as a dashed black line in Figure 5, yields a slope of 1.136 with an R^2 of 0.5448.

The cluster of gray data points in Figure 5 represents the normalized moment capacity of all tabulated W-shapes with ratios of A_{fn} to A_{fg} associated with two standard holes in each flange for bolts ranging from ½ in. to 1½ in. diameter using the provisions in Section F13 of AISC 360-16 (data points where a reduction in strength was not required are not shown). The cluster of gray data points can be conveniently represented using a line with a slope of 1.139. Note that the experimental data lie well above the line representing the provisions in Section F13 of AISC 360-16. Further, note that the provisions include a discontinuity for the moment capacity at the ratio of $A_{fn}/A_{fg} = 0.7692$, which corresponds to the ratio of $Y_T F_y/F_u$ for A992 steel, below which the provisions are particularly conservative. Finally, while the cluster of gray data points in Figure 5 represents all combinations of standard bolt holes in all possible beams without regard to workable gages or minimum edge distances, the author notes that fewer points would likely result if workable gages and minimum edge distances were taken into account.

Proposed Model 1

As was noted in the previous section, there is a discontinuity in the AISC 360-16 provisions at $A_{fn}/A_{fg} = Y_T F_y/F_u$. For A992 steel, this discontinuity corresponds to an offset in the curve of $M_n/M_{pe} = 1 - 0.8761 = 0.1239$ when $A_{fn}/A_{fg} \le Y_T F_y/F_u$. One potential improvement would be to remove the discontinuity from the provisions in Section F13 of AISC 360-16 by simply shifting the sloped portion of the strength model shown in Figure 5 upward to provide a continuous, but not smooth, solution. This results in the solution shown as the lower of the two black lines in Figure 6, which will be referred to herein as proposed model 1a. The black diamonds in Figure 6 again represent the 18 pertinent data points.

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3 Note that data points for BFP-03 and BFP-05 lie on top of each other, making it appear as though there are only 17 data points in the figure.

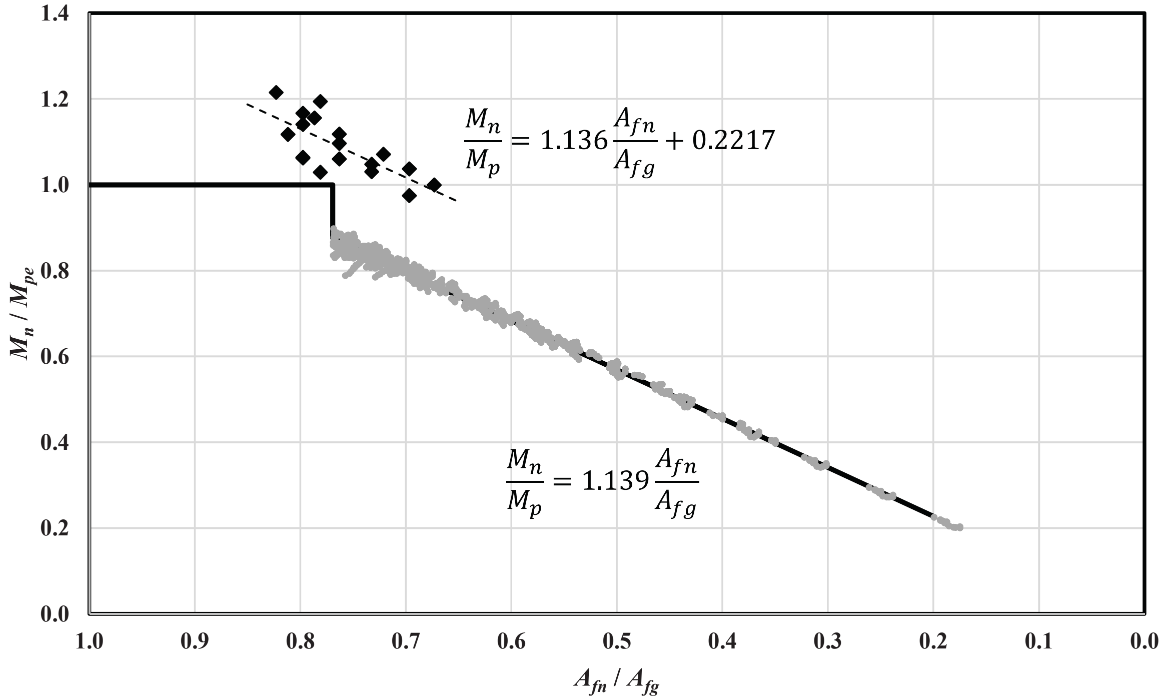

Fig. 5. Solution space for W-shapes with A992 steel for provisions in AISC 360-16.

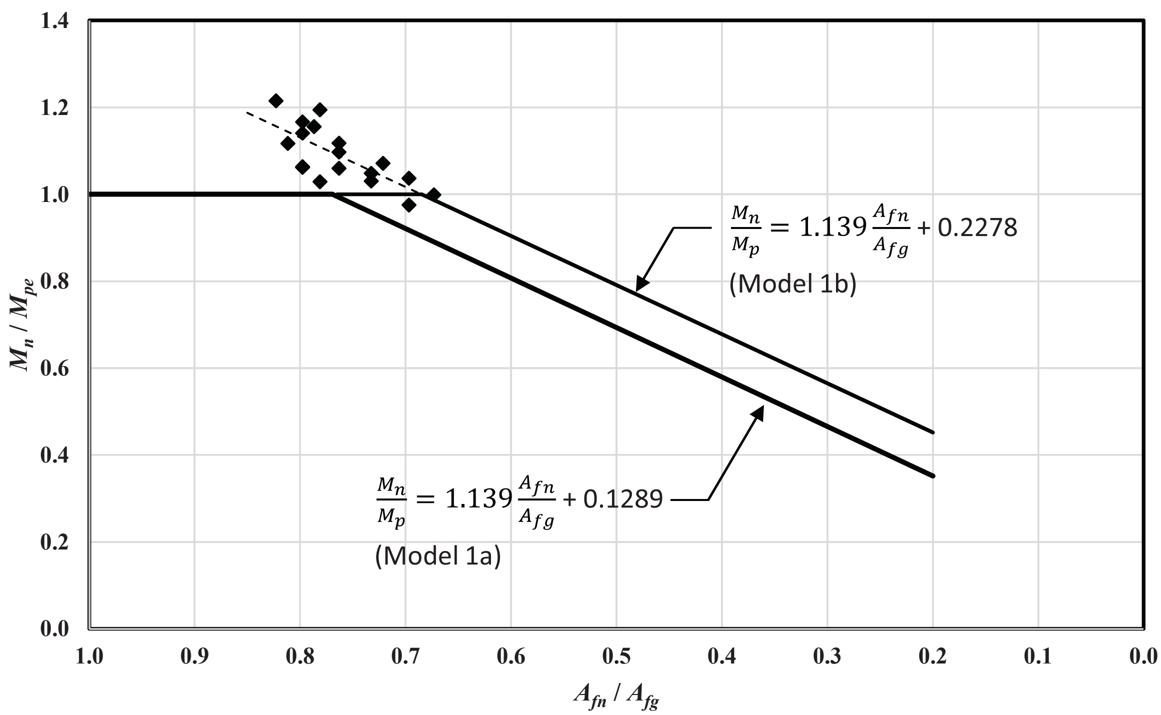

Fig. 6. Solution space for W-shapes with A992 steel for proposed models 1a and 1b.

Table 5. Details of the University of Minnesota Tests											
				Mill Certified				@ Center Line		@ Crit Net Section	
Test ID	Beam Section	Bolt Diameter (in.)	Tension Flange	F_y (ksi)	F_u (ksi)	Plastic Moment (k-in.)	Expected Failure	$M_{Failure}$ (k-in.)	% Above	$M_{Failure}$ (k-in.)	% Above
GIR-6*	Built-Up	1	A709-70W	90.0	103.0	3,141	NSF	3,460	10%	3,007	–4%
GIR-7	Built-Up	⅞	A709-70W	88.1	101.2	3,031	LTB	3,634	20%	3,091	2%
GIR-8	Built-Up	1	A709-70W	88.1	101.2	3,031	NSF	3,566	18%	3,097	2%
GIR-10**	Built-Up	⅞	A572-50	55.0	71.5	2,381	NSF	3,215	35%	2,794	17%
*GIR-6 Incorporated Oversized Holes **R_yF_y and R_tF_u Reported for GIR-10 (F_y and F_u at nominal values)			NSF - Net Section Fracture LTB - Lateral-Torsional Buckling					Ave: 21% StDev: 10.4%		Ave: 4.3% StDev: 9.2%	

When $F_uA_{fn} < Y_TF_yA_{fg}$, this strength model, model 1a, could be presented mathematically as:

$$M_n = \frac{F_uA_{fn}}{A_{fg}}S_x + F_yS_x\left(\frac{Z_x}{S_x} - Y_T\right) \tag{8}$$

If the discussion is restricted to W-shapes rolled from A992 steel, where $F_u/F_y = 1.300$, $Y_T = 1.0$, and it is observed that on average $Z_x/S_x = 1.147$, then when $F_uA_{fn} < Y_TF_yA_{fg}$, model 1a could be presented mathematically as:

$$M_n = 1.139F_yZ_x\left(\frac{A_{fn}}{A_{fg}} + 0.1132\right) \le F_yZ_x \tag{9}$$

One shortcoming of model 1a, presented as Equation 9, is that it would still require a strength reduction for many beams that were shown experimentally to reach their expected plastic moment. Nine of the 18 beams would require a reduction in strength, wherein only one of those nine failed to achieve its expected plastic moment. In a different context, if these nine excluded beams were employed in a BFP connection, they would either be deemed not conforming to the specification or would require a reinforcement of the net section.

To address this issue, the constant in Equation 9 can be increased to shift the value of A_{fn}/A_{fg} that results in a need to reduce the moment strength of the beam. Using a constant of 0.2 instead of 0.1132—as is shown in Equation 10 and as shown as the upper black line in Figure 6, hereafter referred to as proposed model 1b—results in a reduced beam strength that matches very well with the regression analysis of the 18 experimental data points. This would correspond to the requirement of a strength reduction for beams with A_{fn}/A_{fg} less than 0.6850, which in turn corresponds to being able to employ the full plastic moment for all but one of the beams in the 18 data points.

$$M_n = 1.139F_yZ_x\left(\frac{A_{fn}}{A_{fg}} + 0.2\right) \le F_yZ_x \tag{10}$$

Proposed Model 2

The provisions in the double-tee chapter of AISC 358-16 address limits in the net section of the tension flange that will ensure that the plastic moment of the beam can be reached prior to a net-section fracture of the tension flange occurring. As a result, those provisions make no mention of a reduction in the flexural strength of a beam when the design check shown as Equation 7 is not satisfied. Still, a strength reduction can be inferred from Equation 7 if one assumes that the flexural strength would be equal to the lesser of the net-section moment at F_u and the gross-section plastic moment strength represented by the left-hand side and right-hand side, respectively, of the inequality in Equation 7. Doing so, and limiting the discussion to A992 steel, one can show that the moment strength when $Z_{x,net}/Z_x$ is less than 0.7692 can be represented as shown in Equation 11 and as shown in Figure 7 where the independent parameter is presented as the ratio of $Z_{x,net}$ to Z_x.

$$M_n = F_uZ_{x,net} = 1.300F_yZ_{x,net} \tag{11}$$

If the 18 admissible data points shown in Figures 5 and 6 are also plotted on this chart, a regression analysis can be performed as is represented by Equation 12 and shown in Figure 7. The R^2 value associated with this regression analysis is 0.4219. This relationship can be extrapolated to provide a moment capacity as shown in Figure 7, hereafter referred to as proposed model 2.

$$M_n = 1.895F_yZ_x\left(\frac{Z_{x,net}}{Z_x} - 0.2699\right) \le F_yZ_x \tag{12}$$

Proposed Model 3

If one considers the strength provided by Equation 4 but adds to it the plastic moment provided by the web of the sections, the following equation results, which will be referred to as proposed model 3a.

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x + F_u Z_{x,web} \le F_y Z_x \quad (13)$$

The first term to the left of the inequality in Equation 13 represents the strength of the flange at fracture, while the second term to the left of the inequality represents the strength of the web when the flange is at fracture. It is acknowledged that, because the first term includes S_x, it inherently includes some contribution from the web of the section. It is noted, however, that S_x is dominated by the area of the flange. If model 3 is applied to all tabulated W-shapes with ratios of A_{fn} to A_{fg} associated with two standard holes in each flange for bolts ranging from ½ in. to 1½ in. diameter, then the cluster of gray data points shown in Figure 8 results (data points where a reduction in strength was not required are not shown). It can be seen that this leads to some scatter when plotted as a function of A_{fn}/A_{fg}, but the results provide a strong correlation to the experimental data.

Figure 9 shows the correlation that is obtained from model 3a when the model is applied to the 18 admissible connections using measured material strengths and compared to the experimental results. In Figure 9, both axes have been normalized by the expected plastic moment. The correlation is shown as the dashed line, which has a slope of 1.116 (i.e., a professional factor) and an $R^2 = 0.7278$. The regression analysis is performed on the 16 solid data points because experiments FS-09 and DT04 both failed via bolt fracture. These two data points, represented as open diamonds in Figure 9, were not included in the regression. In addition to the strong correlation, model 3a also has the advantage of correctly indicating which 16 of the 18 beams successfully reached their expected plastic moment before failing.

The possibility of capturing the web strength using F_y instead of F_u, as shown in Equation 14 and referred to herein as proposed model 3b, was also considered. The professional factor that was found for model 3b was 1.030 with an $R^2 = 0.7609$, both of which are better than for model 3a, but model 3b predicted that 6 of the 18 beams would fail via beam net fracture prior to reaching their expected plastic moment. While those six beams included the two that actually were not able to achieve their expected plastic moment, and because this paper is written with the bolted moment connection in mind, the author felt that this shortcoming was not offset by the superior professional factor. Thus, model 3b was not investigated further.

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x + F_y Z_{x,web} \le F_y Z_x \quad (14)$$

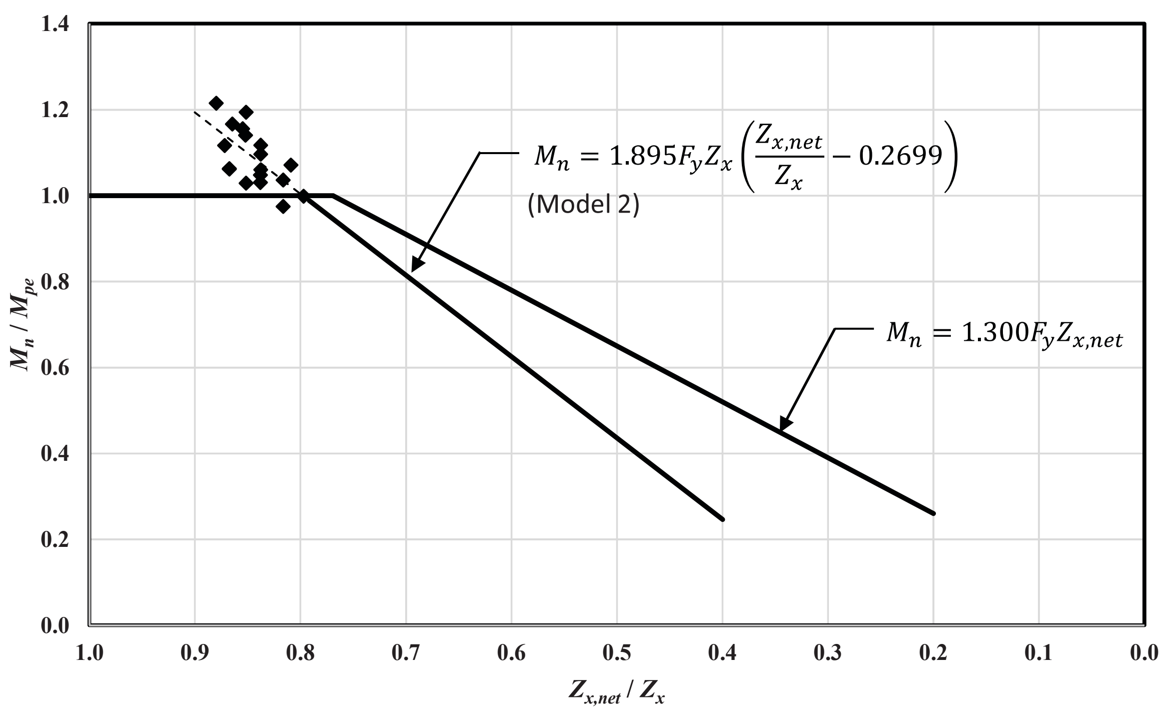

Fig. 7. Solution space for W-shapes with A992 steel for proposed model 2.

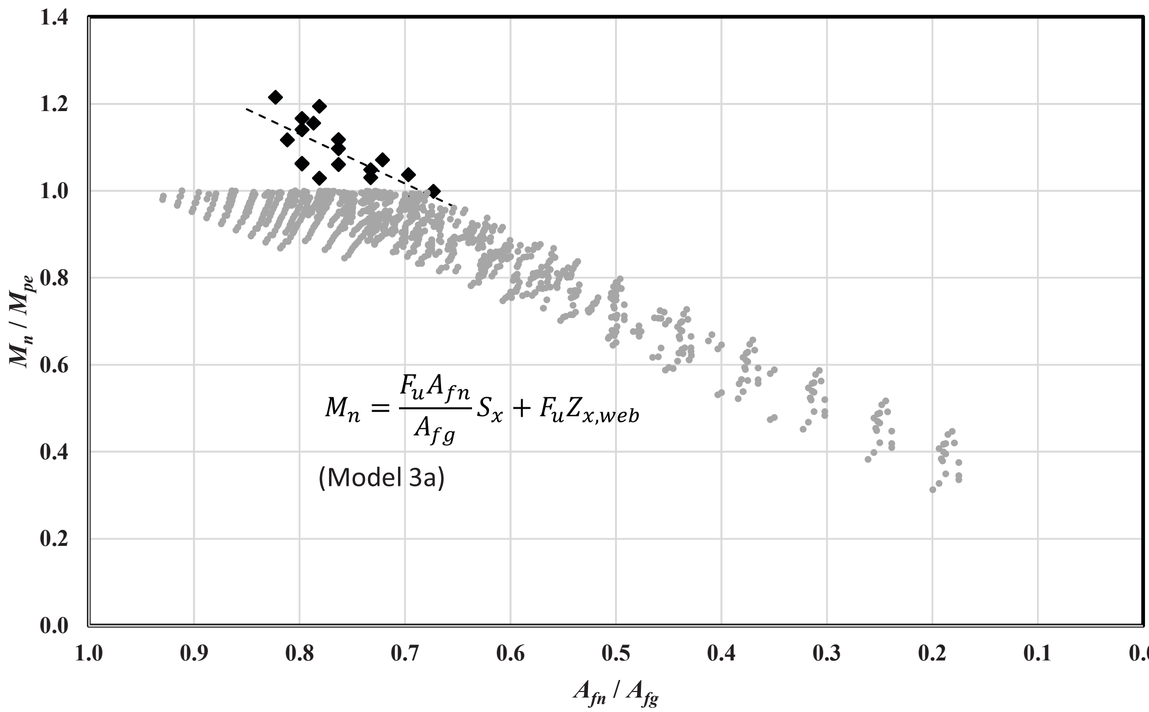

Fig. 8. Solution space for W-shapes with A992 steel for proposed model 3a.

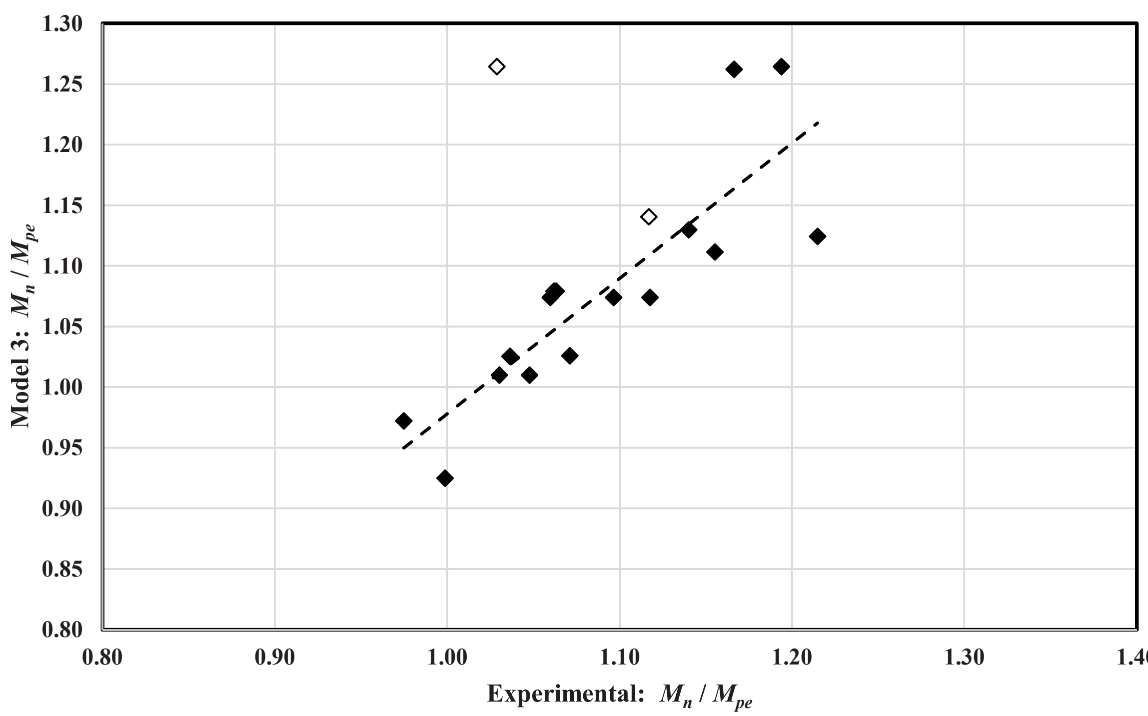

Fig. 9. Experimental correlation for proposed model 3a.

CONCLUSIONS

The experimental data summarized in this paper certainly show that beams with net sections that violate the design check in the 2016 AISC specification AISC 360-16 are, in many cases, able to reach the full plastic moment of their gross section before fracturing. Further, it is the opinion of the author that the provisions of Section F13 in AISC 360-16 are not rational because the strength model includes a discontinuity when examined as a function of the independent parameter, A_{fn}/A_{fg}. It is the author's opinion that the discontinuity at $A_{fn}/A_{fg} = Y_T F_y/F_u$ is excessively penal and unwarranted when compared to the experimental data.

Three models are proposed for the flexural strength of beams with holes in the tension flanges. Model 1 is based on a linear regression analysis of the experimental data with A_{fn}/A_{fg} as the independent parameter. Model 2 is also based on a linear regression of the experimental data but with $Z_{x,net}/Z_x$ as the independent variable. In both cases, the plastic moment of the beam is imposed as an upper limit of flexural strength.

Model 1 has the advantage of familiarity in that it, like the model in AISC 360-16, is a function of A_{fn}/A_{fg}. Model 1 also has the advantage of a slightly higher correlation constant of $R^2 = 0.5448$ compared to the $R^2 = 0.4219$ for model 2. Model 2, however, enjoys the advantage that it, in the opinion of the author, is more reflective of the mechanics associated with the failure mechanism that it is predicting because it is a function of Z_{net}, which is a parameter associated with flexure, as opposed to A_{fn}, which is a parameter associated with axial behavior. As such, model 2 inherently recognizes the contribution of the web in aiding the flanges in carrying moment and resisting a net-section fracture, whereas model 1 does not.

Model 3a enjoys the advantages of (1) having a rational basis, (2) being familiar to practicing engineers, (3) a strong correlation with experimental data with a professional factor of 1.116 and $R^2 = 0.7278$, and (4) being able to accurately predict which beam configurations will be able to reach their expected plastic moment. It is because of this that the author recommends model 3a for use in designing moment connections where components are bolted to the beam flanges.

It should be noted that all three proposals rely on extrapolation well beyond the limits of the existing experimental data. Prudence would demand that additional experimentation be conducted on beams at lower levels of A_{fn}/A_{fg} or $Z_{x,net}/Z_x$ to verify the applicability of the models before they are implemented. At a minimum, limits of applicability should be applied to the models based on the available data set.

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