



## Net Section Fracture Assessment of Welded Rectangular Hollow Structural Sections

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### Abstract

Rectangular Hollow Sections (RHS) because of their high resistance to tension, as well as compression, are commonly used as a bracing member with slotted gusset plate connections in steel structures. Since in this type of connection only part of the section contributes in transferring the tensile load to the gusset plate, shear lag failure may occur in the connection. The AISC specification decreases the effective section net area by a factor to consider the effect of shear lag for a limited connection configuration. This study investigates the effective parameters on the shear lag phenomenon for rectangular hollow section members connected at corners using a single concentric gusset plate. The results of the numerical analysis show that the connection length and connection eccentricity are the only effective parameters in the shear lag, and the effect of gusset plate thickness is negligible because of the symmetric connection. The ultimate tensile capacity of the suggested connection in this study were compared to the typical RHS connection presented in the AISC and the similar double angle sections connected at both legs. The comparison indicates that tensile performance of the suggested connection in this study because of its lower connection eccentricity is much higher than the typical slotted connection and double angle connections. Therefore, a new equation is suggested based on the finite element analyses to modify the AISC equation for these connections.

*Keywords:* Shear Lag; Tension Member; Welded Connection; Rectangular Hollow Sections; Net Section Fracture.

### 1. Introduction

Rectangular hollow sections (RHS) with the slotted gusset plate welded connection are commonly used in steel structures as a bracing member because of their high resistance to tension as well as compression. Three limit states are normally considered in the design of these members in tension (Equations 1 to 5): tensile yielding in the gross section; shear rupture along shear failure paths; and tensile rupture in the net section. The presence of bending moment or stress concentration in tension members reduce the section capacity. In bracing members, moment and stress concentration may develop from connection eccentricity when the neutral axis of the bracing member does not coincide with the centroid of connection lines depending on the connection types. Gusset plate connections represent the easiest method of connecting RHS brace members and since in this type of connection only part of the section contributes in transferring the tensile load to the gusset plate, stress concentration near the connection area in addition to the secondary moment may results in connection failure under a load less than the section capacity which is known as a shear lag effect (see Figure 1).

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The shear lag failure in these connections is dictated by parameters such as connection length, type of cross-section, gusset plate thickness, and connection eccentricity. The AISC Specification [1] decreases the effective section net area by a factor (U) to consider the effect of shear lag. The basis of this equation is the experimental research conducted by Munse and Chesson [2-3] on riveted and bolted connections to determine the effective factors in strength reduction of a tension member. Numerous experiments were performed on different sections including plates, angles, I shaped built up members, channels and several equations were suggested which were subsequently used by the AISC Specification.

Limit states for designing tensile members:

For tensile yielding in the gross section:

$$P_n = F_y A_g \tag{1}$$

For shear rupture in the net section:

$$P_n = \min \{ 0.6 F_y A_{gv}, 0.6 F_u A_{nv} \} \tag{2}$$

For tensile rupture in the net section

$$P_n = F_u A_e \tag{3}$$

$$A_e = A_n U \tag{4}$$

The shear lag factor (U), for a single concentric gusset plate (Figure 2a) is determined as Equation 5.

$$U = 1 - \bar{x}/l, l \geq H$$

$$\bar{x} = (B^2 + 2BH)/4(B + H) \tag{5}$$

Where  $F_y$ = specified minimum yield stress,  $A_g$ =gross area of member,  $A_{gv}$ = gross area subject to shear,  $A_{nv}$ = net area subject to shear,  $A_e$ =effective net area,  $l$ = Weld length,  $B$ = Overall width of HSS member in the perpendicular plane of the connection and  $H$ = Overall height of HSS member in the plane of the connection as shown in Figure 2a.

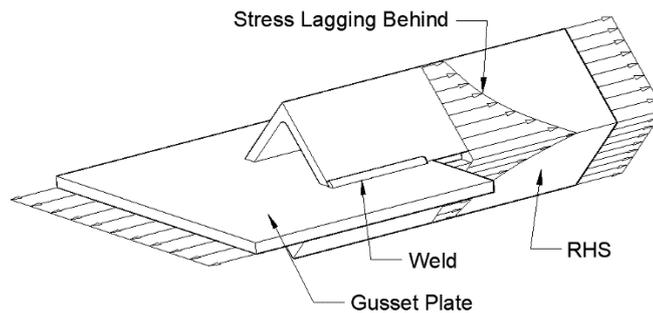


Figure 1. Shear lag in RHS Sections

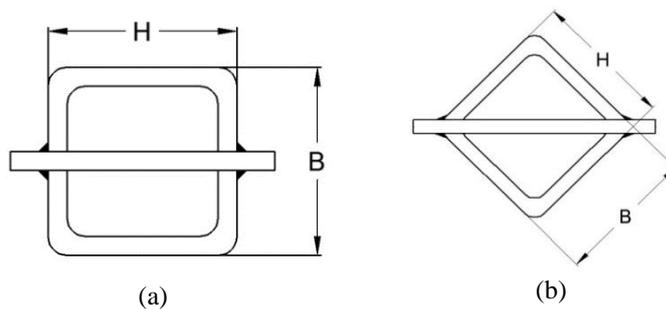


Figure 2. Slotted rectangular hollow sections connection; (a) Typical connection, (b) Connected at corners

Hollow structural sections, angles, channels, tees, and I-beam sections have been frequently tested under axial displacement, as such sections are popularly used as bracing members. The objective is to understand the tension behavior and predict the shear lag failure of such members in an attempt to improve the performance of these elements. Fang et al. [4] investigated the ultimate tensile capacity of angles and tees connected to a single gusset plate, the results showed that the balance weld can increase the section efficiency of these connections. Moreover, they also investigated the behaviour of coped steel I-beams with single-sided connections [5] and it was found that increasing the rotational stiffness of the connection could increase the block shear capacity. Behaviour of T-end connections welded to rectangular hollow section members subjected to tension was studied by Saidani [6]. The experimental results revealed that thickness of cap plates has a negligible effect on the ultimate joint capacity. Abedin et al. [7, 8] studied tension

behaviour of single and double angle connections. The results of the numerical analyses showed that, in single angle connections, the effects of connection length, connection eccentricity, and gusset plate thickness are more pronounced, while in double-angle connections the last parameter is not very critical.

Few studies have been carried out to investigate the shear lag effect on stainless steel [9, 10] and high strength steel connections [11-14]. The results show that the ductility of stainless steel is sufficient to ensure extensive redistribution of stresses prior to fracture. However, design equations would lead to unsafe estimates of the ultimate tensile capacities of high strength steel connections. Moreover, experimental and numerical studies on cold formed steel sections subjected to tension indicated that design codes equations are unsafe in some cases for such members and consequently new equations were suggested to improve the design codes [15, 16].

Rectangular and circular hollow sections have been frequently tested under cyclic loading. The objective was to understand the behaviour of these members with a concentric gusset plate connection under seismic excitations and improving the performance of such connections. In a study by Han et al., [17] the effect of width-thickness ratio on the seismic behaviour of HSS bracing members was investigated. This study showed that small width-thickness ratio can lead to an early fracture at the slotted end of the bracing members. Hadad et al. [18-20] investigated the behaviour of concentric HSS braces subjected to a series of cyclic loadings. It was found that the gusset plate thickness increases the fracture life and energy dissipation of the connection. New fracture and energy life equations were proposed for improving the design.

Several studies on the tensile behaviour of slotted HSS connections have previously been carried out. Most of studies showed that the current equations for shear lag design might be too conservative [21-23]. Korol [24] tested eighteen slotted connections including rectangular and square HSS members. He concluded that for connections with the  $L/w$  ratio around unity, connection efficiencies are close to 1.0, and block shear is the failure mode of connections where the  $L/w$  ratio is less than 0.6. Cheng et al. [25] tested nine specimens of slotted circular HSS connections that were connected to the gusset plate at their ends. The results showed that the section yielding was the failure mode for most of the specimens with a high level of ductility. In studies by Willibald et al. [26, 27], tension behavior of slotted HSS connections was investigated. The results indicated that net section efficiencies attained in the tests were much higher than those prescribed by the design specifications. Moreover, recent studies [28, 29] have shown that the negative effects developed by the shear lag in slotted end connections to hollow structural sections (HSS) may be negligible if a minimum weld length ( $L_w$ ) more than the distance between welds ( $w$ ) is used. As a result, the failure mode in the connections with the ratio of  $L_w/w \geq 1.0$  is mainly section fracture rather than failure in the connection.

Design specifications provide different equations for limited connection types to predict the effect of shear lag, however, these equations are based on the results of riveted and bolted connections, and according to the recent studies, applying such provisions can lead to a conservative design for welded connections. Moreover, effect of some parameters such as the thickness of gusset plate and symmetry in connections like the RHS connection as shown in Figure 3 are not studied before. Moreover, design codes only provide shear lag equation for the typical RHS connection as shown in Figure 2a. Therefore, considering the shear lag factor according to design specifications for other connection configurations (Figure 2b) would result in a conservative estimation. In this paper, tensile behaviour of a new RHS slotted connection as shown in Figure 3 is investigated. Note that, RHS connections connected at corners provide a higher resistance to buckling in the connection plane under the compressive loading because of the higher radius of gyration rather than the typical slotted RHS connections (Figure 2a). Moreover, the tensile performance of the suggested connection in this study because of its lower connection eccentricity is much higher than the typical slotted RHS connection.

The purpose of this paper is to study the significant parameters in shear lag factor estimation for specific slotted RHS connections, as shown in Figure 3. Particularly, the effects of gusset plate thickness ( $t_g$ ), connection length ( $L_w$ ), gusset plate-free length ( $L_g$ ), and member-free length ( $L$ ) on RHS with three different sizes are investigated. To this end, detailed non-linear finite element models of the RHS connections were developed and validated using experimental results available in the literature. A parametric study was conducted to identify the important parameters that influence the shear lag effect. The FE results were compared to the predictions suggested by the AISC specification and based on the results, a simple equation to predict the shear lag in the RHS connections were developed to improve the design codes and are presented herein.

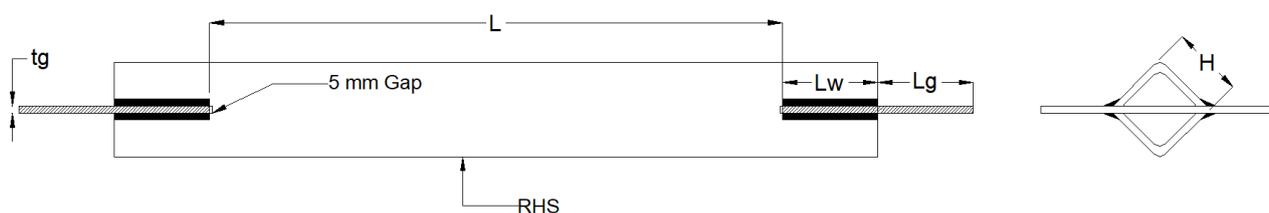


Figure 3. The RHS connection connected at corners with a single concentric gusset plate

## 2. Finite Element Analysis

The ABAQUS [30] software package was selected to perform finite element analysis (FEA) for investigating the shear lag effect on RHS members connected at corners to a single concentric gusset plate at both ends. A total of forty-eight (48) members with three different RHS sizes were modeled for considering the effective parameters on the shear lag phenomenon. For simplicity of modeling, only one-eighth of connections were modeled, and symmetrical boundary conditions were applied to the symmetric planes. All members were subjected to a uniform linear displacement load at the end of gusset plate until the failure of the connection. The analysis is performed considering the effects of geometrical and material nonlinearity. The material was assumed to behave as an isotropic with elastic-plastic hardening material based on Guo’s [31] tension test. The nominal stress-strain was converted to the true stress-strain curve calculated from Equation 6 and was extrapolated using power law (Equation 7), as shown in Figure 4. Three-dimensional 8-node solid elements with the reduced integration and hybrid formulation were utilized for the members.

$$\sigma_{true} = \sigma_{nom} (1 + \epsilon_{nom}) \tag{6}$$

$$\epsilon_{true}^{pl} = \text{Ln} (1 + \sigma_{nom}) - (\sigma_{true}/E)$$

$$\sigma = k \epsilon^m \tag{7}$$

Where  $\sigma_{true}$  = True stress,  $\sigma_{nom}$  = Nominal stress,  $\epsilon_{nom}$  = Nominal strain, and  $\epsilon_{true}^{pl}$  = True plastic strain.

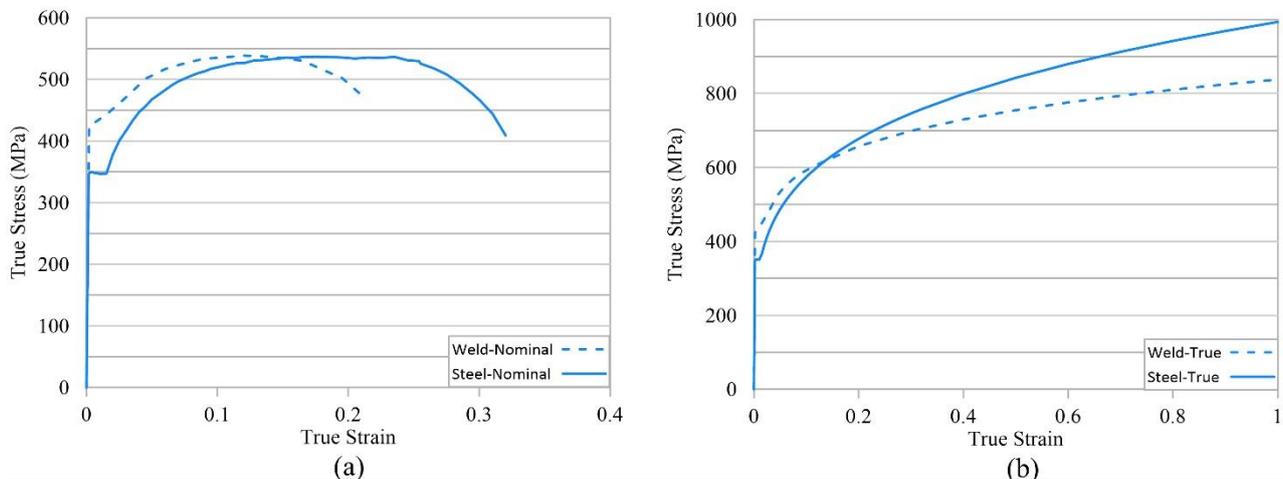


Figure 4. Material. (a) Nominal Stress-Strain (b) True Stress-Strain

The ductile damage model in ABAQUS is used for predicting the fracture in the connections. The development of fracture in this model states a gradual loss of material strength starting with damage initiation (damage approaches critical value) and has no effect on the previous pure plastic behavior. Figure 5 shows the stress-strain curve considering the progressive damage degradation compared to the true stress-strain curve and Figure 6 indicates a typical finite element model of the RHS member connected at corners. It should be noted that, depending on the connection parameters, e.g., connection length and gusset plate thickness, various failures such as gusset plate rupture, net section fracture, and shear rupture may happen.

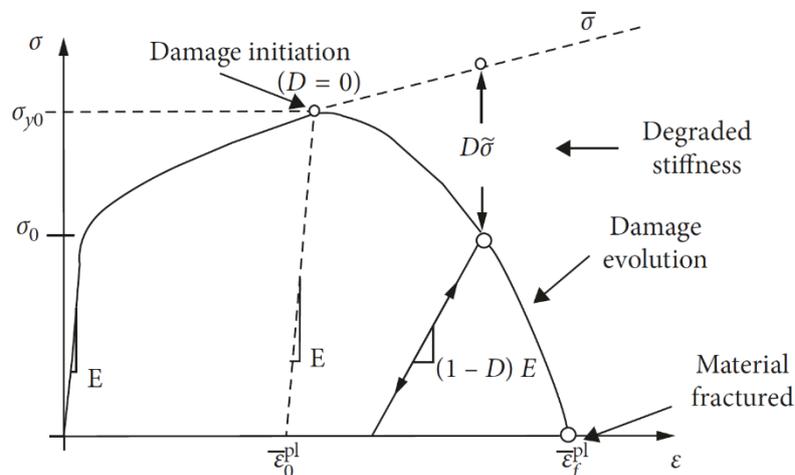


Figure 5. The ductile damage model in ABAQUS [30]

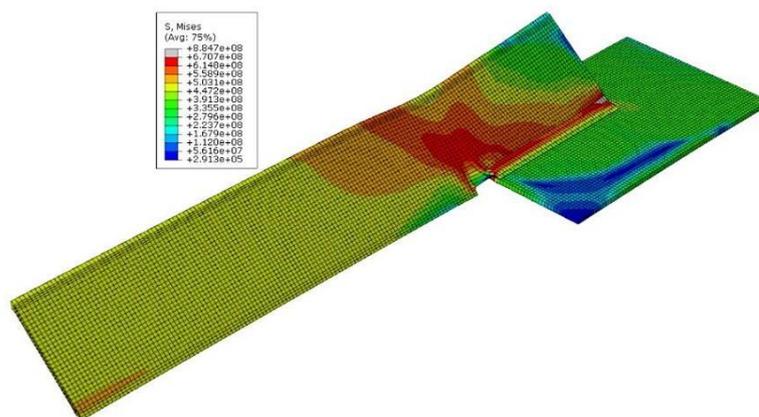


Figure 6. Typical FE model of the RHS connection

### 3. Finite Element Validation

Experimental full-scale test results conducted on hot-rolled steel channel sections in tension by Hui Guo [31] were used for validating the finite element model in this study. A total of ten full-scale channel sections with back-to-back welded connections to the gusset plate were tested, and only four of them failed by the section fracture, with a distance from the connection, and weld failure was the failure mode of the remaining specimens. Therefore, specimens with gross section failure mode were selected for the finite element validation. Table 1 summarizes the results of experimental tests and numerical analysis. A comparison between the results shows that the finite element model can predict the ultimate load as well as the failure mode for all the specimens. Moreover, the load-displacement curves obtained from the FE analysis were compared to the experimental test results to validate the global behavior of the models (Figure 7).

Table 1. Comparison of experimental test results [31] and numerical analysis

Specimen No.	Connection Type	Test Failure Mode	$L_w/d$	$P_{ult,Test}$ (kN)	FEM Failure Mode	$P_{ult,FEM}$ (kN)	$P_{Test}/P_{FEM}$
1	Double	Gross Section	1.51	750	Gross Section	759	0.99
2	Double	Gross Section	0.79	730	Gross Section	760	0.96
8	Single	Gross Section	1.32	369	Gross Section	378	0.98
9	Double	Gross Section	1.32	742	Gross Section	760	0.98

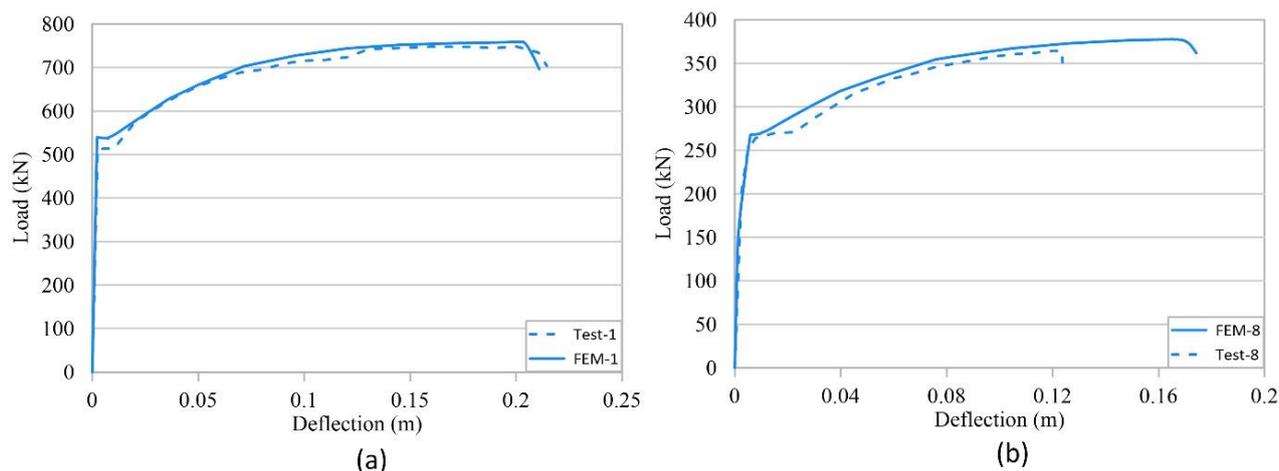


Figure 7. Finite element validation. (a) Specimen 1 (Double Section) (b) Specimen 8 (Single Section)

### 4. Parametric Study

A parametric study was conducted on the RHS connections using the validated FE model to investigate the shear lag effect on the ultimate load carrying capacity of the connections. A total of forty-eight (48) members with three different RHS sizes were used, as shown in Table 2, to cover different available RHS sizes. RHS members were connected to a single gusset plate at corners using the slotted connection. All the models had a gap of 5mm between the gusset plate and the RHS wall. A configuration of the connection length ( $L_w$ ) varying from 0.8H to 1.8H, gusset

plate thickness ( $t_G$ ) varying from 10 mm to 30 mm and different member length ( $L$ ) and gusset plate-free lengths ( $L_G$ ) was considered to study the effect of these parameters on the ultimate load carrying capacity of the connection. Furthermore, the out-of-plane eccentricity was considered as an individual parameter by using different RHS sizes. Various such configurations were considered, as illustrated in Table 3. The results of numerical study are discussed and analyzed for all the parameters in the subsequent sections.

**Table 2. Details of the Rectangular Hollow Sections**

Rectangular Hollow Section	H (mm)	s (mm)	A (cm <sup>2</sup> )
RHS 10	100	5	18.8
RHS 16	160	8	47
RHS 20	200	10	73.4

Note: H: Size; s: Thickness; A: Area of section

For easy referencing, models were assigned a code starting with the section size followed by three numbers (after the hyphen), representing the connection length ( $L_w$ ), the weld size ( $a$ ), and the thickness of the gusset plate ( $t_G$ ), respectively. For instance, the code RHS10-140-8-15 refers to the model of RHS 10 connected to a 15 mm thick gusset plate by the connection length of 140 mm and a weld size of 8 mm.

#### 4.1. Connection Length ( $L_w$ )

Table 3 and Figure 8 present the effect of connection length on the ultimate load carrying capacity of the connections. The results showed that the connection efficiency increases with increasing connection length for all RHS connections. For example, the shear lag coefficient of RHS20 models with the connection length of 220 mm and 280 mm were 0.8 and 0.91, respectively, which indicated 13.8% improvement in the model efficiency with a 27% increase in the length of connection. In general, increasing the connection length increases the connection efficiency linearly up to a certain point, which is the optimum connection length with a uniform stress distribution along the connection. By increasing the connection length beyond the optimum point, there is significantly less improvement in the tension efficiency of the connection. Moreover, it was observed that by increasing the section size (out-of-plane eccentricity), the connection efficiency decreased significantly. According to the test results, the shear lag coefficient of RHS16 and RHS20 with the same connection length of 220 mm were 0.94 and 0.80, respectively, which indicated a decrease of 14.9% in the connection efficiency by increasing the out-of-plane eccentricity. It worth mentioning that, in some models, short connection length caused the shear rupture of the member along the connection line.

**Table 3. Results of the parametric study**

Model	Section	$L_w$ (mm)	$a$ (mm)	$t_G$ (mm)	Failure Mode	Ultimate Load (KN)	$F_u A_g$ (KN)	$U_{AISC}$	$U_{FEM}$	$U_{FEM}/U_{AISC}$
RHS10-140-8-10	RHS 10	140	8	10	NF	934.7	971.9	0.73	0.96	1.31
RHS10-140-8-15		140	8	15	NF	903.8	932.8	0.73	0.97	1.32
RHS10-140-8-20		140	8	20	NF	876.7	893.7	0.73	0.98	1.34
RHS10-140-8-30		140	8	30	NF	808.0	815.6	0.73	0.99	1.35
RHS10-160-5-10		160	5	10	NF	952.4	971.9	0.79	0.98	1.24
RHS10-160-8-10		160	8	10	NF	955.5	971.9	0.79	0.98	1.24
RHS10-160-10-10		160	10	10	NF	947.9	971.9	0.79	0.98	1.23
RHS10-100-5-15		100	5	15	NF	813.4	932.8	0.63	0.87	1.40
RHS10-120-5-15		120	5	15	NF	888.1	932.8	0.69	0.93	1.35
RHS10-140-5-15		140	5	15	NF	904.6	932.8	0.73	0.97	1.32
RHS10-160-5-15	160	5	15	NF	924.2	932.8	0.77	0.99	1.29	
RHS10-180-5-15	180	5	15	GF	945.4	932.8	-	-	-	
RHS16-200-10-10	RHS 16	200	10	10	SR	2139.7	2566.8	-	-	-
RHS16-200-10-15		200	10	15	NF	2231.5	2504.1	0.63	0.89	1.43
RHS16-200-10-20		200	10	20	NF	2205.6	2441.5	0.63	0.90	1.45
RHS16-200-10-30		200	10	30	NF	2124.8	2316.3	0.63	0.92	1.47
RHS16-160-10-15		160	10	15	NF	1971.2	2504.1	0.63	0.79	1.26
RHS16-160-12-15		160	12	15	NF	1979.1	2504.1	0.63	0.79	1.26

RHS16-140-15-15	140	15	15	SR	1845.3	2504.1	-	-	-	
RHS16-160-15-15	160	15	15	NF	1982.4	2504.1	0.63	0.79	1.27	
RHS16-180-15-15	180	15	15	NF	2130.2	2504.1	0.67	0.85	1.28	
RHS16-200-15-15	200	15	15	NF	2250.4	2504.1	0.70	0.90	1.28	
RHS16-220-15-15	220	15	15	NF	2315.2	2504.1	0.73	0.92	1.27	
RHS16-240-15-15	240	15	15	NF	2361.9	2504.1	0.75	0.94	1.26	
RHS16-260-15-15	260	15	15	NF	2381.5	2504.1	0.77	0.95	1.24	
RHS20-260-12-10	260	12	10	SR	3170.4	4051.3	-	-	-	
RHS20-260-12-15	260	12	15	NF	3437.7	3972.2	0.71	0.87	1.22	
RHS20-260-12-20	260	12	20	NF	3434.5	3893.9	0.71	0.88	1.24	
RHS20-260-12-30	260	12	30	NF	3331.1	3737.4	0.71	0.89	1.25	
RHS20-280-10-20	280	10	20	NF	3526.4	3893.9	0.73	0.91	1.24	
RHS20-280-12-20	RHS 20	280	12	20	NF	3547.4	3893.9	0.73	0.91	1.24
RHS20-220-15-20	220	15	20	NF	3134.1	3893.9	0.66	0.80	1.22	
RHS20-240-15-20	240	15	20	NF	3299.1	3893.9	0.69	0.85	1.23	
RHS20-260-15-20	260	15	20	NF	3424.3	3893.9	0.71	0.88	1.24	
RHS20-280-15-20	280	15	20	NF	3524.5	3893.9	0.73	0.91	1.24	
RHS20-300-15-20	300	15	20	NF	3597.9	3893.9	0.75	0.92	1.23	

Note: GF: Gross section Fracture, NF: Net section Fracture, SR: Shear Rupture.

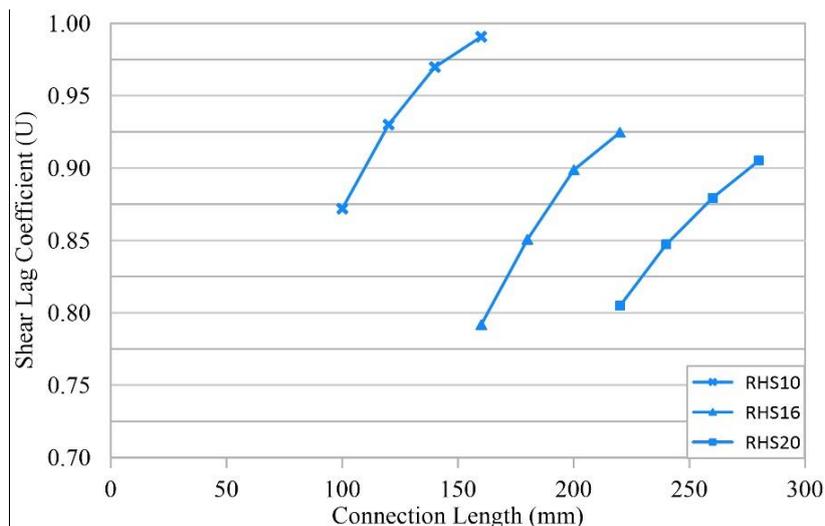


Figure 8. Parametric study - connection length

#### 4.2. Gusset Plate Thickness ( $t_g$ )

To illustrate the effect of gusset plate thickness on the shear lag of RHS slotted connection, the FE results for shear lag coefficients are plotted against the gusset plate thickness for the models which gusset plate thickness is the only variable parameter as shown in Figure 9. In the slotted connections, increasing the thickness of the gusset plate decreases the effective section net area of the member and, consequently, decrease the ultimate load carrying capacity of the connection. However, the shear lag coefficient, which is the ratio of these two values, was not significantly affected by the gusset plate thickness. In another word, because of symmetry, there would be no secondary bending moment due to eccentric loading in the slotted RHS connections. Therefore, the gusset plate thickness has a negligible effect on the bending deformation of the gusset plate and the ultimate tensile capacity of the connection.

According to the FE results, increasing the gusset plate thickness in the slotted connections increases the slotted area at the end of the RHS members; consequently, decrease the stress concentration at this area (critical section) and has a negligible effect on the connection capacity (see Figure 10). The results indicate an average increase of 2.6% in the efficiency of the connections with a 100% increase in gusset plate thickness.

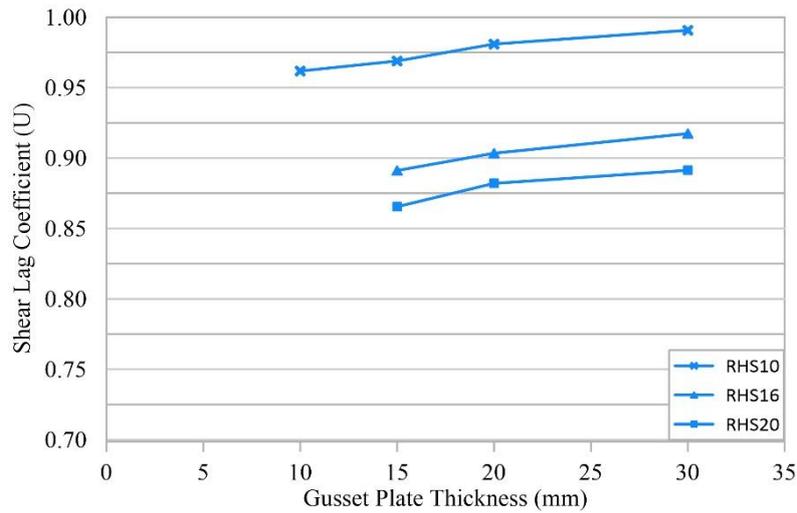


Figure 9. Parametric study - gusset plate thickness

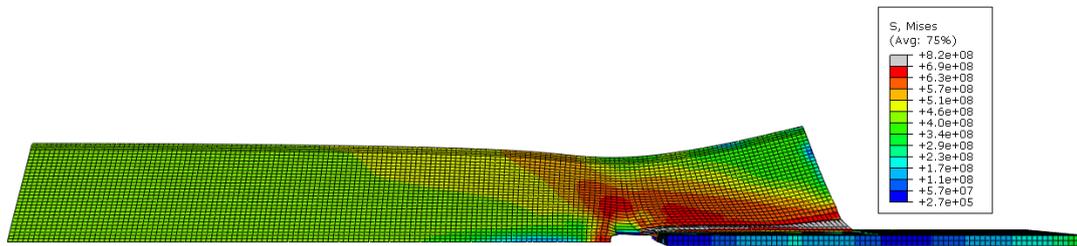


Figure 10. Stress distribution for the slotted RHS connection

### 4.3. Member and Gusset Plate-free Length

In order to investigate the effect of gusset plate-free length ( $L_G$ ) and member free-length ( $L$ ) on the efficiency of the slotted RHS connection, twelve models were developed. In these models, member and gusset plate-free length vary from 800 mm to 1600 and 100 mm to 300 mm, respectively, and all the other parameters were kept constant. Table 4 indicates the results of the parametric study on the member and gusset plate-free length. According to the results, it can be concluded that these two parameters do not have much impact on the ultimate load carrying capacity of slotted RHS connections.

Table 4. Effect of member and gusset plate-free length on the efficiency of the RHS connections

Model	Section	$L_w$ (mm)	$t_G$ (mm)	$L$ (mm)	$L_G$ (mm)	Failure mode	Ultimate load (KN)	$F_u A_g$ (KN)	$U_{AISC}$	$U_{FEM}$	$U_{FEM}/U_{AISC}$
RHS16-200-15-15(800)	RHS16	200	15	800	100	NF	2246.9	2504	0.70	0.90	1.29
RHS16-200-15-15(1200)		200	15	1200	100	NF	2250.4		0.70	0.90	1.29
RHS16-200-15-15(1600)		200	15	1600	100	NF	2244.8		0.70	0.90	1.29
RHS16-200-15-15(100)		200	15	1200	100	NF	2251.1		0.70	0.90	1.29
RHS16-200-15-15(200)		200	15	1200	200	NF	2250.4		0.70	0.90	1.29
RHS16-200-15-15(300)		200	15	1200	300	NF	2254.0		0.70	0.90	1.29
RHS20-220-15-20(800)		RHS20	220	20	800	100	NF		3143.0	3894	0.71
RHS20-220-15-20(1200)	220		20	1200	100	NF	3134.1	0.71	0.80		1.13
RHS20-220-15-20(1600)	220		20	1600	100	NF	3138.1	0.71	0.81		1.14
RHS20-220-15-20(100)	220		20	1200	100	NF	3152.3	0.71	0.81		1.14
RHS20-220-15-20(200)	220		20	1200	200	NF	3135.5	0.71	0.81		1.14
RHS20-220-15-20(300)	220		20	1200	300	NF	3134.6	0.71	0.80		1.13

Note: GF: NF: Net section Fracture.

### 5. Design Considerations

Considering the parametric study results, it was concluded that only connection length and eccentricity are the effective parameters in the shear lag phenomenon, while because of the connection symmetry, the gusset plate thickness does not have a significant impact on the ultimate load capacity despite of unsymmetrical connections. Other parameters, such as member and gusset plate-free length do not affect the ultimate tensile capacity of the RHS connections.

By comparing the shear lag coefficients obtained from this study and the coefficients suggested by AISC specification (Equation 5), as shown in Tables 3 and 4, it can be concluded that the design specification provides conservative predications of the ultimate load carrying capacity of the RHS connections connected at corners. The main reason is that the out-of-plan eccentricity of RHS connection connected at corners (Figure 2b) is much less than the typical connection (Figure 2a). However, AISC specification estimates  $\bar{x}$  using a simple equation considering the outside dimensions of RHS sections, as shown in Figure 2. Therefore, an equation to characterize the net section efficiency factor using the connection out-of-plane eccentricity to connection length ratio is proposed to modify the AISC equation for these connections. The proposed equation is expressed as:

$$U = 1.2 - \bar{x}/L_w \leq 1$$

$$\bar{x} = 0.4H \tag{8}$$

With an upper limit of 1. This equation is obtained based on the best curve fitting the FE results with a range of  $\bar{x}/L_w$  ratios between 0.26 and 0.43 as shown in Figure 11.

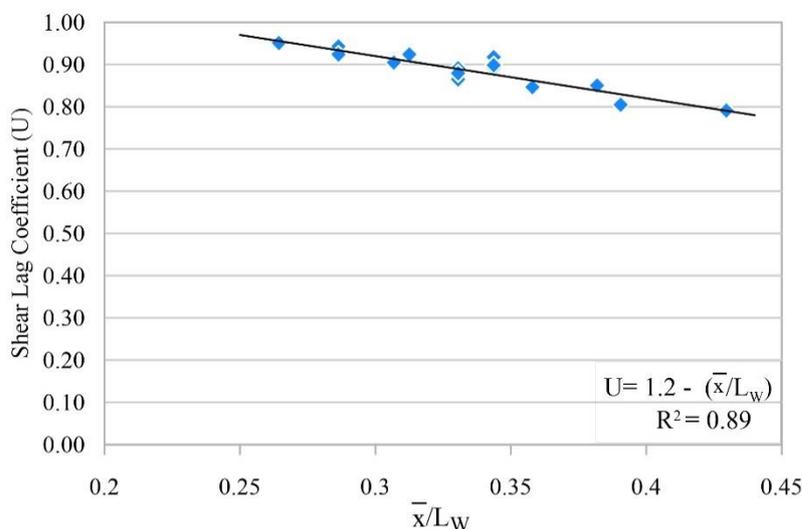


Figure 11. The proposed equation for the RHS connections connected at corners compared to FE results

Shear lag coefficient of the typical RHS slotted connection (Figure 2a) were obtained using the AISC prediction (Equation 5) and were compared to the valued obtained from the proposed Equation 8 for the suggested connection configuration in this study (Figure 2b) as shown in Figure 12. In this figure, shear lag coefficients of two different connection configurations were plotted versus various RHS size (H) to the connection length ratios ( $H/L_w$ ) for the three different section sizes studied in this paper. The results show that section efficiency of the suggested connection because of its lower connection eccentricity is much higher than the typical slotted RHS connection. (up to 28% in cases considered in this study). For example, shear lag coefficient for the RHS10 connected to the gusset plate with a 120 mm connection length ( $H/L_w = 0.83$ ) using typical connection is 0.69. However, shear lag coefficient for this section can be increased to 0.87 only if the suggested connection configuration is used.

Moreover, since configuration of the suggested connection in this study is similar to double angle sections connected at both legs (Figure 13), the results of this study were compared to the shear lag coefficient of double angle connections studied by Abedin et al. [7]. Table 5 shows the comparison of shear lag coefficients for two similar RHS and double angle connections with same leg sizes and connection lengths. The results indicate that tensile performance of the RHS connections is higher than the angles for the short connection lengths (up to 5% in cases considered). The reason is that connecting RHS to the gusset plate using the slotted connection, results in a lower out-of-plane eccentricity than the double angle connected at both legs. Moreover, the results show a more uniform stress distribution at the end of the slotted connection compared to the stresses at toe of the critical section in the angles.

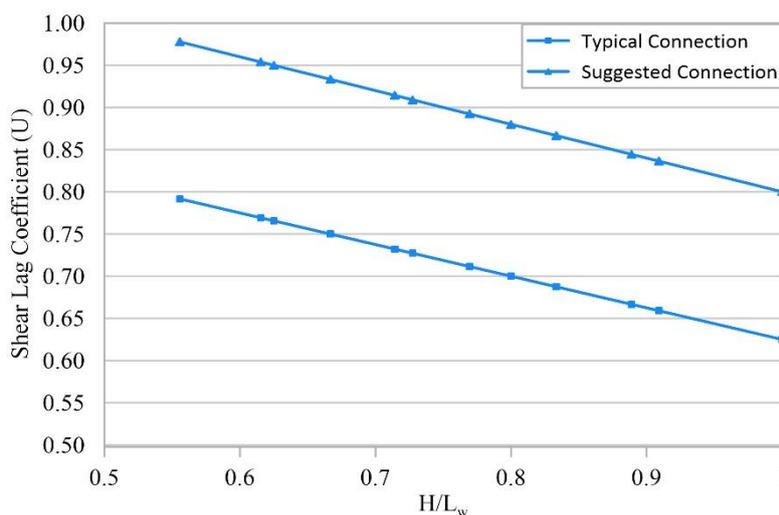


Figure 12. Comparison of the shear lag coefficient in the typical RHS connection and the configuration suggested in this study

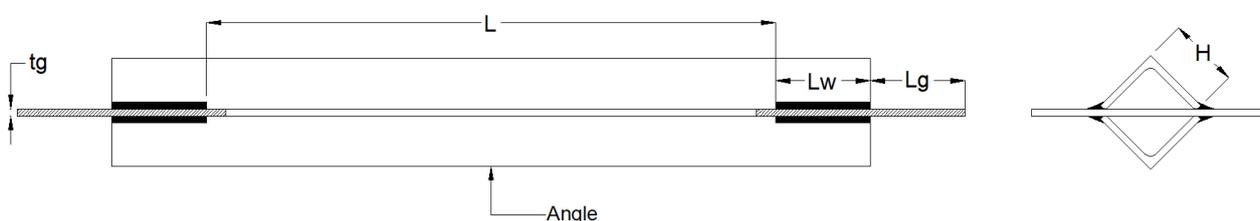


Figure 13. Double angles section connected at both legs similar to the suggested RHS connection

Table 5. Comparison of the shear lag coefficient in the suggested RHS connection and similar double angle connections

Section	L <sub>w</sub> (mm)	H/ L <sub>w</sub>	Shear lag coefficient (U)		U <sub>RHS</sub> /U <sub>Double Angles</sub>
			Double Angles	RHS	
RHS10 & L10	140	0.71	0.96	0.97	1.01
	160	0.63	0.98	0.99	1.01
RHS20 & L20	260	0.77	0.84	0.88	1.05
	280	0.71	0.86	0.91	1.05
	300	0.67	0.92	0.92	1.00

### 6. Conclusion

A numerical analysis for parametric study of shear lag effect on RHS members connected at corners to a single concentric gusset plate was conducted. Forty-eight models of tension RHS with welded connection, including test parameters of out-of-plane eccentricity, connection length, gusset plate thickness, member and gusset plate-free length were conducted using finite element analyses. The FE models were validated using experimental test results. Various failure modes such as shear rupture, net section fracture, and gross section fracture were observed depending on the connection geometry, and the models with net section fracture were selected for the parametric study on the shear lag effect. According to the FE results, as expected, it was observed that decreasing out-of-plane eccentricity and increasing connection length result in the improvement of connection efficiency. In addition, the effect of other parameters, such as member and gusset plate-free length and gusset plate thickness were found to be negligible. The comparison of the FE results and AISC specification estimations indicated that the current design equation is quite conservative for the ultimate load carrying capacity of RHS members connected at corners (up to 47% in cases considered). Therefore, a modified equation is suggested for these connections to improve the design of tension members. The results show that tensile performance of the suggested connection in this study because of its lower connection eccentricity is much higher than the typical slotted RHS connection (up to 28% in cases considered). Moreover, since configuration of the suggested connection in this study is similar to the double angle sections connected at both legs, the results were compared to the previous study on double angle connections. The comparison illustrates that section efficiency of the RHS connections is higher than the angles for the short connection lengths because of the lower out-of-plane eccentricity in the slotted connection.

## 7. Conflicts of Interest

The authors declare no conflict of interest.

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