Single-Plate Shear Connection Design to Meet Structural Integrity Requirements

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ABSTRACT

Specific prescriptive structural integrity provisions have been added to both model and local building codes since the collapse of the buildings at the World Trade Center site in 2001. The first building code to incorporate specific requirements was the 2008 New York City Building Code, which was followed quickly by the 2009 International Building Code. This paper demonstrates how properly designed single-plate shear connections comply with the structural integrity provisions of IBC 2009 and also make appropriate comparisons with NYC 2008.

Keywords: building code, structural integrity, single-plate shear connection.

Structural integrity has always been one of the goals for the structural engineer carrying out an engineering design and for the committees writing design standards. However, it has only been since the collapse of the buildings at the World Trade Center site that requirements with the stated purpose of addressing structural integrity have appeared in U.S. building codes. The first building code to incorporate specific requirements was the New York City Building Code (NYC, 2008) which was followed quickly by the International Building Code (ICC, 2009). Although the requirements of these two building codes are similar, there are some specifics that make their application different. Appendix A presents the provisions of the New York City code that are pertinent to this discussion, and Appendix B presents the appropriate International Building Code provisions. Appendix C summarizes the AISC *Specification* equations referenced in this paper.

This paper will demonstrate how properly designed single-plate shear connections comply with the structural integrity provisions of IBC 2009 and will also make appropriate comparisons with NYC 2008.

INTERNATIONAL BUILDING CODE REQUIREMENTS

Section 1614 of IBC 2009 provides structural integrity requirements that apply to high-rise buildings in occupancy categories III and IV. Simply stated, this means buildings with an occupied floor more than 75 ft above fire department vehicle access that represent a substantial hazard to human life in the event of failure or that are essential facilities. Thus, the number of buildings that these requirements apply to is limited.

Two types of member connections are addressed for steel frame structures, Section 1614.3.2.1 addresses column splices and 1614.3.2.2 addresses beam connections. This paper looks at one particular beam connection, the single-plate shear connection as shown in Figure 1. The provisions state that "all beams and girders shall have a minimum nominal axial tensile strength equal to the required vertical shear strength for allowable stress design (ASD) or two-thirds of the required shear strength for load and resistance factor design (LRFD) but not less than 10 kips." It also states that "For the purpose of this section, the shear force and axial tensile force need not be considered to act simultaneously." Using the terminology of AISC 360-05 *Specification for Structural Steel Buildings* (AISC, 2005a), these requirements can be stated as

$$T_n \geq V_a \qquad \text{(ASD)} \tag{1}$$

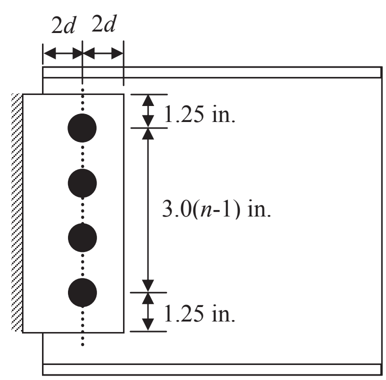

Fig. 1. Conventional configuration single-plate shear connection.

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$$T_n \geq \frac{2}{3}V_a \qquad \text{(LRFD)} \tag{2}$$

and

$$T_n \geq 10.0 \text{ kips} \tag{3}$$

where

T_n = nominal tensile strength
V_a = required shear strength for ASD
V_u = required shear strength for LRFD

Section 1614.3.2.2 provides an exception that will be addressed later.

Design of the connection for shear under normal loading requires that available strength be greater than or equal to required strength. This can be stated as

$$V_a \leq \frac{V_n}{\Omega} \qquad \text{(ASD)} \tag{4}$$

or

$$V_u \leq \phi V_n \qquad \text{(LRFD)} \tag{5}$$

where

V_n = nominal shear strength
Ω = safety factor for ASD
ϕ = resistance factor for LRFD

The right hand side of Equations 4 and 5 is the available shear strength for ASD and LRFD, respectively. Although the required shear strength must always be less than or equal to the available shear strength, it will be useful in this presentation to look at the limit where required shear strength is equal to the available shear strength. Thus, solving Equations 4 and 5 at the limit for V_n yields

$$\Omega V_a = V_n \qquad \text{(ASD)} \tag{6}$$

$$\frac{V_u}{\phi} = V_n \qquad \text{(LRFD)} \tag{7}$$

Setting Equations 6 and 7 equal and solving for V_a yields

$$V_a = \frac{V_u}{\phi\Omega} \tag{8}$$

Calibration of the ASD and LRFD provisions in AISC 360-05 is based on the fixed relationship $\phi\Omega = 1.5$. Thus, Equation 8 becomes

$$V_a = \frac{V_u}{1.5} = \frac{2}{3}V_u \tag{9}$$

If Equation 9 is substituted into Equation 1, the structural integrity requirements for ASD are seen to be exactly the same as those for LRFD, thus

$$T_n \geq \frac{2}{3}V_u \tag{10}$$

Equation 10 has the nominal tension strength on one side and the required shear strength on the other. If it is again recognized that the required strength cannot exceed the minimum design strength, $V_u = \phi V_n$ may be substituted into Equation 10 and the building code requirement restated as

$$1.5T_n \geq \phi V_n \tag{11}$$

Using Equation 11, all limit states may be checked without regard to any actual loading condition. If it can be shown that $1.5T_n$ is greater than the design shear strength for any particular limit state, Equation 11 is satisfied and the building code requirements have been satisfied.

In addition, it is not necessary to know the exact minimum available shear strength for every possible limit state if an upper-bound shear strength and a lower-bound tension strength are used. If it can be shown that $1.5T_n$ is greater than the available shear strength for any one shear limit state, it will be greater than the controlling minimum available shear strength. For many of the limit states to be checked in this paper, similar limit states will be considered for shear and tension because this will likely make the comparisons easiest. However, for some limit states, it will be easier to compare the results from different limit states.

SINGLE-PLATE SHEAR CONNECTION— CONVENTIONAL CONFIGURATION

Conventional configuration single-plate shear connections are defined in Part 10, page 10-101 of the 13th Edition *Steel Construction Manual* (AISC, 2005b). Design strength values are provided in Tables 10-9a and 10-9b.

The following eight limit states must be checked for a tension force applied to the single-plate shear connection shown in Figure 1:

- Bolt shear
- Weld
- Plate yield
- Plate rupture
- Bolt bearing and tearout on plate
- Bolt bearing and tearout on beam web
- Block shear on plate
- Block shear on beam web

1. Bolt Shear

Bolt shear strength is controlled by AISC *Specification* Eq. J3-1 and is independent of the direction of application of the load. Thus, the nominal shear strength of a bolt can be taken as r_n. With $\phi = 0.75$ and n being the number of bolts,

$$\phi V_n = 0.75nr_n \tag{12}$$

and

$$1.5T_n = 1.5nr_n \tag{13}$$

Thus, it can be seen that Equation 11 is always satisfied for bolt shear since

$$1.5nr_n > 0.75nr_n \tag{14}$$

2. Weld

The conventional configuration single-plate shear connection is required to have two fillet welds ⅝ the thickness of the plate, t_p. This ensures that the weld will not be a critical element when using 36-ksi or 50-ksi plate material. The design strength of the weld, based on AISC *Specification* Eq. J2-4 and $\phi = 0.75$, is

$$\phi R_n = 1.392Dl \tag{15}$$

where D is the weld size in sixteenths of an inch and l is the length in inches. For two ⅝-in. welds, $D = 20$ and the design shear strength is

$$\phi V_n = 1.392\left(20t_p\right)l = 27.8t_pl \tag{16}$$

For tension, two modifications will be made. First, the tension force is applied at 90° to the longitudinal axis of the weld so an increase of 1.5 in the strength is applicable according to AISC *Specification* Eq. J2-5. Second, the design strength of the weld as given in Equation 15 includes the resistance factor, $\phi = 0.75$, so this factor must be removed. Thus,

$$1.5T_n = 1.5\left(1.5\left[1.392\left(20t_p\right)l\right]/\,0.75\right) = 83.5t_pl \tag{17}$$

From Equations 16 and 17, it can be seen that Equation 11 is always satisfied for weld strength because

$$83.5t_pl > 27.8t_pl \tag{18}$$

3. Plate Yield

The limit state for yielding of the plate in shear is controlled by AISC *Specification* Eq. J4-3. Therefore, with $\phi = 1.00$,

$$\phi V_n = 1.00\left(0.6F_y\right)t_pl = 0.6F_yt_pl \tag{19}$$

For the limit state of yielding of the plate in tension, AISC *Specification* Eq. J4-1 controls. Thus,

$$1.5T_n = 1.5F_yt_pl \tag{20}$$

From Equations 19 and 20, it can be seen that Equation 11 is always satisfied for plate yielding because

$$1.5F_yt_pl > 0.6F_yt_pl \tag{21}$$

Table 1. Design Shear Strength as a Function of Bolt Diameter

Bolt diameter, d (in.)	Design shear strength, ϕV_n
¾	$1.35nt_pF_u$
⅝	$1.58nt_pF_u$
1	$1.80nt_pF_u$
1⅛	$2.03nt_pF_u$

4. Plate Rupture

The limit state for plate rupture in shear is given by AISC *Specification* Eq. J4-4. Therefore, with $\phi = 0.75$,

$$\phi V_n = 0.75\left(0.6F_ut_pl_n\right) = 0.45F_ut_pl_n \tag{22}$$

where l_n is the net shear area. Tensile rupture is given by AISC *Specification* Eq. J4-2. By definition, the net area of the plate for shear and tension are the same. Thus,

$$1.5T_n = 1.5F_ut_pl_n \tag{23}$$

From Equations 22 and 23, it can be seen that Equation 11 is always satisfied for plate rupture because

$$1.5F_ut_pl_n > 0.45F_ut_pl_n \tag{24}$$

5. Bolt Bearing and Tearout on Plate

Bearing and tearout are controlled by the provisions of AISC *Specification* Section J3.10. For shear, deformation at service load is a normal design consideration. The requirements stated in AISC *Specification* Eq. J3-6a apply when using standard or short slotted holes, both of which are permitted for conventional configuration single-plate shear connections. For the design shear strength, all bolts are assumed to be at their full bearing strength as given by the right side of AISC *Specification* Eq. J3-6a. This is an upper limit on the design shear strength, so its use will be conservative, regardless of the tearout strength dictated by the distance L_{ev}. Thus, with $\phi = 0.75$,

$$\phi V_n = n\left[0.75(2.4d)t_pF_u\right] = 1.8ndt_pF_u \tag{25}$$

where d is the bolt diameter.

With strength seen as a function of bolt diameter, Table 1 shows the solution of Equation 25 for four different bolt diameters used in the conventional configuration single-plate shear connection.

Table 2. Bolt Tearout Length and Tensile Strength for SSL Holes		
Bolt diameter, d (in.)	Tearout length, L_c (in.)	Tensile strength, $1.5T_n$
¾	1.000	$2.25nt_pF_u$
⅝	1.188	$2.67nt_pF_u$
1	1.344	$3.02nt_pF_u$
1⅛	1.500	$3.38nt_pF_u$

Table 3. Bolt Tearout Length and Tensile Strength for Standard Holes with L_{eh} = 1.5d		
Bolt diameter, d (in.)	Tearout length, L_c (in.)	Tensile strength, $1.5T_n$
¾	0.719	$1.62nt_pF_u$
⅝	0.844	$1.90nt_pF_u$
1	0.969	$2.18nt_pF_u$
1⅛	1.094	$2.46nt_pF_u$

For consideration of structural integrity, deformation does not need to be a design consideration, as provided in Section B3.2 of the final draft of the 2010 AISC *Specification*. Thus, the nominal tensile strength is controlled by 2005 AISC *Specification* Eq. J3-6b. The conventional configuration, *Manual* page 10-102, requires that the horizontal distance from the bolt hole center to the loaded edge, L_{eh}, be at least 2d. Thus, tearout would control over bearing and

$$1.5T_n = 1.5\left[n\left(1.5L_c t_p F_u\right)\right] = 2.25nL_c t_p F_u \tag{26}$$

With $L_{eh} = 2d$ as stated earlier, the clear distance for tearout with standard holes is $L_c = 2d - 0.5(d + \frac{1}{16}) = 1.5d - 0.0313$. For short slotted holes, hole dimensions are given in AISC *Specification* Table J3.3. Table 2 shows the clear distance for tearout, L_c, as a function of bolt diameter for short slotted holes and the corresponding tensile strength, $1.5T_n$. Because the tearout strength will be greater for standard holes than for short slotted holes, short slotted holes will provide a conservative comparison.

A comparison between Tables 1 and 2 shows that for each bolt size and either standard holes or short slotted holes, Equation 11 is satisfied for bolt bearing and tearout of the plate. If deformation were to be a design consideration for tension, the tensile strength in Table 2 would be reduced by 0.8 and the tensile strength would still be greater than the shear strength.

6. Bolt Bearing and Tearout on Beam Web

The assessment of bolt bearing and tearout on the beam web follows the same procedure as for the plate. For shear, all bolts are assumed to be at their full bearing strength. This will be used even if the beam is coped because it gives an upper bound on the shear strength. The tearout distance for the beam web is taken as 2d, as it was for the plate. Thus, the values in Tables 1 and 2 may be used for the beam web by simply replacing t_p with t_b. Thus, Equation 11 is satisfied for bolt bearing and tearout of the beam web for standard and short slotted holes.

Table 4. Net Length for Determination of Ultimate Shear Strength	
Bolt diameter, d (in.)	Net length, l_{n2} (in.)
¾	1.06
⅝	1.25
1	1.44
1⅛	1.63

If only standard holes are used in the beam web, the tearout distance can be reduced to 1.5d and Equation 11 is still satisfied as seen by comparing the results in Table 3 with those in Table 1.

7. Block Shear on Plate

Block shear strength is controlled by the provisions of AISC *Specification* Section J4.3 and specifically Eq. J4-5. For the design shear strength, it will be assumed that shear rupture in combination with tension rupture will control. This will give an upper-bound strength if shear yielding happened to control. In addition, for one line of bolts $U_{bs} = 1.0$. Figure 2(a) shows the single-plate with the block shear failure planes for a shear loading noted. AISC *Specification* Eq. J4-5, considering only rupture and $\phi = 0.75$, can be written as

$$\phi V_n = 0.75\left(F_u l_{n2} t_p + 0.6F_u l_{n1} t_p + 0.6F_u l_{n3} t_p\right) \tag{27}$$

For the geometry of the conventional configuration single-plate shear connection, with a bolt spacing of 3.0 in., $l_1 = 3.0(n - 1)$ in., $l_2 = 2d$ in., $l_3 = 1.25$ in. For standard holes and short slotted holes, the net length for line 2 will always be greater than the net length for line 3. Thus, it will be conservative to let $l_{n3} = l_{n2}$. Therefore

$$\phi V_n = \left(0.45l_{n1} + 1.2l_{n2}\right)F_u t_p \tag{28}$$

Table 4 shows the net length, l_{n2}, for standard holes to be used in Equation 28.

Table 5. Determination of Controlling Limit State for Tension				
Bolt diameter, d (in.)	Gross length, $l_{g2} = 2d$ (in.)	Net length, l_{n2} (in.)	$\frac{F_y}{F_u} l_{g2}$ (in.)	
			A36	A572 Gr. 50
¾	1.50	0.969	0.932	1.15
⅝	1.75	1.16	1.09	1.35
1	2.00	1.31	1.24	1.54
1⅛	2.25	1.47	1.40	1.73

The block shear planes for the tension force are shown in Figure 2(b). With the shear rupture terms controlling, AISC *Specification* Eq. J4-5 becomes

$$\begin{aligned} 1.5T_n &= 1.5\left[F_u l_{n1} t_p + 2\left(0.6F_u l_{n2} t_p\right)\right] \\ &= \left[1.5l_{n1} + 1.2\left(1.5l_{n2}\right)\right] F_u t_p \end{aligned} \tag{29}$$

and with shear yielding terms controlling, Eq. J4-5 becomes

$$\begin{aligned} 1.5T_n &= 1.5\left(F_u l_{n1} t_p + 1.2F_y l_{g2} t_p\right) \\ &= \left[1.5l_{n1} + 1.2\left(1.5\frac{F_y}{F_u} l_{g2}\right)\right] F_u t_p \end{aligned} \tag{30}$$

A comparison of Equations 29 and 30 shows that the smaller of l_{n2} or $(F_y/F_u)l_{g2}$ will give the lower bound tensile strength. Table 5 shows the lengths needed to determine which limit state controls for the four considered bolt diameters using short slotted holes with $l_{n2} = 2d - 0.5(SSL + 1/16)$.

For these four bolt diameters and A36 steel, $(F_y/F_u)l_{g2}$ is smaller than l_{n2}, so yielding will control and the comparisons required to satisfy Equation 11 will be between Equations 28 and 30. For A572 Gr. 50 steel, l_{n2} is smaller than $(F_y/F_u)l_{g2}$,

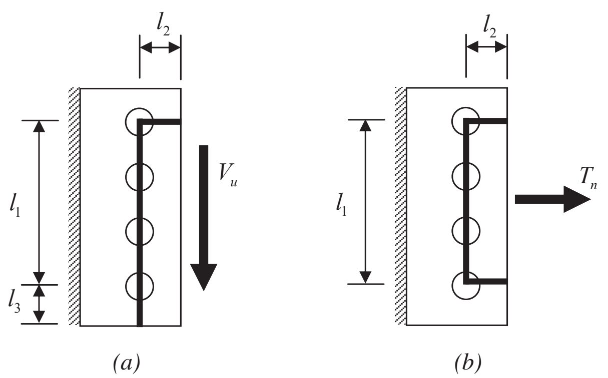

Figure 2. Geometry for block shear on plate.

Table 6. Second Term in Equations 29 and 30				
Bolt diameter, d (in.)	A36		A570 Gr. 50	
	$\frac{F_y}{F_u} l_{g2}$ (in.)	$1.5\left(\frac{F_y}{F_u} l_{g2}\right)$ (in.)	Net length, l_{n2} (in.)	$1.5l_{n2}$ (in.)
¾	0.932	1.40	0.969	1.45
⅝	1.09	1.64	1.16	1.74
1	1.24	1.86	1.31	1.97
1⅛	1.40	2.10	1.47	2.21

Table 7. Bearing and Tearout on Beam Web	
Bolt diameter, d (in.)	Design shear strength, ϕV_n
¾	$1.35nt_bF_u$
⅝	$1.58nt_bF_u$
1	$1.80nt_bF_u$
1⅛	$2.03nt_bF_u$

so rupture controls and the comparisons required to satisfy Equation 11 will be between Equations 28 and 29.

The comparison between shear and tension will be made in two parts. First, because the definition of l_{n1} is the same for shear and tension, the contribution for length l_{n1} will always be greater in resisting tension. That leaves the comparison to be based on the second term of each equation. The definition of l_{n2} is different for shear and tension because standard holes are used to get the maximum for shear and short slotted holes are used to get the minimum for tension. Table 6 gives the appropriate second term for tension strength to be compared to the second term for shear strength.

A comparison between Tables 4 and 6 shows that Equation 11 is satisfied for block shear in the plate for the four bolt diameters considered, for standard or short slotted holes, and for both A36 and A572 Gr. 50 steels.

8. Block Shear on Beam Web

For the first seven limit states considered in this paper for the single-plate shear connection, the same limit state was assessed for design shear strength and nominal tension strength. Although it is not imperative that it be done this way, it generally makes the comparisons easier because the same parameters are being used for each strength. For the case of block shear on the beam web, it will be easier to consider block shear for the tension force while considering bearing and tearout for the shear force. Table 7, taken from

Table 8a. Block Shear Strength for Tension Force—Tension Component		
Bolt diameter, d (in.)	$l_{n1} = (n-1)(3.0 - d - 1/8)$ (in.)	T_n (tension)
3/4	$2.125(n-1)$	$2.125(n-1)t_bF_u$
5/8	$2.00(n-1)$	$2.00(n-1)t_bF_u$
1	$1.875(n-1)$	$1.875(n-1)t_bF_u$
1 1/8	$1.75(n-1)$	$1.75(n-1)t_bF_u$

Table 8b. Block Shear Strength for Tension Force—Shear Component		
Bolt diameter, d (in.)	$l_{n2} = 2d - 0.5(SSL + 1/16)$. (in.)	T_n (shear)
3/4	0.969	$1.16t_bF_u$
5/8	1.16	$1.39t_bF_u$
1	1.31	$1.57t_bF_u$
1 1/8	1.47	$1.76t_bF_u$

Table 1 with the beam web thickness replacing the plate thickness (limit state 6), gives the shear strength that will be used to assess the beam web for block shear in tension.

Figure 3 shows the beam web with the block shear planes defined for resisting the tension force. Because the beam will be A992 steel, the controlling limit state for the shear planes will be rupture and Equation 29, modified to represent the beam web rather than the plate, is

$$1.5T_n = \left[1.5l_{n1} + 1.2\left(1.5l_{n2}\right)\right]F_ut_b \tag{31}$$

The net length for the tension rupture term, l_{n1}, is a function of the number of bolts and bolt spacing. Using a standard spacing of 3.0 in. yields

$$l_{n1} = (n-1)\left(3.0 - d - 1/8\right) \tag{32}$$

The net length for the shear rupture term, l_{n2}, with $L_{eh} = 2d$ and short slotted holes can be determined from

$$l_{n2} = 2d - 0.5\left(SSL + 1/16\right) \tag{33}$$

where *SSL* is the slot length as defined in AISC *Specification* Table J3.3.

Tables 8a, 8b and 8c give the net lengths and the nominal tensile strength as a function of the number of bolts, *n*.

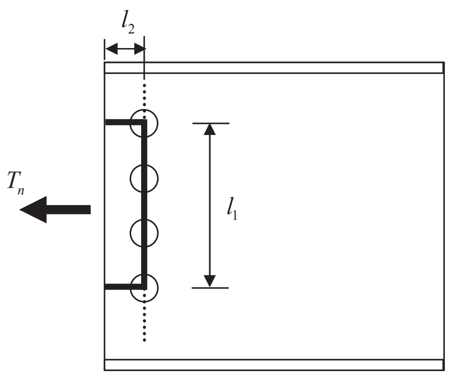

Figure 3. Geometry for block shear on beam web.

Table 8c. Block Shear Strength for Tension Force Total	
Bolt diameter, d (in.)	$1.5T_n$
3/4	$(3.19(n-1) + 1.74)t_bF_u$
5/8	$(3.00(n-1) + 2.09)t_bF_u$
1	$(2.81(n-1) + 2.36)t_bF_u$
1 1/8	$(2.63(n-1) + 2.64)t_bF_u$

In order to make the necessary comparisons called for by Equation 11, an assessment must be made for number of bolts from 2 through 12, as provided in the conventional configuration definition. As an example, five 1.00-in.-diameter bolts will be checked. From Table 7,

$$\phi V_n = 1.80nt_bF_u = 9.00t_bF_u \tag{34}$$

and from Table 8c,

$$1.5T_n = \left[2.81(n-1) + 2.36\right]t_bF_u = 13.6t_bF_u \tag{35}$$

Similarly, in all cases, Equation 11 is satisfied for the block shear pattern shown in Figure 3.

If the beam is coped, a comparison can be made using only one shear plane for the tension strength. That is accomplished by considering only one half of the value given in Table 8b. Equation 11 is satisfied for all combinations of bolts considered in this way for the coped beam except for two bolts with 1.125-in. diameter. However, for this one condition, a very small contribution from the tension plane above the top hole will add sufficient strength to satisfy Equation 11. Thus, block shear on the beam web is satisfied for all conditions considered.

Summary for IBC

It has been shown that for every applicable limit state required to resist the structural integrity tensile force, the single-plate shear connection has sufficient strength. These connections have been shown to satisfy the basic

Table 9. Minimum Edge Distance for NYC 2008					
	Standard Holes			Short Slotted Holes	
Bolt diameter, *d* (in.)	L_{eh} (in.)	L_{eh}/d	SSL	L_{eh} (in.)	L_{eh}/d
¾	1.606	2.14	1.0000	1.700	2.27
⅝	1.869	2.14	1.1250	1.963	2.24
1	2.131	2.13	1.3125	2.256	2.26
1⅛	2.394	2.13	1.5000	2.550	2.27

requirement without relying on the exception provided in IBC 2009 Section 1614.3.2.2. Thus, no further calculations will be required in the design phase for conventional configuration single-plate shear connections to show compliance with the IBC structural integrity provisions.

NYC BUILDING CODE REQUIREMENTS

The requirements of the New York City Building Code (NYC, 2008) are given in Appendix A. A review of these requirements shows that all bolted connections must have two bolts and that in connections that are part of the lateral load resisting system, bearing type connections are acceptable but that the bolts must be pretensioned. For beam end connections that are not part of the lateral load resisting system, "all beams and girders shall have a design axial tension strength equal to the larger of the provided vertical shear strength of the connection at either end but not less than 10 kips." This edition of the NYC Building Code permits design by ASD according to the 1989 ASD *Specification* or LRFD according to the 1999 LRFD *Specification*. Unfortunately, the terminology used in the building code is not consistent with the specifications referenced. Using AISC 360-05, these requirements, in the form of Equations 1 and 2, can be stated as

$$\frac{T_n}{\Omega} \geq \frac{V_n}{\Omega} \qquad \text{(ASD)} \tag{36}$$

$$\phi T_n \geq \phi V_n \qquad \text{(LRFD)} \tag{37}$$

It should be noted that the safety factors, Ω, and the resistance factors, ϕ, shown are not necessarily equal on each side of Equations 36 and 37 because they will vary depending on the limit state being considered. For single-plate shear connections, the only limit state to be evaluated for resisting the structural integrity tension force, as given in Section 2213.2.3.1, is bolt bearing and tearout without deformation being a design consideration. Bolt shear is not mentioned in the NYC Building Code because the available strength of the bolts in shear is the same when they are resisting beam shear or beam tension.

Thus, the nominal axial tension strength is to be determined according to AISC *Specification* Eq. J3-6b, where deformation is not a design consideration. The right side of Equation 37 is the same as the right side of Equation 11. Thus, an approach similar to that used to show compliance with the requirements of IBC 2009 can be used here. The bearing and tearout limit states were addressed in cases 5 and 6 for IBC 2009. From Equation 26, without the 1.5 increase,

$$T_n = \left[n\left(1.5L_c t_p F_u\right)\right] = 1.5nL_c t_p F_u \tag{38}$$

In order to satisfy Equation 37, using Equation 25 for the provided shear strength, Equation 38 for the nominal tensile strength, and ϕ = 0.75,

$$0.75\left(1.5nL_c t_p F_u\right) \geq 1.8nt_p dF_u \tag{39}$$

This results in the requirement that

$$L_c \geq 1.6d \tag{40}$$

For standard holes,

$$L_c = L_{eh} - \frac{1}{2}\left(d + \frac{1}{16}\right) \tag{41}$$

Using Equations 40 and 41, the minimum edge distance for standard holes is

$$L_{eh} = 2.1d + 0.0313 \tag{42}$$

For short slotted holes,

$$L_c = L_{eh} - \frac{1}{2}(SSL) \tag{43}$$

Using Equations 42 and 43, the minimum edge distance for short slotted holes is

$$L_{eh} = 1.6d + \frac{SSL}{2} \tag{44}$$

Table 9 shows the minimum edge distances to permit the conclusion that Equation 39 would always be satisfied.

With a slight increase in the conventional configuration edge distance from 2d to 2.3d in both the plate and the beam web, single-plate shear connections will always be adequate to resist the structural integrity tension force. However, that is not the best way to show compliance with the NYC Building Code requirements. Because there is only one limit state to be considered in resisting the structural integrity tension force, it will be a relatively simple task to determine the design strength, ϕT_n, of each conventional configuration single-plate shear connection given in the 13th Edition *Manual*. This strength is given in Appendix D of this paper—Table D-1 for standard holes and Table D-2 for short slotted holes, based on Equation 38 and $\phi = 0.75$. A comparison of the data in Appendix D with that in *Manual* Table 10-9 shows that in all cases the conventional configuration single-plate shear connection has sufficient design strength in tension to satisfy the NYC Building Code structural integrity provisions.

For any case where the beam web controls the shear strength of the connection, the tension strength can be determined using the results shown in Appendix D, modified proportionally for the appropriate beam web thickness. This would then be compared to the shear strength of the connection to confirm that Equation 37 is satisfied.

There is one additional issue that must be addressed with regard to the NYC Building Code provisions. That is the requirement that the structural integrity tension force for beams that are not symmetrically loaded be determined using the largest provided shear strength on either end of the member. There are an infinite number of combinations possible for the required shear strength at a beam end. The simplest way to satisfy this requirement, if the beam is not symmetrical, is to the make the connections symmetrical. That way, the connection at each end will be guaranteed to meet the structural integrity requirements.

CONCLUSIONS

Conventional configuration single-plate shear connections, as provided in Tables 10-9a and 10-9b of the 13th Edition *Steel Construction Manual*, were evaluated for compliance with the structural integrity provisions of the 2009 International Building Code (ICC, 2009) and the 2008 New York City Building Code (NYC, 2008). It was shown that these connections, when designed to resist shear according to the provisions of ANSI/AISC 360-05, will satisfy the structural integrity provisions of both building codes without any modification to their design. Clearly for conventional configuration single-plate shear connections, structural integrity, as defined by these two building codes, has already been assured by the provisions of the AISC *Specification* (AISC, 2005a) and the *Steel Construction Manual* (AISC, 2005b), without the need for the special requirements of building codes.

REFERENCES

AISC (2005a), *Specification for Structural Steel Buildings*, AISC/ANSI 360, American Institute of Steel Construction, Chicago, IL.

AISC (2005b), *Steel Construction Manual*, 13th Edition, American Institute of Steel Construction, Chicago, IL.

ICC (2009), *International Building Code*, International Code Council, Country Club Hills, IL.

NYC (2008), 2008 New York City *Building Code*, NYCBC, New York, NY.

APPENDIX A

2008 New York City Building Code Structural Integrity Provisions

SECTION BC 2213
STRUCTURAL INTEGRITY REQUIREMENTS

2213.2 Continuity and ties. The following requirements shall be met:

1. All bolted connections shall have at least two bolts.
2. Bolted connections of all columns, beams, braces and other structural elements that are part of the lateral load resisting system shall be designed as bearing-type connections with pretensioned bolts or as slip critical connections.
3. End connections of all beams and girders shall have a design axial tension strength equal to the larger of the provided vertical shear strength of the connections at either end, but not less than 10 kips (45 kN). For the design of the connections, the shear force and axial tensile force need not be considered to act simultaneously. For the purpose of this provision, a connection shall be considered compliant if it meets the following requirements:

   3.1 For single plate shear connections, the nominal axial tension strength shall be determined for the limit state of bolt bearing, where deformation is not a consideration, on the plate and beam web.

   3.2 For single angle and double angle shear connections, the nominal tension strength shall be determined for the limit state of bolt bearing, where deformation is not a consideration, on the angles and beam web and for tension yielding on the gross area of the angles.

   3.3 All other connections shall be designed for the required tension force in accordance with either AISC-LRFD, AISC 335, or AISC-HSS.

For the purpose of meeting this integrity provision only, bolts in connections with short-slotted holes parallel to the direction of the tension force are permitted. For the purpose of checking bearing, these bolts shall be assumed to be located at the end of the slots.

APPENDIX B

2009 International Building Code Structural Integrity Provisions

SECTION 1614
STRUCTURAL INTEGRITY

1614.1 General. Buildings classified as high-rise buildings in accordance with Section 403 and assigned to *Occupancy Category* III or IV shall comply with the requirements of this section. Frame structures shall comply with the requirements of Section 1614.3. Bearing wall structures shall comply with the requirements of Section 1614.4.

1614.3.2 Structural steel, open web steel joist or joist girder, or composite steel and concrete frame structures. Frame structures constructed with a structural steel frame or a frame composed of open web steel joists, joist girders with or without other structural steel elements or a frame composed of steel or composite steel joists and reinforced concrete elements shall conform to the requirements of this section.

1614.3.2.1 Columns. Each column splice shall have the minimum design strength in tension to transfer the design dead and live load tributary to the column between the splice and the splice or base immediately below.

1614.3.2.2 Beams. End connections of all beams and girders shall have a minimum nominal axial tensile strength equal to the required vertical shear strength for *allowable stress design* (ASD) or two-thirds of the required shear strength for *load and resistance factor design* (LRFD) but not less than 10 kips (45 kN). For the purpose of this section, the shear force and the axial tensile force need not be considered to act simultaneously.

Exception: Where beams, girders, open web joist and joist girders support a concrete slab or concrete slab on metal deck that is attached to the beam or girder with not less than 3/8-inch-diameter (9.5 mm) headed shear studs, at spacing of not more that 12 inches (305 mm) on center, averaged over the length of the member, or other attachment having equivalent shear strength, and the slab contains continuous distributed reinforcement in each of two orthogonal directions with an area not less than 0.0015 times the concrete area, the nominal axial tension strength of the end connection shall be permitted to be taken as half the required vertical shear strength for ASD or one-third of the required shear strength for LRFD, but not less than 10 kips (45 kN).

APPENDIX C

REFERENCED AISC 360-05 EQUATIONS

The following equations from the 2005 AISC *Specification for Structural Steel Buildings* have been referenced in this paper.

J2. Welds

$$R_n = F_w A_w \tag{J2-4}$$

$$F_w = 0.60 F_{EXX}\left(1.0 + 0.50 \sin^{1.5}\theta\right) \tag{J2-5}$$

$$\phi = 0.75$$

J3. Bolts and Threaded Parts

$$R_n = F_n A_b \tag{J3-1}$$

$$R_n = 1.2 L_c t F_u \le 2.4 d t F_u \tag{J3-6a}$$

$$R_n = 1.5 L_c t F_u \le 3.0 d t F_u \tag{J3-6b}$$

$$\phi = 0.75$$

J4. Affected Elements of Members and Connecting Elements

$$R_n = F_y A_g \tag{J4-1}$$

$$\phi = 0.90$$

$$R_n = F_u A_e \tag{J4-2}$$

$$\phi = 0.75$$

$$R_n = 0.60 F_y A_g \tag{J4-3}$$

$$\phi = 1.0$$

$$R_n = 0.60 F_u A_{nv} \tag{J4-4}$$

$$\phi = 0.75$$

$$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} \tag{J4-5}$$

$$\phi = 0.75$$

APPENDIX D

TENSION STRENGTH FOR USE WITH THE 2008 NEW YORK CITY BUILDING CODE

Table D-1. Design Tensile Strength for Plates with Standard Holes

F_y = 36 ksi
Bolt Diameter, ¾ in.
Standard Holes, clear distance, L_c = 1.0938 in.

n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	214	268	321	375	428	482
11	196	245	294	344	393	442
10	178	223	268	312	357	401
9	161	201	241	281	321	361
8	143	178	214	250	286	321
7	125	156	187	219	250	281
6	107	134	161	187	214	241
5	89.2	112	134	156	178	201
4	71.4	89.2	107	125	143	161
3	53.5	66.9	80.3	93.7	107	120
2	35.7	44.6	53.5	62.4	71.4	80.3

F_y = 36 ksi
Bolt Diameter, ⅞-in.
Standard Holes, clear distance, L_c = 1.2813 in.

n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	251	314	376	439	502	564
11	230	287	345	402	460	517
10	209	261	314	366	418	470
9	188	235	282	329	376	423
8	167	209	251	293	334	376
7	146	183	220	256	293	329
6	125	157	188	220	251	282
5	105	131	157	183	209	235
4	83.6	105	125	146	167	188
3	62.7	78.4	94.1	110	125	141
2	41.8	52.3	62.7	73.2	83.6	94.1

Table D-1. Design Tensile Strength for Plates with Standard Holes (cont.)

$F_y = 36$ ksi
Bolt Diameter, 1 in.
Standard Holes, clear distance, $L_c = 1.4688$ in.

n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	288	359	431	503	575	647
11	264	329	395	461	527	593
10	240	300	359	419	479	539
9	216	270	323	377	431	485
8	192	240	288	335	383	431
7	168	210	252	294	335	377
6	144	180	216	252	288	323
5	120	150	180	210	240	270
4	95.8	120	144	168	192	216
3	71.9	89.8	108	126	144	162
2	47.9	59.9	71.9	83.9	95.8	108

$F_y = 36$ ksi
Bolt Diameter, 1⅛ in.
Standard Holes, clear distance, $L_c = 1.6563$ in.

n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	324	405	486	567	648	730
11	297	372	446	520	594	669
10	270	338	405	473	540	608
9	243	304	365	426	486	547
8	216	270	324	378	432	486
7	189	236	284	331	378	426
6	162	203	243	284	324	365
5	135	169	203	236	270	304
4	108	135	162	189	216	243
3	81.1	101	122	142	162	182
2	54.0	67.5	81.1	94.6	108	122

Table D-1. Design Tensile Strength for Plates with Standard Holes (cont.)

$F_y = 50$ ksi
Bolt Diameter, ¾ in.
Standard Holes, clear distance, $L_c = 1.0938$ in.

n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	240	300	360	420	480	540
11	220	275	330	385	440	495
10	200	250	300	350	400	450
9	180	225	270	315	360	405
8	160	200	240	280	320	360
7	140	175	210	245	280	315
6	120	150	180	210	240	270
5	100	125	150	175	200	225
4	80.0	100	120	140	160	180
3	60.0	75.0	90.0	105	120	135
2	40.0	50.0	60.0	70.0	80.0	90.0

$F_y = 50$ ksi
Bolt Diameter, ⅞ in.
Standard Holes, clear distance, $L_c = 1.2813$ in.

n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	281	351	422	492	562	632
11	258	322	387	451	515	580
10	234	293	351	410	469	527
9	211	264	316	369	422	474
8	187	234	281	328	375	422
7	164	205	246	287	328	369
6	141	176	211	246	281	316
5	117	146	176	205	234	264
4	93.7	117	141	164	187	211
3	70.3	87.8	105	123	141	158
2	46.8	58.6	70.3	82.0	93.7	105

Table D-1. Design Tensile Strength for Plates with Standard Holes (cont.)						
F_y = 50 ksi Bolt Diameter, 1 in. Standard Holes, clear distance, L_c = 1.4688 in.						
n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	322	403	483	564	644	725
11	295	369	443	517	591	665
10	269	336	403	470	537	604
9	242	302	363	423	483	544
8	215	269	322	376	430	483
7	188	235	282	329	376	423
6	161	201	242	282	322	363
5	134	168	201	235	269	302
4	107	134	161	188	215	242
3	80.6	101	121	141	161	181
2	53.7	67.1	80.6	94.0	107	121
F_y = 50 ksi Bolt Diameter, 1⅛ in. Standard Holes, clear distance, L_c = 1.6563 in.						
n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	363	454	545	636	727	818
11	333	416	500	583	666	749
10	303	379	454	530	606	681
9	273	341	409	477	545	613
8	242	303	363	424	485	545
7	212	265	318	371	424	477
6	182	227	273	318	363	409
5	151	189	227	265	303	341
4	121	151	182	212	242	273
3	90.8	114	136	159	182	204
2	60.6	75.7	90.8	106	121	136

Table D-2. Design Tensile Strength for Plates with Short Slotted Holes						
F_y = 36 ksi Bolt Diameter, ¾ in. Short Slotted Holes, clear distance, L_c = 1.00 in.						
n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	196	245	294	343	392	440
11	179	224	269	314	359	404
10	163	204	245	286	326	367
9	147	184	220	257	294	330
8	131	163	196	228	261	294
7	114	143	171	200	228	257
6	97.9	122	147	171	196	220
5	81.6	102	122	143	163	184
4	65.3	81.6	97.9	114	131	147
3	48.9	61.2	73.4	85.6	97.9	110
2	32.6	40.8	48.9	57.1	65.3	73.4
F_y = 36 ksi Bolt Diameter, ⅞ in. Short Slotted Holes, clear distance, L_c = 1.188 in.						
n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	233	291	349	407	465	523
11	213	267	320	373	426	480
10	194	242	291	339	388	436
9	174	218	262	305	349	392
8	155	194	233	271	310	349
7	136	170	204	237	271	305
6	116	145	174	204	233	262
5	96.9	121	145	170	194	218
4	77.5	96.9	116	136	155	174
3	58.1	72.7	87.2	102	116	131
2	38.8	48.4	58.1	67.8	77.5	87.2

Table D-2. Design Tensile Strength for Plates with Short Slotted Holes (cont.)

F_y = 36 ksi
Bolt Diameter, 1 in.
Short Slotted Holes, clear distance, L_c = 1.344 in.

n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	263	329	395	460	526	592
11	241	302	362	422	482	543
10	219	274	329	384	439	493
9	197	247	296	345	395	444
8	175	219	263	307	351	395
7	154	192	230	269	307	345
6	132	164	197	230	263	296
5	110	137	164	192	219	247
4	87.7	110	132	154	175	197
3	65.8	82.2	98.7	115	132	148
2	43.8	54.8	65.8	76.7	87.7	98.7

F_y = 36 ksi
Bolt Diameter, 1⅛ in.
Short Slotted Holes, clear distance, L_c = 1.50 in.

n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	294	367	440	514	587	661
11	269	336	404	471	538	606
10	245	306	367	428	489	551
9	220	275	330	385	440	496
8	196	245	294	343	392	440
7	171	214	257	300	343	385
6	147	184	220	257	294	330
5	122	153	184	214	245	275
4	97.9	122	147	171	196	220
3	73.4	91.8	110	129	147	165
2	48.9	61.2	73.4	85.6	97.9	110

Table D-2. Design Tensile Strength for Plates with Short Slotted Holes (cont.)

Fy = 50 ksi
Bolt Diameter, ¾ in.
Short Slotted Holes, clear distance, L_c = 1.00 in.

n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	219	274	329	384	439	494
11	201	251	302	352	402	453
10	183	229	274	320	366	411
9	165	206	247	288	329	370
8	146	183	219	256	293	329
7	128	160	192	224	256	288
6	110	137	165	192	219	247
5	91.4	114	137	160	183	206
4	73.1	91.4	110	128	146	165
3	54.8	68.6	82.3	96.0	110	123
2	36.6	45.7	54.8	64.0	73.1	82.3

F_y = 50 ksi
Bolt Diameter, ⅞ in.
Short Slotted Holes, clear distance, L_c = 1.188 in.

n	Plate Thickness, in.					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	261	326	391	456	521	586
11	239	299	358	418	478	538
10	217	272	326	380	434	489
9	196	244	293	342	391	440
8	174	217	261	304	348	391
7	152	190	228	266	304	342
6	130	163	196	228	261	293
5	109	136	163	190	217	244
4	86.9	109	130	152	174	196
3	65.2	81.4	97.7	114	130	147
2	43.4	54.3	65.2	76.0	86.9	97.7

Table D-2. Design Tensile Strength for Plates with Short Slotted Holes (cont.)						
F_y = 50 ksi Bolt Diameter, 1 in. Short Slotted Holes, clear distance, L_c = 1.344 in.						
n	**Plate Thickness, in.**					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	295	369	442	516	590	663
11	270	338	405	473	541	608
10	246	307	369	430	491	553
9	221	276	332	387	442	498
8	197	246	295	344	393	442
7	172	215	258	301	344	387
6	147	184	221	258	295	332
5	123	154	184	215	246	276
4	98.3	123	147	172	197	221
3	73.7	92.1	111	129	147	166
2	49.1	61.4	73.7	86.0	98.3	111
F_y = 50 ksi Bolt Diameter, 1⅛ in. Short Slotted Holes, clear distance, L_c = 1.50 in.						
n	**Plate Thickness, in.**					
	¼	5⁄16	⅜	7⁄16	½	9⁄16
12	329	411	494	576	658	740
11	302	377	453	528	603	679
10	274	343	411	480	548	617
9	247	309	370	432	494	555
8	219	274	329	384	439	494
7	192	240	288	336	384	432
6	165	206	247	288	329	370
5	137	171	206	240	274	309
4	110	137	165	192	219	247
3	82.3	103	123	144	165	185
2	54.8	68.6	82.3	96.0	110	123