A FORMULATED APPROACH TO CISC SHEAR CONNECTION RESISTANCE AND FLEXIBILITY DESIGN

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ABSTRACT

The design of simple shear connections has been extensively studied by several researchers. The design requirements for ductility and strength of these connections have been established through both experimental and theoretical approaches. However, the current Canadian design guidelines such as the Handbook of Steel Construction published by the Canadian Institute of Steel Construction (CISC) do not provide a formulated approach for the design of shear connections. In addition, the CISC design approach is somewhat outdated when it comes to providing adequate rotational ductility. In this paper, a formulated approach for the design of bolted double angle shear connections considering both strength and ductility is provided. The proposed design approach is discussed and compared with the current design approach outlined by the American Institute of Steel Construction (AISC). The proposed procedure is explained in detail through a design example.

Keywords: Shear, Connection, Strength, Rotation, Ductility

SYMBOLS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tr>
<td>A_b</td>
<td>Bolt Area</td>
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<tr>
<td>A_g</td>
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<td>A_gv</td>
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<td>Bolt Diameter</td>
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<td>Bolt Tensile Strength</td>
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<td>F_y</td>
<td>Yield Strength</td>
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<td>g</td>
<td>distance between yield lines</td>
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<td>I</td>
<td>Moment of Inertia</td>
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<td>L</td>
<td>Angle Length, Beam Length</td>
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<td>L_eh</td>
<td>Horizontal Bolt Edge Distance</td>
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<td>L_ev</td>
<td>Vertical Bolt Edge Distance</td>
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<tr>
<td>M</td>
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<td>Factored Moment</td>
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<td>M_pl</td>
<td>Plastic Moment</td>
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<tr>
<td>M_y</td>
<td>Moment Resistance</td>
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<tr>
<td>m</td>
<td>Number of Shear Planes</td>
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<td>n</td>
<td># of Vertical Bolt Rows</td>
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<td>Factored Shear Load</td>
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<td>θ</td>
<td>End Rotation</td>
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<td>δ</td>
<td>Deflection</td>
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<tr>
<td>w</td>
<td>Angle leg width</td>
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1. INTRODUCTION

Shear connections in steel construction are commonly used in connecting horizontal beams to supporting columns or girders. In either case the primary function of these connections is to transfer vertical gravity loads to the supports through shear forces.
1.2 Types of Shear Connections

Shear connections can be designed and fabricated in several different configurations. Each of these configurations utilize a set of connecting materials, welds and fasteners in order to provide the required load transfer mechanisms. Figure 1 shows five typical shear connections: (i) shear tab, (ii) double-angle, (iii) end plate, (iv) seated and (v) tee connection. There are advantages and disadvantages associated with each of these connection types. A designer may use one type over another based on different factors such as fabrication preference and costs, finishing type (painting, galvanizing, etc.), erection requirements, and loading category.

![Common Shear Connection Types](image)

1.3 Motivation

This paper focuses on the behaviour and design of shear connections in terms of strength and ductility. In structural design strength can be defined as the resistance that a structural component can provide without failure and excessive deformation. In the same context ductility can be defined as the incremental deflections that occur within the structural component prior to a threshold, such as the start of a reduction in strength. As these definitions imply, strength and ductility of a structural component are not two mutually exclusive design parameters and they must be considered simultaneously. The design of simple shear connections also follows the same design philosophy. However, the current Canadian design guidelines included in the Handbook of Steel Construction published by the Canadian Institute of Steel Construction (CISC) do not provide any formulated procedures outlining the requirements for strength and rotational ductility. In this paper, a formulated approach to shear connection design for strength and ductility is proposed and compared with the current American Institute of Steel Construction (AISC) procedure.

Consideration of axial forces in the design of shear connections is also crucial in some cases. Transferring axial forces may result from lateral wind or earthquake loads, movement of pipes, machines or equipment, failure of a horizontal load transferring member, or lifting and transportation loads in modular structures. In these cases the shear connection must provide adequate strength to transfer the combined shear and axial load while maintaining flexibility requirements. This paper mainly focuses on the design of bolted double angle shear connections subjected to shear loads. The additional requirements for the design of shear connections subjected to combined shear and axial load is currently under study by the authors.

2. THEORETICAL BACKGROUND

The Steel Construction Manual published by AISC summarizes the design requirements for a shear connection, also known as a simple connection, as:

(i) “A simple connection transmits a negligible moment. In the analysis of the structure, simple connections may be assumed to allow unrestrained relative rotation between the framing elements being connected. A simple connection shall have sufficient rotational capacity to accommodate the required rotation determined by the analysis of the structure.”

(ii) “Simple connections of beams, girders and trusses shall be designed as flexible and are permitted to be proportioned for the reaction of shears only, except as otherwise indicated in the design documents. Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic but self-limiting deformation in the connection is permitted to accommodate the end rotation of a simple beam.”
The applied vertical shear force in a simple shear connection can be considered as half the applied load for a symmetrically loaded beam. In addition to the shear force a small bending moment can be developed by a simple shear connection, as shown in Figure 2. The magnitude of the bending moment depends on the rotational stiffness of the connection relative to the rotational stiffness of the framing members. This moment can be determined from the equilibrium of a beam segment between the point of inflection where zero bending moment occurs and the connection. This is done by multiplying the shear force acting on the connection by the distance between the inflection point and the support (Astaneh A., 2005 June). Typically this bending moment is relatively small and is less than twenty-percent of the beam's plastic moment capacity. As per the AISC design requirement, this moment is considered negligible and is ignored. However, the effects of this moment must still be taken into account in the design of the connection by providing adequate rotational ductility (Astaneh & Nader, 1989). Failing to provide adequate ductility could cause premature sudden failure of the connection. As a result, it is crucial that shear connections be designed for both strength and ductility requirements.

Figure 2: Moment Distribution of Uniformly Loaded Beam

2.2 Strength Design

The strength design of bolted double angle shear connections is based on six scenarios, referred to as Ultimate Limit State (ULS) capacities. The governing failure mode is the one that renders the lowest capacity and is therefore referred to as the limiting state. Each of the failure modes has been verified through various experimental studies and are referenced in both CISC and AISC design manuals. The six design limit states can be summarized as:

1. Yielding of the gross area of the framing angles or member
2. Bearing capacity of the bolt holes in the framing angles or member
3. Bolt Tear-Out due to fracture of framing angles or member
4. Net Area Shear Rupture of the framing angles or member
5. Block Shear Rupture of the framing angles or member
6. Bolt Fracture

Failure modes 1 and 2 are considered ductile failure modes while modes 3 through 6 are considered brittle failure modes (Astaneh A., 2005 August). The capacity of the connection is established as the lowest capacity calculated from each failure mode. The capacity calculation of each mode is based on material properties that include: thickness, length, width, yield strength ($F_y$), tensile strength ($F_u$) and applicable resistance factors ($\Phi$). The procedure and formulas used to check the capacity of each failure mode will be discussed later and a design example based on current CISC and AISC design manuals will be presented.

2.3 Ductility Design

As the load carried by a simply supported beam increases, shear forces in the connection also increase and the connection response enters the plastic region. The combination of shear load and associated bending moment cause the connection to experience material yielding. As a result, a reduction in stiffness occurs that allows the connection to rotate relative to the support. If the load is increased further, the beam will eventually collapse due to the formation of a plastic hinge at a location along the beam span. At the point of failure the end rotation of the beam has greatly increased and this rotation must be accommodated by the end connection. If the connection is unable to provide the required rotation as the plastic hinge forms then a premature fracture in the connection can occur. The propagation of a fracture in the shear connection is considered a sudden brittle failure (Astaneh A., 2005 June).
The proposed design procedure in this paper is based on an existing AISC approach for the ductility design of shear connections. Consider a simply supported beam subjected to transversal loading. Such beam and the deformed elastic curve are shown in Figure 3. The elastic end rotation ($\theta_{el}$) of this beam assuming small rotations and linear material properties can be calculated from:

$$\theta_{el} = \frac{M_y L}{3EI}$$

However, due to the linear elastic behaviour assumption $\theta_{el}$ is not applicable as soon as the beam enters the plastic region. In a one span simply supported beam this occurs after the moment at the beam’s mid-span reaches the yield moment. At the yield point any increase in load will result in greater cross-section yielding and larger beam end rotations that cannot be accurately predicted. Assuming a $M_{pl}/M_y$ ratio of 1.25, the theoretical rotation of a simply supported beam can be approximated by $\theta_{pl}$. In a research by Astaneh (1988) a conservative estimate for the connection’s maximum rotational ductility demand ($\theta_{pl}$) corresponding to the beam’s plastic moment capacity has been established. According to Astaneh (1988) the rotational ductility demand can be conservatively estimated as 0.03 radians. Therefore, in this research, a shear connection that can adequately provide a beam end rotation of 0.03 radians is taken as a connection with adequate ductility.

In a research by Thornton (1996) on the design of tee shear connections he formulated the minimum required bolt diameter at the supporting face of a tee style connection that ensures adequate rotational ductility can occur. Thornton used yield line theory and the principle of virtual work to evaluate the theoretical moment couples associated with beam end rotations of 0.03 radians. Figure 4 illustrates a schematic of the yield lines that he assumed would develop as the connection is deformed under gravity loads. The result of Thornton’s work is presented in Equation 2. The theoretical results are in strong agreement with experimental values obtained from Lewitt, Chesson and Munse (1969) (Thornton 1996). The current 14th Edition AISC Manual of Steel Construction references this equation as a method to ensure connection ductility.

$$d_{b,\text{min}} = 0.163t_f \sqrt{\frac{F_y}{b} \left( \frac{b^2}{L^2} + 2 \right)}$$

Figure 3: Deformed Shape of Simply Supported Beam

Figure 4: Shear Tee Connection Yield Line
3. CURRENT CANADIAN DESIGN APPROACH

3.1 Strength Design

The current CISC Handbook of Steel Construction provides a series of tables in the “Framed Beam Shear Connections” section to aid in the design of shear connections. In the forefront of this section several design clauses from CSA S16-09 are referenced and used to populate the tables. A summary of the referenced clauses, failure mode descriptions and strength calculations are listed in Table 1. The use of the CISC Handbook tables provides an efficient manner to design shear connections for strength without computation of the equations illustrated in Table 1. However, the use of these design aid tables is limited due to the fact that these tables are developed for pre-selected connection material grades and set geometric parameters. As shown in Figure 5, CISC current design aids limit the designers to follow specific bolt gauge, bolt pitch, bolt edge distance and angle leg width. The CISC design aid table for bolted double angle connections provides design capacities for connections with up to and including 13 bolt rows. The design parameters in these tables do not allow for a wide range of practical connection design configurations. In Table 1 of this paper, a summary of capacity checks are formulated for the design of bolted-bolted double angle connections. These checks are valid for all design configurations including those that fall outside the parameters of the CISC design aid table. The application of these equations will be presented later in a design example.

Table 1: CSA S16-09 Shear Connection Design Clauses

<table>
<thead>
<tr>
<th>CSA S16-09 Clause</th>
<th>ULS Failure Mode</th>
<th>ULS Design Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.4.1.1</td>
<td>Shear Yielding (Framing Angles &amp; Beam Web)</td>
<td>$V_r = \phi A_g F_y$</td>
</tr>
<tr>
<td>13.12.1.2 (a)</td>
<td>Bolt Bearing (Framing Angles &amp; Beam Web)</td>
<td>$B_r = 3 \phi_b \phi_{ntd} F_u$</td>
</tr>
<tr>
<td>13.11</td>
<td>Bolt Tear Out (Framing Angles)</td>
<td>$V_r = \phi_b [0.6 A_g (F_y + F_u)/2]$</td>
</tr>
<tr>
<td>13.11</td>
<td>Net Area Shear Rupture (Framing Angles)</td>
<td>$V_r = \phi_b [0.6 A_g (F_y + F_u)/2]$</td>
</tr>
<tr>
<td>13.11</td>
<td>Block Shear (Framing Angles)</td>
<td>$V_r = \phi_b [U/A_b F_u + 0.6 A_g (F_y + F_u)/2]$</td>
</tr>
<tr>
<td>13.12.1.2 (c)</td>
<td>Bolt Shear</td>
<td>$V_r = 0.6 \phi_b \phi_{nm} A_b F_u$</td>
</tr>
</tbody>
</table>

![Figure 5: CISC Double Angle Beam Shear Connection Geometry](image)

3.2 Ductility Design

The CISC Handbook provides no formulated guideline to ensure adequate connection ductility for shear connections. However, the handbook provides a general statement, “to ensure connection flexibility the angle thickness selected should not be greater than necessary”. This paper explores the approach developed by Thornton (1996) as discussed earlier, to provide a design guideline for the framing angle thickness as a method to ensure adequate rotational ductility in the connection.

4. DESIGN RECOMMENDATIONS

4.1 Strength Design

In design of bolted double angle connections the failure modes discussed early must be checked against the applied loads. The following provides a procedure for ULS capacity design of bolted double angle shear connections. As stated earlier, material yielding and bolt-hole bearing are ductile failure modes and are the preferred governing limit
states. Note that the limit states of bolt tear-out, net area shear rupture and block shear rupture are not applicable for uncoped beams as assumed for the calculations provided in this paper.

1.a Yielding of Framing Angle Gross Area: \( V_t \leq V_r = \phi A_g F_s = 2(\phi Lt_a(0.66F_y)) \)

1.b Yielding of Beam Web Gross Area: \( V_t \leq V_r = \phi A_w F_s = \phi dt_w F_s \) (see CSA S16-09 CL13.4.1.1 for \( F_s \))

2.a Bolt Hole Bearing on Angles: \( V_t \leq B_r = 2(3\phi_{bt_a}d_b F_u) \)

2.b Bolt Hole Bearing on Beam Web: \( V_t \leq B_r = 3\phi_{bt_w}d_b F_u \)

3. Bolt Tear-Out of Framing Angles: \( V_t \leq V_r = 2\phi_u[(0.6)(n-1)(pt_a)(F_y+F_u)/2] + 2\phi_u[(0.6)(2(L_e-t_a)(F_y+F_u)/2] \)

4. Net Area Shear Rupture of the Framing Angles: \( V_t \leq V_r = \phi_u[0.6A_{gv}(F_y+F_u)/2] = 2\phi_u[0.6(L_t)(F_y+F_u)/2] \)

5. Block Shear Rupture of Framing Angles: \( V_t \leq V_r = \phi_u[U_i A_e F_u + 0.6A_{gv}(F_y+F_u)/2] = 2\phi_u[0.6(L_{et_a}t_c)(F_y+F_u)/2] \)

6.a Bolt Fracture – Supported Beam Web Bolt Group

When an eccentricity is created in a bolted connection the bolt group must be designed to resist the combined effect of direct shear and induced moment. This results in a reduced bolt group capacity. For bolted double angle connections, research performed by Astaneh and McMullin (1993) (Astaneh A., 2005 August) determined that the point of inflection moves towards the bolt centerline as shear load increases. At connection failure, the inflection point is at or very near the bolt centerline. As a result, Astaneh and McMullin (1993) recommended that the vertical shear load is assumed to be acting at the bolt centerline with no need to consider eccentricity, as shown in Figure 6 (Astaneh, 2005 August). In addition, this paper assumes that the shear plane intercepts the bolt threads. Therefore, a 0.7 reduction in bolt shear strength is included in Equation 3.

\[ V_t \leq V_r = 0.7(0.6\phi_{bt_a}d_b F_u) \]

6.b Bolt Fracture – Supporting Face Bolt Group

Based on the point of inflection shown in Figure 6, the supporting face bolt group is subject to combined shear and bending forces. To complete a conservative connection design, the bolt group should be designed for the combined effects. The Research Council on Structural Connections (RCSC, 2009) provides a method to determine the ultimate limit state interaction for bolts subjected to combined shear and tension. The interaction can be expressed using an elliptical interaction equation (RCSC, 2009). As proposed by Astaneh (2005), the elliptical interaction can be modified to represent the effects of combined shear and bending as shown in Equation 4. In this equation, the applied bending moment \( (M_i) \) is determined by multiplying the applied shear force \( (V_i) \) by the eccentricity \( (e) \). The results of this interaction may be conservative since in the case of combined shear and tension, all bolts are subjected to the same combined shear and tension load. Whereas, in the case of combined shear and bending, bolts located furthest from the centerline of the connection experience greater tension forces. In the case of combined shear and positive bending, bolts located at the bottom of the connection may experience no tension at all (Astaneh, 2005 August).
The shear resistance of the bolts ($V_r$) is determined using Equation 3. To determine the bending capacity of the bolt group ($M_r$) it is assumed that the bending moment is resisted by a couple created between the tension force in the upper bolt rows and a bearing compression force created at the bottom of the angles. The compression stress block depth can be determined using a simplified rectangular stress distribution and horizontal equilibrium as shown in Figure 6 and Equation 5. The bearing compression force ($C$) is determined using CISC S16-09 Clause 13.10.

Then summing moments produces the bending capacity, Equation 6. In this procedure, $T$ is the factored tensile load of the bolt group including prying action forces. The magnitude of $T$ can be calculated using the prying action formulas presented in the CISC Handbook of Steel Construction. The application of these formulas and prying action calculations are demonstrated in the design example. Note that the number of bolts assumed to carry tension load due to bending is the total number of vertical bolt rows minus the bottom most row (number of bolt rows in tension = $n-1$).

Calculations should also be completed for failure modes of material yielding, material rupture, bolt bearing and bolt tear out at the supporting member.

### 4.2 Ductility Design

Bolted double angle connections provide rotational ductility by bending deformation of the angle’s outstanding legs as shown in Figure 7. The provided ductility is primarily a function of angle material thickness and center-to-center bolt gauge in the supporting member. If the limited preselected angle sizes and corresponding bolt gauges are used according to the CISC “Framed Beam Shear Connections” section then adequate rotational ductility is provided. However, if the angle sizes or gauges do not follow the tabulated values a lack of rotational ductility may exist. To eliminate this possibility consider the following approach to calculate adequate framing angle thickness. These formulations are based on the work completed by Thornton (1996).

Assume that as the shear connection is loaded it rotates about the bottom of the outstanding angle leg and yield lines form as shown in Figure 7. As the angles rotate the bending couple creates a distributed shear force $V$ along the vertical bolt lines. A virtual work expression for work done by the shear force $V$ can be written as Equation 7. The left side of the equation represents the external work caused by the bending couple. The right side of the equation is the internal work done by the angles along the yield lines ($m_p$, $\theta$, $g$ and $\phi$ are defined in Equations 8, 9, 10 and 11 respectively).
\[ 7 \quad \sum V \delta = \sum m_p L \theta + m_p \sqrt{L^2 + b^2} \Phi \]

\[ 8 \quad m_p = \frac{1}{4} F_y t_i^2 \]

\[ 9 \quad \theta = \frac{\delta}{g} \]

\[ 10 \quad g = (b/L) y \]

\[ 11 \quad \Phi = \theta \sqrt{1 + (b/L)^2} \]

By subbing in \( \delta, g, \Phi \) and integrating the left side once from 0 to \( L \) produces Equation 12.

\[ 12 \quad V0bL/2 = m_p L \theta + m_p \theta \sqrt{1 + (b/L)^2} \sqrt{L^2 + b^2} \]

Setting \( \eta = b/L \) and inputting the expression for \( m_p \) Equation 12 can be rewritten as Equation 13.

\[ 13 \quad V = \frac{F_y t_i^2}{2b} \left[ 2 + \eta^2 \right] \]

Up to this point, the derivation of Equation 13 follows the procedure completed by Thornton (1996). The development of Equation 2 by Thornton (1996) assumed a bolt tensile strength of 90 ksi and a vertical bolt pitch (spacing) of 3in. In practice, these parameters can vary for different connection designs. As a result, this paper explores the derivation of a new expression that incorporates both of these parameters as variable inputs. By equating the distributed shear force to the tensile strength of a bolt and adding a 25% strength increase factor Equation 15 is developed. This expression is similar to the one presented by Thornton (1996) but with the addition of input variables bolt pitch (\( p \)) and bolt tensile strength (\( F_{nt} \)). It is typical in connection design to begin the design process knowing the desired bolt size and grade. These parameters can be predetermined by job requirements, costs or specific fabrication processes. Based on bolt diameter (\( d_b \)) the maximum allowable angle thickness that provides adequate rotation can be determined using Equation 16. This Equation must be satisfied in addition to the connection strength design requirements. The set of Equations 15 and 16 allow the designer to choose an approach that fits their design parameters. This enables the selection of bolt size and evaluation of angle thickness or selection of angle thickness and calculation of minimum bolt diameter.

\[ 14 \quad F_{nt} A_b = 1.25 pV \]

\[ 15 \quad d_{b \text{min}} = 0.9 t_i \sqrt{\frac{pF_y}{F_{nt} b (\eta^2 + 2)}} \]

\[ 16 \quad t_{\text{max}} = \frac{1.11 d_b}{\sqrt{\frac{pF_y}{F_{nt} b (\eta^2 + 2)}}} \]

5. DESIGN EXAMPLE

For the connection detail shown in Figure 8, the following design example calculates the shear connection ULS capacities. As discussed in the previous section, ductility design of the connection at the ULS is also included. The factored applied shear force on the connection is 300 kN. The beam is supported by a column flange that acts as a fully rigid support with sufficient capacity.
Yielding of Angles

AISC  J4.2(a) Rn = φ0.6FA0 = 2(1)(0.6)(300MPa)(229mm)(9.5mm)  \[ \text{Rn} = 783.2 \text{kN} \]
CISC  CL13.4.1 Rn = φ0.9FA0 = 2(0.9)(0.6)(300MPa)(229mm)(9.5mm)  \[ \text{Rn} = 775.3 \text{kN} \]

Yielding of Beam Web

AISC  J4.2(a) Rn = φ0.6FA0 = (1)(0.6)(350MPa)(310mm)(9.4mm)  \[ \text{Rn} = 611.9 \text{kN} \]
CISC  CL13.4.1 Rn = φ0.9FA0 = (0.9)(0.6)(350MPa)(310mm)(9.4mm)  \[ \text{Rn} = 605.8 \text{kN} \]

Bolt Hole Bearing on Angles

AISC  J3.10 Rn = 2n(φ2.4dFtFz)  \[ \text{Rn} = 2(3)(0.75)(2.4)(19mm)(9.5mm)(450MPa) \]
CISC  CL13.12 1.2(a) Rn = 3φ0.6dFtFz = (3)(0.8)(3x2)(19mm)(9.5mm)(450MPa)  \[ \text{Rn} = 1169.6 \text{kN} \]

Bolt Hole Bearing on Beam Web

AISC  J3.10 10 Rn = n(φ2.4dFtFz) = (3)(0.75)(2.4)(19mm)(9.4mm)(450MPa)  \[ \text{Rn} = 434.0 \text{kN} \]
CISC  CL13.12 1.2(a) Rn = 3φ0.6dFtFz = (3)(0.8)(3)(19mm)(9.4mm)(450MPa)  \[ \text{Rn} = 578.7 \text{kN} \]

Bolt Tear-out of Angles (Interior Bolts)

AISC  J3.10 Interior Bolt Rn = 2[(0.6)dFtFz] = 2(0.6)(0.75)(2)(76mm)(9.5mm)(375MPa)  \[ \text{Rn} = 415.5 \text{kN per Bolt} \]
CISC  CL13.11 Interior Bolt Rn = 2[(0.6)dFtFz] = 2(0.6)(0.75)(2)(76mm)(9.5mm)(375MPa)  \[ \text{Rn} = 487.4 \text{kN per Bolt} \]

Bolt Tear-out of Angles (Exterior Bolts)

AISC  J3.10 Exterior Bolt Rn = 2[(0.6)dFtFz] = 2(0.6)(0.75)(2)(76mm)(9.5mm)(375MPa)  \[ \text{Rn} = 207.8 \text{kN per Bolt} \]
CISC  CL13.11 Exterior Bolt Rn = 2[(0.6)dFtFz] = 2(0.6)(0.75)(2)(38mm)(9.5mm)(375MPa)  \[ \text{Rn} = 243.7 \text{kN per Bolt} \]

Bolt Tear-out of Angles (Total)

AISC  Rn = (n-1)(415.5 kN/bolt) + 1(207.8 kN/bolt)  \[ \text{Rn} = 1038.8 \text{kN} \]
CISC  Rn = (n-1)(487.4 kN/bolt) + 1(243.7 kN/bolt)  \[ \text{Rn} = 1218.5 \text{kN} \]

Net area Shear Rupture of Angles

AISC  J4.2(b) Rn = φ0.6FA0 = 2(0.6)(0.6)(450MPa)(229mm-3x22mm)(9.5mm)  \[ \text{Rn} = 627.1 \text{kN} \]
CISC  CL13.11 Rn = 2(0.6)dFtFz = 2(0.6)(0.75)(229mm)(9.5mm)(375MPa)  \[ \text{Rn} = 734.2 \text{kN} \]

Block Shear Rupture of Angles

AISC  J4.3 Rn = φ0.6FA0 + U0FtFa  \[ \text{Rn} = 627.1 \text{kN} \]
CISC  CL13.11 Rn = φ0.6FtFz + 0.6FA0  \[ \text{Rn} = 734.2 \text{kN} \]

Bolt Group in Beam Web

AISC  J3.6 10 Rn = 2n(φ3dFtFz) = 2(3)(0.75)(372MPa)(285mm^2)  \[ \text{Rn} = 477 \text{kN} \]
CISC  CL13.12 1.2(c) Vc = 0.7(0.6)FtFzA0  \[ \text{Vc} = 477 \text{kN} \]

Bolt Group at Supporting Member

AISC  M₁ = cV₁ = (68mm)(300kN)  \[ \text{M₁} = 20.4 \text{kN-m} \]
CISC  \[ \text{P₁} = 44.5 \text{kN per Bolt} \]

\[ \text{a₁} = 43.5 \text{mm} \]
\[ \text{b₁} = 49.5 \text{mm} \]
\[ \delta = 0.72 \]

K = 4b φ0d(φFtFz) = 4(0.6)(9.5mm)(450kN)(9.5mm)(300MPa)  \[ \text{K} = 9.6 \]

\[ \alpha = \frac{K}{1 + \delta} \]
\[ \text{T₁} = 84.5 \text{kN per Bolt} \]

\[ \text{a₁} = 43.5 \text{mm} \]
\[ \text{b₁} = 49.5 \text{mm} \]
\[ \delta = 0.72 \]

\[ \text{T₁} = 84.5 \text{kN per Bolt} \]

\[ \text{a₁} = 43.5 \text{mm} \]
\[ \text{b₁} = 49.5 \text{mm} \]
\[ \delta = 0.72 \]

\[ \text{T₁} = 84.5 \text{kN per Bolt} \]

\[ \text{T₁} = 338 \text{kN} \]
The design of shear connections must satisfy strength requirements while providing rotational ductility. Currently the CISC Handbook of Steel Construction does not provide formulated guidelines for strength design of shear connections. In addition, no limits are directly stated to ensure adequate rotational ductility. In this paper, a formulated procedure outlining the design of bolted double angle shear connections was presented. The recommended procedure includes an ultimate limit state design of shear connections for strength. In addition, two alternative methods are presented to ensure adequate rotational ductility is provided, these include: (i) Limiting connection material thickness based on connection geometry and bolt parameters, or (ii) proportioning of bolt diameter to suit connection geometry and material thickness. The formulated procedure was summarized in a design example of a bolted double angle shear connection. The design example compared the AISC and CISC design calculations and also facilitated a proposed design process to determine the combined effects of shear and bending on the support face bolt group. As illustrated in the design example the bending porting of the interaction equation can render a relatively large contribution to the final connection utilization ratio. For the design example in this paper the moment due to the shear force eccentricity contributed 28% to the connection utilization ratio.

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REFERENCES


