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TECHNICAL NOTE

On Cold-Formed Steel Construction

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DESIGN FOR SPLICING OF COLD-FORMED STEEL WALL STUDS

This Tech Note Updates and Replaces Tech Note W106-16

Summary: This Tech Note discusses design methods for the splicing of two cold-formed steel studs in a curtain wall or interior nonstructural wall condition. Splicing of wall studs may be required in the field to extend studs to the required length.

Disclaimer: Designs cited herein are not intended to preclude the use of other materials, assemblies, structures or designs when these other designs demonstrate equivalent performance for the intended use. CFSEI documents are not intended to exclude the use and implementation of any other design or construction technique.

INTRODUCTION

The splicing of steel stud framing members should be avoided whenever possible. However, in all construction practices, the need for field modifications sometime arises. In cold-formed steel construction, these modifications often include the need to splice two studs together in order to attain a desired span or wall height. In any case where a splice is required, the strength of the splice must be evaluated to confirm it is adequate to resist the experienced loads. The stud member is also still required to be designed to attain the full span in all cases. This Tech Note will address how to determine the strength of the splice connection. Figure 1 shows a typical wall detail with spliced studs. In this case, the studs used to construct the wall are standard 12-foot-long members, while the required wall height is 15 feet. Therefore, the contractor is required to extend the members 3 feet by splicing additional stud sections to the 12-foot members.

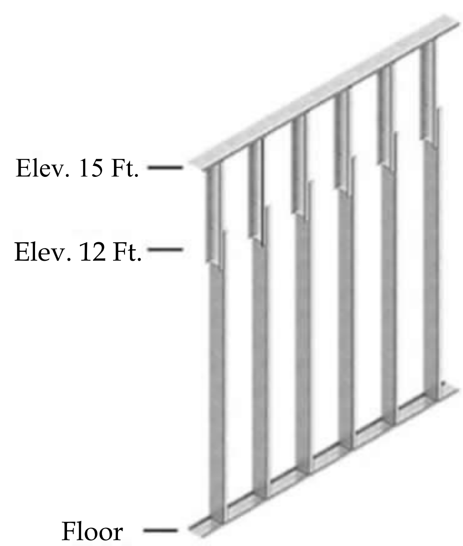

Figure 1: Wall Diagram

TYPES OF STEEL STUD SPLICES

The two most common types of splices used in cold-formed steel construction are the "back-to-back splice" and the "track capped splice". Figure 2 shows details of both splice configurations.

Back-to-Back Splice: As shown, the back-to-back splice is constructed by fastening the two studs together through the web of the members with a spread-out fastener pattern. The advantages of this splice configuration are that it does not require an additional track section and does not result in any build up at the face of the stud. However, the back-to-back splice does result in the line of connection for fastening the sheathing to the stud to shift over one flange width at the splice location.

Figure 2: Splice Types: Back-to-Back (Left), Track Capped (Right)

Track Capped Splice: The track capped splice is constructed by butting two studs together end to end, then capping them with a track section. The track section is then fastened to each stud through the flange of the members, as shown in the Figure. As opposed to the back-to-back configuration, the line of attachment for the sheathing remains straight along the height of the wall. The bearing of the flanges (nesting effect) also provides additional strength to the splice capacity to resist the applied moment. However, the track capped splice method does result in slight build up on the face of the stud due to the additional flange and fastener head.

DESIGN CONSIDERATIONS

In both splice designs, the overlap (lap) distance is critical to the performance of the splice, as will be shown in the following design examples. The fastener connection for the splice is the critical component to analyze the strength of splice configuration. In both cases the load applied to the stud at the splice location is evaluated, and the splice is analyzed based on the fastener's capacity to resist the resulting forces. Each splice design uses a slightly different approach with respect to applying the load to the fastener as will be explained in the following paragraphs and design examples.

Back-to-Back Splice: The design methodology presented for the back-to-back splice is based on the eccentrically loaded connection elastic method as presented in Part 7 of the AISC *Steel Construction Manual* as well as Chapter 8 of *Steel Design* by William Segui. The splice must be designed so that each fastener is able to resist the resultant force from the direct shear and the applied moment at the splice location. Therefore, the applied forces are divided equally among the number of fasteners in the screw pattern. Figure 3 illustrates the concept of determining the resultant force from the applied moment. The forces resulting from the moment are a function of the distance, d, from the location of the applied moment, which is the distance from the center of the screw group to the screw location. In this case, the x and y components from the moment and the shear are summed together, and the fastener connection is designed to resist the resultant vector force. The back-to-back splice is reliant purely on the resistance from the shear capacity of the fasteners through the web. In addition, the back-to-back splice produces some small eccentric forces resulting from the member configuration. Although small, these forces should be taken into account by the design professional, even if solely by inspection.

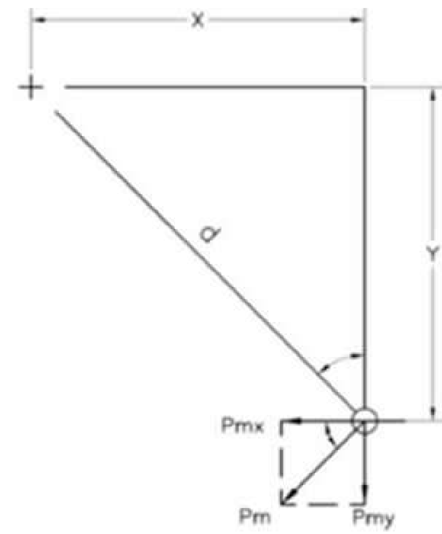

Figure 3: Illustration for Elastic Analysis from Steel Design – Chapter 8

Track Capped Splice: The moment couple method is used to analyze the track capped splice design. Due to the configuration, there are two symmetrical sides of the splice that must resist the applied moment. It is adequate to only analyze the screw pattern located in the higher moment region as it will experience the higher forces. Since there are two moment couples (diagonals) per screw pattern, the moment couple must be adequate to resist half of the applied moment at the splice location. The diagonal distance, D, of the bottom screw pattern is used as the moment arm for the couple, and the force, F, is the required force to resist the moment based on the moment arm distance. Once the required resultant force, F, is determined, the components of the force can be determined based on the geometry of screw pattern. Figure 4 depicts the configuration of the moment couple and component forces. The fastener connection is then analyzed to verify pull-out and shear capacity are adequate to resist the applied forces.

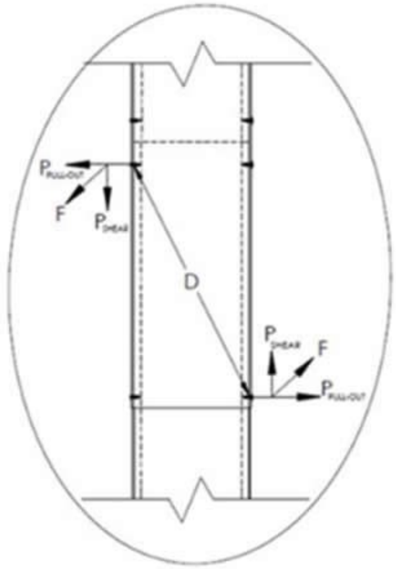

Figure 4: Moment Couple Configuration for Track Capped Splice Design

The factors that affect the load resisting capability of the splice are the steel thickness, ultimate strength of steel, screw diameter, screw shear and pull-out capacity and screw pattern.

By the time that it is determined that a splice is required, typically the wall studs have already been selected and purchased. Therefore, the thickness and strength of the steel is predetermined and the width of the screw pattern is limited by the depth of the stud.

The two controlling variables in designing a splice are then limited to the length of the splice and the fasteners to be used for the connection capacity. Increasing the spacing of the screws increases the moment arm from the location of the applied force to the screw, which decreases the force required to resist the applied moment. Therefore, increasing the splice length will directly relate to a splice that is capable of resisting a larger load. The combination of selecting a screw connection with adequate capacity and selecting a splice length that optimizes the load reduction are essential to a splice design.

The following design examples illustrate the design process for both types of splice designs by verifying that the capacity of the splice is adequate to resist the loads experienced in the specific condition. Another conservative approach is to design the splice to meet the full moment capacity of the member. This process is not dependent on the field conditions, which can provide the installer with more flexibility on the location of the splice.

Steel Stud Splice Design

Wall Configuration

$H := 15$ ft — Wall Height
$H_T := 12$ ft — Top of stud elevation
$s := 16$ in — Stud spacing
$P_w := 5$ psf — Interior lateral load
$P_G := 6$ psf — Finish material gravity load

Stud Properties - 362S162-33

$W_s := 3.625$ in — Depth
$t := 0.0346$ in — Design thickness
$F_u := 45$ ksi — Tensile strength

Back-to-Back Splice Design *(See Figure 5)*

TRY:
$L_s := 18$ in — Splice Length
$d := 0.190$ in — Diameter of #10 screw
$n := 4$ — Number of screws

Although AISI S100, *North American Specification for the Design of Cold-Formed Steel Structural Members* specifies a minimum edge distance of 1.5d, for simplicity a ½-inch horizontal edge distance and 1-inch vertical edge distance are used for the screw pattern."

Therefore, the screw spacing per the pattern in Figure 1 is as follows:

$h := L_s - 2 \text{ in.} = 16 \text{ in}$ $\quad$ $w := W_s - 1 \text{ in.} = 2.625 \text{ in}$

$y := \frac{h}{2} = 8 \text{ in}$ $\quad$ $x := \frac{w}{2} = 1.31 \text{ in}$

Splice centerline elevation:

$H_{CL} := H_T - \frac{L}{2} = 11.25 \text{ ft}$

Shear force acting at supports:

$W := P_w \cdot s = 6.7 \frac{lb}{ft}$ $\quad$ $V_x := \frac{W \cdot H}{2} = 50 \text{ lb}$

Forces acting on splice at centerline:

Horizontal shear force:
$V_{xc} := V_x - (W \cdot (H - H_{CL})) = 25 \text{ lb}$

Vertical shear force:
$VYC := P_G \cdot s \cdot (H - H_{CL}) = 30 \text{ lb}$

Applied moment:

$M_{CL} := (H - H_{CL})\left(V_x - \frac{Vs-Vx}{2}\right) = 140.6\ ft \bullet lb$

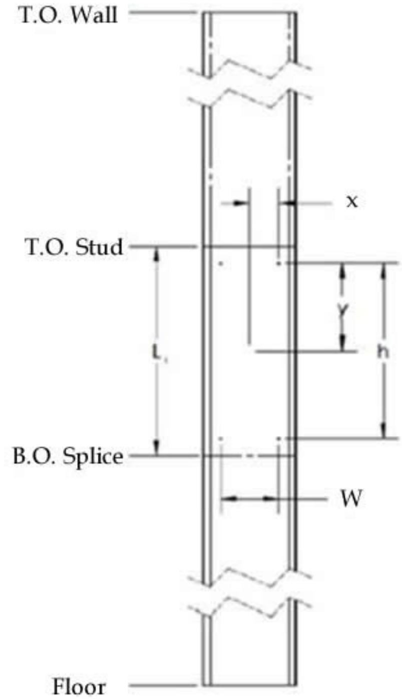

Figure 5: Back-to-Back Detail

Based on the shear analysis of eccentric loaded connections:

The horizontal and vertical components of force resulting from direct shear (per screw):

$P_{vx} := \frac{Vx}{n} = 6.3\ lb$ $P_{vy} := \frac{VYC}{n} = 7.5\ lb$

The horizontal and vertical components of force resulting from moment (per screw):

∑(x2 + y2) = n(x2 + y2)

$$P_{my} := \frac{M_{CL} \bullet x}{\sum_{n=1}^{4}(x^2 + y^2)} = 8.4\ lb \qquad P_{mx} := \frac{M_{CL} \bullet y}{\sum_{n=1}^{4}(x^2 + y^2)} = 51.4\ lb$$

Combined horizontal and vertical forces experienced (per screw):

$P_y := P_{vy} + P_{my} = 15.9$ lb $P_x := P_{vx} + P_{mx} = 57.7$ lb

Total force on each fastener: $P := \sqrt{Px^2 + Py^2} = 59.9$ lb

Verify the capacity of the screw connection is adequate per AISI S100 - Section J4.3.1.

Since $t_1 = t_2$, then the connection shear capacity shall be taken as the smallest of:

$P_{ns1} := 2.7 \bullet t \bullet d \bullet F_u = 798.7$ lb $P_{ns2} := 4.2\ (t^3 \bullet d)^{0.5} \bullet F_u = 530.2$ lb [Controls]

$\Omega = 3.0$ Section J4.3.2 (ASD) $P_{Shear} := \frac{Pm2}{\Omega} = 176.7$ lb

P_{Shear} = >P Therefore screw connection is adequate.

This Technical Note will establish the screw shear capacity from CFSEI Tech Note F701, *Evaluation of Screw Strength*, which compiles manufacturer published data for screw shear and tension values. The average of the screw shear strength values is used in the evaluation of this design example. Per Tech Note F701:

No. 10 Screw with diameter 0.190-inches: $P_{ss} := 1644$ lb $\frac{Pm}{\Omega} = 548\ lb$

$\frac{Pm}{\Omega} > P$ Therefore, shear capacity of screw is adequate.

Back to back splice design is adequate!

Track Capped Splice Design

Figure 6 shows the capped track splice configuration and corresponding screw pattern. The analysis of the track capped splice capacity will be based at the center of the screw pattern on the bottom half of the splice since it is located in the higher moment region. The same overall splice length from the previous design example is utilized for comparison purposes.

Elevation for center line of bottom screw pattern:

$$H_{CL} := H_T - \frac{L_3}{4} = 11.6\ ft$$

Forces acting on splice at centerline:

Horizontal shear force: $V_{xc} := V_X - (W \bullet (H - H_{CL}))$
$= 27.5$ lb

Vertical shear force: $V_{YC} := P_G \bullet s \bullet (H - H_{CL}) = 27$ lb

Applied Moment:

$$M_{CL} := (H - H_{CL}) \left(V_X - \frac{V_X - V_{XC}}{2}\right) = 130.8\ ft \bullet lb$$

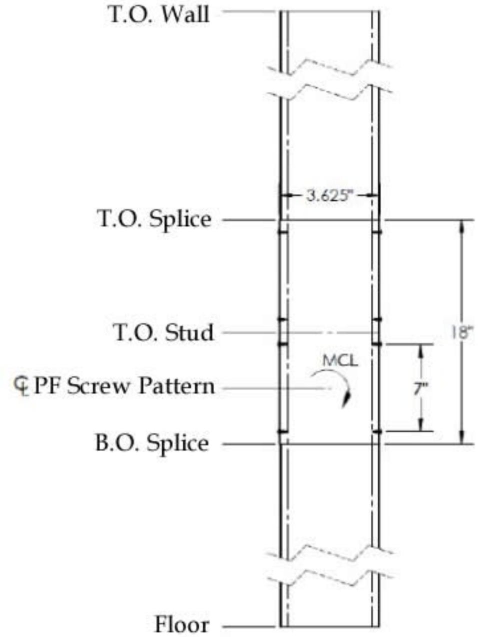

Figure 6: Track Capped Splice Design

Determine Moment arm for couple per Figure 4:

Length of bottom splice: $L_{bs} := \frac{L_x}{2} = 9\ in$

Vertical screw spacing: $y := L_{bs} - 2\ in = 7\ in$

Horizontal screw spacing: $x := W_s = 3.6\ in$

Moment Arm for couple: $D := \sqrt{y^2 + x^2} = 7.9\ in = 0.66\ ft$

Since there will be two diagonal moment couples resisting the applied moment:

$M_a := \frac{M_{cl}}{2} = 65.4\ ft \bullet lb$ Required moment per moment couple

Therefore, solving the moment couple equation for the resulting force per screw:

$$F := \frac{M_{\infty}}{D} = 99.1\ lb$$

The Shear and Pull-Out components of the force on each screw is as follows and shown by Figure 7:

$\Theta := \arctan\left(\frac{x}{y}\right) = 27.4\ deg$ $\phi := 90\ deg - \Theta = 62.6\ deg$

$F_{Shear} := F \bullet \cos(\phi) = 45.6\ lb$

$F_{PullOut} := F \bullet \sin(\phi) = 88\ lb$

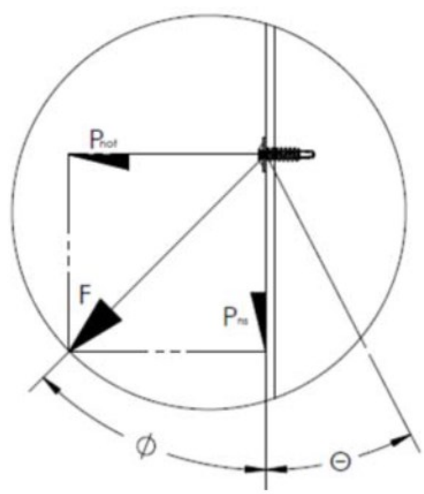

Figure 7: Geometry Detail for Force Component

Analyze the connections at each screw location based on combined shear and pull-out as outlined in AISI S100 - J4.5.2.1:

$\Omega = 2.55$

Shear Capacity $P_{nv} = 4.2\ (t^3\ d)^{0.5}\ F_u = 530.2\ lb$ Eq. J4.5..2-2

Pull-Out Capacity $P_{not} = 0.85\ t\ d\ F_u = 251.5\ lb$ Eq. J4.5.2-3

$\frac{\overline{V}}{P_{nv}} + \frac{\overline{T}}{P_{not}} = 0.44 \quad \frac{1.15}{\Omega} < = 0.45$ with $\Omega = 2.55$ Eq. J4.5.2-1a

Based on the author's engineering judgment, since the shear and pull-out forces are components of a resultant force, the two forces will only act simultaneously. Therefore, the shear and tension component forces are not required to be verified independently as stated in Section J4.5.2. Based upon Equation J4.5.2.1-1, the fastener connection capacity is adequate.

Verify the track section is adequate to resist the applied moment. Per a manufacturer's technical guide, the allowable bending moment of the 362T150-33 section is:

$M_{a\_track} := 3.6\ in \bullet kip = 296.7\ ft \bullet lb$

$M_{a\_track} > M_{CL}$ Therefore, the track section is adequate.

Therefore, the Track-Capped Splice design is adequate!

REFERENCES

1. AISC, *Steel Construction Manual*, 2023 Edition, American Institute of Steel Construction, Chicago, IL.

2. AISI S100-16 (2020), *North American Specification for the Design of Cold-Formed Steel Structural Members,* 2016 Edition (Reaffirmed 2020) with Supplement 2, American Iron and Steel Institute, Washington, D.C.

3. Tech Note F701-23, *Evaluation of Screw Strength*, Cold-Formed Steel Engineers Institute, Falls Church, VA.

4. Segui, W.T., *Steel Design*, Fourth Edition (2006), Thomson, Toronto, Ontario, Canada.

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