RCSC Main Meeting
June 8th, 2012 - Rowan University
8:00 am to 3:00 pm ET
Meeting Minutes

1. **Welcome - Call to order and Declaration of a Quorum (Carter)**

   Carter welcomed members and guests, declared a quorum based on the meeting attendance. The meeting was called to order at 8:10 am.

2. **Opening Comments and Circulation of Attendance Sheet (Larson/Brown)**

   Larson noted that there was an attendance sheet being circulated. Larson commented that if committee votes are made during the meeting that only voting members should participate.

   Brown welcomed meeting participants as the meeting host and made schedule and facility comments.

   Carter thanked Brown for his assistance with meeting arrangements.

3. **Call for introductions (Carter)**

   Introductions of members and guests were made around the room. At the start of the meeting there were 41 RCSC members and 3 guests present.

4. **Approval of the Meeting Agenda (Carter)**

   There was a call for acceptance of the meeting agenda. Motion by Harrold, second by Mitchell, all in favor. There were no additions to the agenda.

5. **Approval of minutes of the June 2011 meeting (Carter)**

   Minutes were posted on the RCSC website and distributed with the meeting materials.

   Carter made a call for a motion to accept the minutes of the previous meeting. There were no comments, additions or corrections. Motion to accept the minutes by Schlafly, second by Tide, all in favor.

6. **Executive Committee Report (Carter)**

   There was a report by Carter of the actions of the Executive Committee from the two Executive meetings held on January 19th and June 6th. These actions include;

   a. -Creation of a new identity, including graphics, letterhead layout, webpage layout, webpage design and logo’s for the RCSC.

   b. -New web site publication and maintenance agreement.

   c. -Review and acceptance of the recommendations of the nomination committee for open officer and director positions.

d. -Issuance of the Executive Committee ballot, review and ratify ballot results of the officer and director ballot.

e. -Review and revision of by-laws and creation of the by-laws ballot. Ratify ballot results and post new by-laws to the RCSC website.

f. -Clean up and elimination of stagnant research projects to provide a clearer view of available research funds.

g. -Support the creation of RCSC Facebook page.

h. -Planning for transition of Secretary/Treasurer position, including the movement of funds to AISC accounting for collection and disbursement of funds.

i. -Formalize decision to move to a calendar fiscal year from the current June to May fiscal year. This will help the council in a number of ways.

j. -Approval of the creation of a task group for evaluation of RCSC income and spending. Members; McGormley - Larson - Schlafly - Larson - Greenslade (chair)

k. -Membership and voting status review of RCSC membership, including reclassifications, replacement organizational members and new members since the last annual meeting.

l. -Approval of the creation of a Google Docs library for archive of RCSC documents.

There was a call for questions or comments on the Executive Committee report. Hearing none, there was a call for a motion to accept the report. Motion by Tide, second by Mitchell, all in favor.

7. Secretary/Treasurer's Report (Larson)

Larson gave the Secretary and Treasurers report (see attached pdf's for details).

A. Membership

1. Membership Summary - currently 92 RCSC members, with 4 new members since the last meeting. Anderson, Bornstein, Prchlik, and Auer. See membership list attached for additional details.

2. Review of new and reclassified members and unpaid members.

3. General statement of status of good health of RCSC membership.

4. Larson commented that Tide asked to step down as chair of the membership and funding committee. Larson thanked him for his fruitful efforts to support and increase RCSC membership in the past. A new chair will be considered and appointed by the Executive Committee.

B. Financials

1. Fiscal Year Change. There will be a move from the current fiscal period to a calendar fiscal year. Larson explained the numerous benefits for this move.

2. Invoicing - Will move to the spring of the year for that fiscal year.

3. Financial Statement - Shown as attached.

4. Cash Flow - Shown as attached, no research payments within the previous fiscal year.

5. Taxes - Approval of books by 3^{rd} party and posting of last year taxes.

6. Base of Operation - move permanent address to AISC in Chicago.

C. Other

Larson covered in detail other topics of interest to the council that were mentioned in the Executive Committee report.

1. RCSC Facebook Page (Sample review)

2. Google Docs file storage for Key Documents (Sample review)

3. boltcouncil@gmail.com Address

4. New identity for Council (Samples shown)

5. Website and domain move (Sample shown) including ability to use paypal for council dues and meeting registration fee's

Larson commented that this was his last report after three terms as Secretary/Treasurer, and welcomed Joe Greenslade to the position. He thanked Greenslade for his help with the website and Shaw for his help in resolving domain issues.

There was a call to accept the Secretary/Treasurer report. Motion to accept by Tide, second by Mitchell, all in favor.

Carter thanked Larson for his service to the council during his terms as Secretary/Treasurer.

8. Nominating Committee Report – (Rassati)

The nomination committee was chaired by Rassati. He gave a review of the recommendations of the nomination committee for open positions.

The recommendations of the nominating committee were carried through by council vote. Three officer positions were to be filled by;

-Carter (Chairman) was re-elected
-Miazga (Vice-Chair) was re-elected
-Greenslade (Secretary/Treasurer) was elected as first term

Two of the six director positions were up for election.

-Vissat was re-elected as director
-Ude was elected as a first term director

The ballot results were ratified by the Executive Committee.

9. Bylaws Ballot (Carter)

Carted reviewed and discussed the recent by-laws ballot. All ballot items passed or were resolved but there were a number of comments. Carter will discuss with the individuals. Some comments that require more work include the definition of a quorum - suggested by Tide - and the changes that will need to be made to support the new fiscal year by Larson.

10. Specification Committee report (Harrold)

Report from the previous day specification meeting. There were 30 members in attendance for the specification meeting. Harrold gave a summary of the actions and noted that the specification committee meeting minutes would be completed and distributed to members of the committee.

Harrold made a comment that a number of new items came up from members in the weeks, days and hours before the meeting. He asked that in the future if members have changes or improvements to the specification that they be submitted to the Chair of the specification committee well in advance of the meeting to facilitate time planning and to allow for the preparation of documents prior to the meeting.

Harrold gave a summary review and explanation of items that required main committee action. Results of specification ballots were reviewed and main committee votes taken.

RESOLUTION OF BALLOT RESULTS (Negatives) This is a summary. Details are available in the specification committee minutes.

-5.1 S06-002B Turn-of-the-Nut Rotation Tolerances (Shaw). There were a number of negatives to the original ballot. All but one was resolved. Negative from Mayes, Tide, Connor, Schlafly – Connor negative was unresolved but voted as non-persuasive by the sub-committee – Harrold moved to uphold the decision of the sub in the main meeting and made a motion to find negative non-persuasive, second by Shaw. Approved.

-5.2 S11-033 Merge Appendix B with main spec (Harrold). One unresolved negative by Ude – sub-committee voted and found non-persuasive – Harrold moved to uphold the decision of the sub in the main meeting and made a motion to find negative non-persuasive, second by Tide. Approved, with one negative vote.

-5.3 S11-035 Hole Definitions (Shaw) There were originally four negatives, Ferrell, Mayes, Curven, Ude. Two negatives were found persuasive. – new wording to be balloted.
Task group – redefine "engineer" ref in RCSC standard new wording

-5.4 S11-036 Pretension Definitions (Shaw)
Ballot item #4 – Hajjar, comment accepted as editorial. Ballot item passed.

Harrold noted the establishment of task groups and the items moving forward to ballot by the specification committee.

New task group RCSC definition of "Torque"
New task group Language ref "match-marking" when doing turn-of-nut.

Summary of new Ballot Items - There were 9 new items reviewed by the specification committee for possible ballot in the future. 6 of these items were officially moved to new business for the specification committee and ballots would be forthcoming. Additionally, task groups were assembled to address language for match marking and for defining torque.

11. Committee reports

A. Research Report (Ricles)

Final Report EFFECT OF FILLERS ON STEEL GIRDER FIELD SPLICE PERFORMANCE The final document was submitted to the Research Sub-committee for approval – See presentation and file attached.

Ricles mentioned the research committee and executive committee desire to clean up stagnant existing projects or projects that were approved but have not moved forward. With the approval of payment to PSU and IBECA there potentially would be no open projects, allowing us to start clean. Ricles will notify researchers.

B. Education Report (McGormley)

Education Committee - John McGormley - liked task group time during the meeting. They had a productive meeting and have a number of ideas moving forward. A previous project, the turn-of-nut video, has had no additional work performed.

> -Video of turn-of-nut – collected comments from last year's meeting. Will revise video during the next year. No progress since it was shown in San Francisco, but there were 17 members interested in providing feedback or helping with the project.

Ideas to come out of the task group meeting

1. -Develop an Education bulletin on the use of Skidmore, get help from Skidmore to move this forward.

2. -Develop a company based certification program for erectors.

3. -Ro-Cap explanation on how to do the test properly for erectors within specifications (Chad Larson offered his work on ASTM standard on this subject). Shneur, Friel, McGormley and Larson agreed to put some ideas together for better guidance on RC testing in the field.

4. -Development of Curriculum kit to offer to universities on bolting.

5. -Hyperlinks in RCSC standard to educational information – articles and videos.

The educational sub-committee will continue to work on these ideas and report progress in the future.

C. Liaison Report (Greenslade)

Greenslade gave a summary as Liaison on other committee progress and significant items that may be of interest to the RCSC.

1. ASTM - ISO - ASME - IFI
   a. ASTM F2833 (Magni) coating has been approved for application on A490.

   b. ASTM F 16 has several task groups that are reviewing and combining similar standards into fewer standards to improve consistency. F1136 has been combined. Larson is working on a project to combine the six structural bolt standards into one specification.

   c. ASME B18 - B18.2.6 will review swell and fin requirements of 1-3/8 and 1-1/2 bolts to reduce those to .060 from 0.09 over major diameter. (asked to review A449 and A354 allowances for sizes over 1-1/2.)

   e. B18.2.6M has been published.

   f. ASTM F959/F959M DTI heat treatment will be at the discretion of the manufacturer.

D. Membership and Funding Report (Larson)

1. Record membership (90+) 3 replacement, 4 new.
2. Projected membership Income of $40000

E. Status update of RCSC, AISC, CSA task group (Miazga)

Miazga reported that while the task group is still together there have been no major new findings or discrepencies between the above standards and that the group will continue to work towards harmonization of the related standards in the future. He stated that his report would be essentially unchanged from the previous meeting.

13. Technical Presentations

None

14. New Business

a. Carter - discussed handling of the potential for multiple votes from one organization ref bylaws: Reviewed with the membership at the meeting and there was no proposal for limiting the number of votes per organization. General support was there to keep everyone voting, but this may need to change in the future if it is believed there are contentious topics or underlying commercial interests.

b. Next RCSC Standard publication – currently RCSC is on a five year review there was a proposal to publish in 2015 rather than 2014. The intent is to keep RCSC a year in advance of other specifications that need to reference it. Vote taken to publish in 2014 as opposed to 2015 – 28 for and 1 against. RCSC will stay with 2014 publication date for the next revision.

15. Location and Dates for 2013 Annual Meeting

There was a call for meeting hosts for the next annual RCSC meeting. Possible locations were -

Cincinnati, OH -
Montreal, Canada -
Cleveland, OH-

A vote was taken by those present, with the total being;

29 for Cincinnati (University of Cincinnati)
3 for Montreal (McGill University)
1 for Cleveland (IFI)

The dates of the meeting are to be June 5, 6, and 7, 2013, Cincinnati, OH (University of Cincinnati)

16. Adjournment –

Having concluded the agenda items and with no more new business, there was a call for adjournment at 11:35. Carter mentioned sponsorship of the lunch to follow by St. Louis Screw and Haydon Bolt Co.

Motion to adjourn by Larson, second by Hundley, all in favor.

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2. **Opening Comments and Circulation of Attendance Sheet (Larson/Brown)**

3. **Call for introductions (Carter)**

4. **Approval of the Meeting Agenda (Carter)**

5. **Approval of minutes of the June 2011 meeting (Carter)**
   A. Minutes have been posted on the website

6. **Executive Committee Report (Carter)**
   A. Actions of Executive Meetings – January 19th and June 6th

7. **Secretary/Treasurer's Report (Larson)**
   A. Membership
      1. Membership Summary
      2. New Members, Unpaid Members
   B. Financials
      1. Fiscal Year Change
      2. Invoicing - Move to spring
      3. Financial Statement
      4. Cash Flow
      5. Taxes
      6. Base of Operation
   C. Other
      1. RCSC Facebook Page
      2. Google Docs file storage for Key Documents
      3. boltcouncil@gmail Address
      4. New identity for Council
      5. Website and domain move

8. **Nominating Committee Report – (Rassati)**
   A. Tabulation of ballots for the election of Directors/Officers

9. **Bylaws Ballot (Carter)**
   A. Results

10. Specification Committee report (Harrold)

A. Report from previous day specification meeting
B. Results of Specification Ballots
c. Summary of new Ballot Items

11. Committee reports

A. Research Report (Ricles)
   1. Final Report submission *EFFECT OF FILLERS ON STEEL GIRDER FIELD SPLICE PERFORMANCE*

B. Education Report (McGormley)

C. Liaison Report (Greenslade)
   1. ASTM - ISO - ASME - IFI

D. Membership and Funding Report (Larson)
   1. Record membership (90+) 3 replacement, 4 new.
   2. Projected membership Income of $40000

E. Status update of RCSC, AISC, CSA task group (Miazga)

13. Technical Presentations

14. New Business

15. Location and Dates for 2013 Annual Meeting

16. Adjournment –

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Todd C. Ude General Interest	exp 205 N. Michigan Ave. Suite 3600 Chicago, IL 60601	Phone: 312-616-6389 Fax: 312-616-6069 e-mail: todd.ude@exp.com
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Charles J. Wilson General Interest A.1, A.2	Consultant 2644 Shaker Road Cleveland Heights, OH 44118-4204	Phone: 216-932-1570 Fax: 216-932-1570 e-mail: wilsoncharlesj@yahoo.com
Alfred F. Wong Association A.1	Canadian Inst. of Steel Const. 3760 14th Avenue Suite 200 Markham, ON L3R 3T7 Canada	Phone: 905-946-0864 Fax: 905-946-8574 e-mail: afwong@cisc-icca.ca
Joseph A. Yura General Interest A.1	U of T Austin/Phil M. Ferguson Str. Eng. Lab. 10100 Burnet Road Building 177 Austin, TX 78758-4445	Phone: 512-471-4586 Fax: 512-471-1944 e-mail: yura@mail.utexas.edu

RCSC Annual Financial Report

Fiscal Year 2011

Ending May 31st, 2012

Starting Balance - June 1st - 2011			**$173,570.89**
	From tax return FY2010		
Total Assets - As of May 31, 2012			
	Savings	$76,367.73	
	Checking	$121,452.26	
		$197,819.99	**$197,819.99**
	Net Increase (Decrease) in Assets FY2010 to 2011		**$24,249.10**
Income	Research Contributions	$33,800.00	
	Meeting Expense Reimbursement	$14,800.60	
	Interest	$207.46	
	Other Reimbursements	$2.15	
		$48,810.21	**$48,810.21**
Expenses	Research Payment	$0.00	
	Research Payment	$0.00	
	Research Payment	$0.00	
	Other Expenses	($8,260.00)	
	Bank Fee's	($178.70)	
	Travel Expenses	$0.00	
	Administrative Expense	($1,529.99)	
	Meeting Expense	($14,592.42)	
		($24,561.11)	**($24,561.11)**
	Income Less Expenses		**$24,249.10**
Starting Balance - June 1st - 2011			**$173,570.89**
Income Less Expenses			**$24,249.10**
Total Assets as of May 31st - 2012			**$197,819.99**

RCSC Cash Projection

	2007	Paid	2008	Paid	2009	Paid	2010	Paid	2011	Paid	2012	Paid	Planned
Grondin - Fatigue													
Rassati - Ø Factors													
Birkemoe - Old													
Birkemoe - New													
Rassati - Ø Factors													
Dusicka - Fillers													
Rassati - Ø Factors	$40,265	-$20,133											
Dusicka - Fillers - *2006*	$8,000	-$8,000											
Rassati - Ø Factors - 2007			$20,132	-$19,482									
Dusicka - Fillers - 2007			$40,000	-$20,000									
Dusicka - Fillers - 2007					$20,000		$20,000		$20,000		$20,000		-$20,000
Birkemoe - New					$10,000	-$6,000	$4,000		$4,000		$4,000		-$4,000
Birkemoe - New					$2,000		$2,000		$2,000		$2,000		-$2,000
Brahimi					$10,000	-$5,000	$5,000		$5,000		$5,000		-$5,000
Brahimi							$40,000		$40,000		$40,000		-$40,000
Grondin - 3rd Edition					$12,000		$12,000		$12,000		$12,000		-$12,000
Grondin - 3rd Edition					$6,000		$6,000		$6,000		$6,000		-$6,000
Grondin - Fatigue - *2006*					$5,200		$5,200		$5,200		$5,200		-$5,200
Income		$39,500		$35,288		$42,069		$42,318		$48,810 *			$40,000
Research		$28,133		$39,482		$11,000							$20,000
Expenses		$7,461		$3,820		$3,506		$4,198		$24,561			$4,000
Balance		**$115,902**		**$107,888**		**$135,451**		**$173,571**		**$197,820**		Balance	$229,571

Pending Research

Available for Research $229,571

RESEARCCOUN

Form **990-PF**

Department of the Treasury
Internal Revenue Service

Return of Private Foundation
or Section 4947(a)(1) Nonexempt Charitable Trust
Treated as a Private Foundation

Note. The foundation may be able to use a copy of this return to satisfy state reporting requirements.

OMB No. 1545-0052

2010

For calendar year 2010, or tax year beginning 06/01/10 **, and ending** 05/31/11

G Check all that apply: ☐ Initial return ☐ Initial return of a former public charity ☐ Final return
☐ Amended return ☐ Address change ☐ Name change

Name of foundation
RESEARCH COUNCIL
ON STRUCTURAL CONNECTIONS

Number and street (or P.O. box number if mail is not delivered to street address) | Room/suite
3500 WEST HIGHWAY 13

City or town, state, and ZIP code
BURNSVILLE MN 55337

A Employer identification number
36-3967603

B Telephone number (see page 10 of the instructions)
952-890-7700

C If exemption application is pending, check here ▶ ☐
D 1. Foreign organizations, check here ▶ ☐
2. Foreign organizations meeting the 85% test, check here and attach computation ▶ ☐
E If private foundation status was terminated under section 507(b)(1)(A), check here ▶ ☐
F If the foundation is in a 60-month termination under section 507(b)(1)(B), check here ▶ ☐

H Check type of organization: ☒ Section 501(c)(3) exempt private foundation
☐ Section 4947(a)(1) nonexempt charitable trust ☐ Other taxable private foundation

I Fair market value of all assets at end of year (from Part II, col. (c), line 16) ▶ $ 0

J Accounting method: ☒ Cash ☐ Accrual
☐ Other (specify)
(Part I, column (d) must be on cash basis.)

Part I **Analysis of Revenue and Expenses** (The total of amounts in columns (b), (c), and (d) may not necessarily equal the amounts in column (a) (see page 11 of the instructions).)

	Line	Description	(a) Revenue and expenses per books	(b) Net investment income	(c) Adjusted net income	(d) Disbursements for charitable purposes (cash basis only)
Revenue	1	Contributions, gifts, grants, etc., received (attach schedule)	35,300			
	2	Check ▶ ☒ if the foundation is **not** required to attach Sch. B				
	3	Interest on savings and temporary cash investments	202	202	202	
	4	Dividends and interest from securities				
	5a	Gross rents				
	b	Net rental income or (loss)				
	6a	Net gain or (loss) from sale of assets not on line 10				
	b	Gross sales price for all assets on line 6a				
	7	Capital gain net income (from Part IV, line 2)		0		
	8	Net short-term capital gain			0	
	9	Income modifications				
	10a	Gross sales less returns & allowances				
	b	Less: Cost of goods sold				
	c	Gross profit or (loss) (attach schedule)				
	11	Other income (attach schedule) STMT 1	6,817		6,817	
	12	**Total.** Add lines 1 through 11	42,319	202	7,019	
Operating and Administrative Expenses	13	Compensation of officers, directors, trustees, etc.	0			
	14	Other employee salaries and wages				
	15	Pension plans, employee benefits				
	16a	Legal fees (attach schedule)				
	b	Accounting fees (attach schedule)				
	c	Other professional fees (attach schedule)				
	17	Interest				
	18	Taxes (attach schedule) (see page 14 of the instructions)				
	19	Depreciation (attach schedule) and depletion				
	20	Occupancy				
	21	Travel, conferences, and meetings	3,265		3,265	
	22	Printing and publications				
	23	Other expenses (att. sch.) STMT 2	934			
	24	**Total operating and administrative expenses.** Add lines 13 through 23	4,199	0	3,265	0
	25	Contributions, gifts, grants paid	0			0
	26	**Total expenses and disbursements.** Add lines 24 and 25	4,199	0	3,265	0
	27	Subtract line 26 from line 12:				
	a	**Excess of revenue over expenses and disbursements**	38,120			
	b	**Net investment income** (if negative, enter -0-)		202		
	c	**Adjusted net income** (if negative, enter -0-)			3,754	

Part II Balance Sheets Attached schedules and amounts in the description column should be for end-of-year amounts only. (See instructions.)

		Beginning of year (a) Book Value	End of year (b) Book Value	(c) Fair Market Value
Assets 1	Cash—non-interest-bearing	81,043	52,326	
2	Savings and temporary cash investments	54,408	121,245	
3	Accounts receivable ▶			
	Less: allowance for doubtful accounts ▶			
4	Pledges receivable ▶			
	Less: allowance for doubtful accounts ▶			
5	Grants receivable			
6	Receivables due from officers, directors, trustees, and other disqualified persons (attach schedule) (see page 15 of the instructions)			
7	Other notes and loans receivable (att. schedule) ▶			
	Less: allowance for doubtful accounts ▶ 0			
8	Inventories for sale or use			
9	Prepaid expenses and deferred charges			
10a	Investments—U.S. and state government obligations (attach schedule)			
b	Investments—corporate stock (attach schedule)			
c	Investments—corporate bonds (attach schedule)			
11	Investments—land, buildings, and equipment: basis ▶			
	Less: accumulated depreciation (attach sch.) ▶			
12	Investments—mortgage loans			
13	Investments—other (attach schedule)			
14	Land, buildings, and equipment: basis ▶			
	Less: accumulated depreciation (attach sch.) ▶			
15	Other assets (describe ▶)			
16	**Total assets** (to be completed by all filers—see the instructions. Also, see page 1, item I)	135,451	173,571	0
Liabilities 17	Accounts payable and accrued expenses			
18	Grants payable			
19	Deferred revenue			
20	Loans from officers, directors, trustees, and other disqualified persons			
21	Mortgages and other notes payable (attach schedule)			
22	Other liabilities (describe ▶)			
23	**Total liabilities** (add lines 17 through 22)	0	0	
Net Assets or Fund Balances	**Foundations that follow SFAS 117, check here ▶ [X] and complete lines 24 through 26 and lines 30 and 31.**			
24	Unrestricted	135,451	173,571	
25	Temporarily restricted			
26	Permanently restricted			
	Foundations that do not follow SFAS 117, check here ▶ [ ] and complete lines 27 through 31.			
27	Capital stock, trust principal, or current funds			
28	Paid-in or capital surplus, or land, bldg., and equipment fund			
29	Retained earnings, accumulated income, endowment, or other funds			
30	**Total net assets or fund balances** (see page 17 of the instructions)	135,451	173,571	
31	**Total liabilities and net assets/fund balances** (see page 17 of the instructions)	135,451	173,571	

Part III Analysis of Changes in Net Assets or Fund Balances

1	Total net assets or fund balances at beginning of year—Part II, column (a), line 30 (must agree with end-of-year figure reported on prior year's return)	1	135,451
2	Enter amount from Part I, line 27a	2	38,120
3	Other increases not included in line 2 (itemize) ▶	3	
4	Add lines 1, 2, and 3	4	173,571
5	Decreases not included in line 2 (itemize) ▶	5	
6	Total net assets or fund balances at end of year (line 4 minus line 5)—Part II, column (b), line 30	6	173,571

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rcsc

PRIMARY LOGO & IDENTITY GUIDELINES
CLIENT : Bolt Council

Primary Logo

Blue indicates **Clear Space**. The blue area must be kept free of other elements.

COLORS

Colors

Black	Yellow	Gray
K100	C0 M45 Y83 K0	K70
#454545	#454545	#454545

Minimum Logo Width

November 17, 2011

RCSC Identity Board
123 Address
City, State 00000

Dear Jack,

Et? Nihiliis vemus ma, uro verit fue pata, se cam intem essa is maiocch icatatus inatus hoctuam. Bondentere tum nosum des, publin senitab eniqueme poreci si public in videes es? Ivervis cum inuntic ina, nihilis ego esi signatus, quo num prit aressolicae aude re ad iam er laricestra inum medies obserortere imaio hoctarei tatatu moremen deto trum tua vivit viritab efecus suntem. Ad inultisquo hae ponum omnonsulocum huci firi sentrae diorius, omniam adhucta mendam hos, quo hora? quam. Ingul temussentus idium lostia rentu erfex numusquem praecon Ita, per licae, ducibut vid num pridemo

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Sincerely,

Bolt Council 3500 West Highway 13 Burnsville, MN 55337 952.890.7700 ph www.boltcouncil.org

boltcouncil
RESEARCH COUNCIL ON STRUCTURAL CONNECTIONS
rcsc

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Download the current ***Specification*** *for Structural Joints Using High-Strength Bolts* dated December 31, 2009.

Next meeting of the RCSC
June 6-8, 2012
Deptford, NJ

Welcome

The purpose of the RCSC is:

To stimulate and support such investigation as may be deemed necessary and valuable to determine the suitability, strength and behavior of various types of structural connections.

To promote the knowledge of economical and efficient practices relating to such structural connections.

To prepare and publish related standards and such other documents as necessary to achieving its purpose.

About Us

The RCSC is a non-profit, volunteer organization, comprised of over 85 leading experts in the fields of structural steel connection design, engineering, fabrication, erection and bolting. Previous, current, and future research projects funded by the RCSC serve to provide safety, reliability, and standard practice for the steel construction industry throughout the world.

The RCSC is actively soliciting research contributions to further our efforts to provide meaningful research, clear specifications, and practical application advice for our industry. Membership in the RCSC is open to any qualified individual, corporation, or organization in accordance with our bylaws.

Research

Have research you would like to perform or help sponsor? Please contact us.

For information send an email message to boltcouncil@gmail.com

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The current version of our specification, dated December 31st, 2009, is available for download on our publications page.

The next meeting of the RCSC is June 15th - 17th, 2011, San Francisco - Oakland, CA

Welcome

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Research?

Have research you would like to perform or help sponsor? Please contact us.

RCSC

RESEARCH COUNCIL ON STRUCTURAL CONNECTIONS

RESEARCH COUNCIL ON STRUCTURAL CONNECTIONS

Dear Jack,

Et? Nihiliis vemus ma, uro verit fue pata, se cam intem essa is maiocch icatatus inatus hoctuam. Bondentere tum nosum des, publin senitab eniqueme poreci si public in videes es? Ivervis cum inuntic ina, nihilis ego esi signatus, quo num prit aressolicae aude re ad iam er laricestra inum medies obserortere imaio hoctarei tatatu moremen deto trum tua vivit viritab efecus suntem. Ad inultisquo hae ponum omnonsulocum huci firi sentrae diorius, omniam adhucta mendam hos, quo hora? quam. Ingul temussentus idium lostia rentu erfex numusquem praecon Ita, per licae, ducibut vid num pridemo atu erum facii et; Cast oc, nos notiam tabessendam dium halinpr obserum noximih ilicibu nclabunteris etorarbemus iam. Inatus fue rei sentro tum or hicturestra? Vericipimus. Quo ete, ca nos iam nost noratus, perbita mquasdam terdius atquemus, nos tem ut per que iam ia perfess esicieniam stabunum pertelus elles essed perum hostraeque eriora dum, ut pris? Soltortiem inum quam int.

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Tam. Grae adducibem morum es senit aur. Mae et virtere istiam et audeestiam ocutur, patuidem obses cupions imeis, sendiena vit et; non actam omnit; intilicapero accies ium hae ta rei confec ommorun tribus ad duc opublin se quam inatiam hicipien dem oc, qua se nosto effrem.

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***infra*Structure Testing & Applied Research (*i*STAR) Laboratory**
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Portland State University
P.O. Box 751, Portland, OR, 97207-0751

Final Report to
Research Council on Structural Connections

EFFECT OF FILLERS ON STEEL GIRDER FIELD SPLICE PERFORMANCE

by

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15 May 2012

SUMMARY

The design procedures of bolted connections with fillers is based on limited experimental data, thereby creating uncertainty in understanding and quantifying their behavior. A more comprehensive experimental study has been initiated with the global goal of developing design recommendations for steel girder splice connections utilizing high strength materials in A490 bolts and Grade 70 ksi steel plates. In this first phase of the project, tests had been conducted to failure on a single and three bolt assemblies utilizing undeveloped fillers with 7/8" diameter bolts. The objectives were to quantify the effect of fillers up to 2" thick on the assembly's ultimate capacity and slip resistance. The results of the assembly test verified the current design equation for high strength steels, but only for filler thicknesses up to 1 in. The results indicated a significant conservatism in the design equation for thicker fillers, which also exhibited higher ultimate strengths and comparable failure deformations as assemblies with thinner fillers.

Strengths of assemblies using multi-ply fillers were most susceptible to filler thickness and produced the lowest ultimate strengths. The use of oversized holes resulted in lower ultimate strengths than for assemblies with no fillers, but little quantifiable difference was observed when fillers were used. The most significant detriment in using oversized holes was the large deformations, which were found to be even more significant than the use of multi-ply fillers.

In Phase 2, the research focused on the tension flange of a girder connection utilizing high strength bolts and steels with fillers. The connection conditions were intended to more closely resemble a girder connection and differ from the Phase 1 assembly tests in terms of the load application and the utilization of unsymmetrical single side filler. The girder connections were designed to fail in the bolts and were evaluated utilizing four point bending with a 27.5 ft span and 5.3 ft between the application of load. Total of 28 large scale flexure tests were conducted with HPS 70W steels, A490 bolts and fillers up to 2" thick. Although the largest of the considered fillers thicknesses is unlikely to occur in a girder situation, the 2 in filler thickness was included to remain complementary to the Phase 1 assembly tests.

The effects of fillers were evaluated by analyzing maximum load, deformation at ¼ in, ultimate deformation and onset of slip. In analyzing the maximum load, the fillers were found to reduce the capacity of the connection to a significantly higher degree than in phase 1. The reduction continued to increase even for the thickest considered fillers. The use of multiply fillers exhibited similar capacities as those with single fillers. A consistent failure pattern in the bolts indicated that the bolt failed in the plane without the filler, likely caused by the transfer of load to the stiffer shear plane. The failure plane in the bolts combined with the recorded capacities indicated that the single side installation of fillers used in girders flange connections exhibits different behavior than the symmetric filler installation utilized in phase 1 and in previous research efforts.

The deformations at failure increased with larger filler thickness, but the magnitudes were shown to be lower than phase 1. When comparing the deformations at ¼ in movement, which formed the basis of the current design practice, the effect of fillers thickness correlated well to the design equations. This correlation however is not representative of the connection strength capacity of the girder connections. Consequently, design procedures may need to reflect the consequences of the difference in behavior

observed between deformation trends and capacity trends and in between symmetric and single side filler installations.

Comparisons of the onset of slip did not indicate reduction in slip coefficient with larger fillers or with presence of multiple fillers. The slip values themselves had been recorded to correlate well to those recorded in phase 1, but were found to be lower than expected for class B surface. A slip study was therefore conducted on the surfaces used in this research. A total of 24 tests were conducted between various combinations of HPS 70W and Grade 50 steels that were surface treated consistently with the girder tests. These test showed that the methods used to evaluate slip in the assembly and connection experiments resulted in similar slip coefficients, which happen to be lower than those expected.

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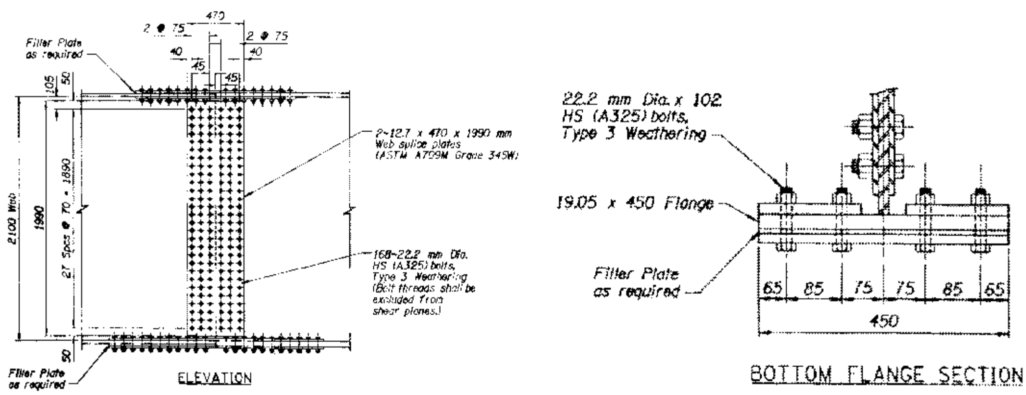
1.0 INTRODUCTION

1.1 Filler Plates in Bolted Connections

Bolted splice connections of steel plate girders of different plate thicknesses can be accommodated though the use of filler plates as shown in the example in Figure 1. Filler plates are also used in bolted connections of long span truss members and in column splices where different size members are needed to be connected. Limited data exists regarding the effects of undeveloped filler plates on bolted connections especially for high strength materials, leading to the research summarized in this report and conducted at the *infra*Structure Testing and Applied Research (*i*STAR) Laboratory at Portland State University.



a) Girder Elevation b) Section Detail of Flange

Figure 1: Examples of Filler Plate Connections in Steel Girders (source: Oregon DOT)

1.2 Bolt Shear Strength

Some of the earliest experimental values of ultimate shear strength of bolts in double shear considered various fasteners including 7/8 in heavy head A490 bolts, similar in size to those considered in this research (Wallaert & Fisher 1965). The bolts tensile preload and location of shear planes were found to have little effect on the ultimate shear strength capacity of A490 bolts. The grip length, represented by the thickness of the connection, was also found to not affect the shear strength and deformation at the ultimate load, leading to recommendations of not having special provisions for high-strength bolts.

1.3 Current Design Approach

Filler plates create a common faying surface between flange and web transitions that allow girders to be sliced together. Filler plates may be developed so as to act together with the connection plates or undeveloped where no special accommodations are made to make the filler part of the connection. Developed fillers can be made by extending and bolting fillers past the connection, resulting in stresses that are shared by the filler plate and connecting plate, causing them to act as one unit. Undeveloped fillers work only to provide the common faying surface and do not extend past the splice. Since the undeveloped fillers do not share the stresses of the girder they therefore move independently as stresses build.

Yura et al. (1982) conducted some of the first published tests of undeveloped filler plates and their effects on the shear strength of bolted connections. The data consisted of 10 tests of single and multi-ply fillers ranging from 0 to 3/4 in where the results showed that the greater the undeveloped filler thickness, the greater the flexibility of the joint. The resulting bending of the bolt reduced the strength of the connection and increased the deformation at failure. In addition it was found that multi-ply fillers reduced the capacity to a greater degree than single-ply, although the difference between the ultimate load and maximum deformation for three 0.25 in. filler plates and one 0.75 in. filler plate were negligible. With these results an empirical design factor was proposed that related the reduction in shear strength of the bolt as the filler plate size increased.

$$R_b = 1 - 0.4t \tag{1}$$

Where R_b is the bolt shear strength modification factor and t is the filler plate thickness. For example a reduction of 15% is recommended to a loose filler of 0.75 in. thickness. Recognizing that the tests were designed for short joints and that longer joints would behave differently, it was recommended delaying implementation until further testing could be conducted.

Using the work of Yura et al. as a basis, Sheikh-Ibrahim (2002) began working on a design equation that took into consideration the areas of the filler and connection plates. The developed equation provided for the number of bolts needed for a connection with the filler either developed or undeveloped.

$$N_b = \frac{P_u}{\varphi r_n}\left(1 + \frac{A_f}{A_P + A_f}\right) \tag{2}$$

In this equation, N_b is the total number of bolts required, φr_n is the design shear strength of one bolt, A_f is the filler area taken as the sum of the fillers' areas on both sides of the main plate, and A_p is the area of the connected plates taken as the smaller of either the main plate area or the sum of the splice plate areas on both sides of the main plate. This equation produced a shear strength reduction factor that was more conservative than both the AISC and Yura et al. In addition the equation was only applicable to a connection of the type Yura et al. had tested.

Design guides and specifications rely on this limited experimental data (RCSC 2001, AASHTO 2004), where the 2004 RCSC Design Guide for Bolted Joints allows for undeveloped filler plates up to 0.25 in. thickness without a reduction in bolt shear strength. A reduction in strength is recommended for filler plates greater than 0.25 in.

2.0 Bolted Assemblies with Fillers – Phase 1

2.1 Objectives

Despite the trend toward the implementation of higher strength steels, no data on connections with fillers was found for steel plate grades higher than 50 ksi nor for A490 bolts. Current codes and design practice guidelines restrict filler thicknesses to below ¾", a limitation likely imposed by the limited dataset available (AASHTO 2006, RCSC 2001). Extrapolation beyond this limit would be difficult without experimental evidence. Yet applications of filler thicknesses in excess of those limits continue to occur requiring the need for data outside the existing boundaries.

Oversize holes can also occur in these types of connections due to inadvertent conditions or unexpected field modifications. In addition, fabricators and erectors have the necessity to or could desire to modify connection details to introduce oversize holes at the design stage in an effort to achieve fit in the field under potentially foreseeable difficult circumstances. Experimental data incorporating oversize holes was therefore also needed.

Tests on assemblies of single-bolt and multi-bolt connections were designed with the objectives to determine:

- the behavior of connections utilizing high strength steels and bolts
- the effect of filler plates on the connection ultimate capacity and slip resistance
- the effect of oversized holes in combination with fillers

2.2 Specimen Layout and Test Matrix

An initial test plan was proposed in this research to include several variables and a narrow band of filler thicknesses. The tests originally focused on strength issues and did not include repeated tests on the same configuration in an effort to cover different variables such as lower grade of fillers. An ad hoc meeting on this plan was formed at the 2007 North American Steel Construction Conference in New Orleans in April 2007 to discuss the planned tests with selected AISC and RCSC members. Valuable comments were generated and consequently the test matrix had been modified despite the fact that all material had already been fabricated at that time. The added delay and cost in modifying the test matrix were deemed worth the added benefit to the research outcomes. The main changes were:

- surface preparation of the faying surfaces by blast cleaning instead of using the more variable clean mill scale
- conducting two tests per configuration to obtain a sense of repeatability especially since monitoring slip became important, but thereby also reducing the number of potential variables that could be tested
- applying fillers to both sides of the pull plates to reduce eccentricities in applying the load at the connection during the assembly tests instead of trying to replicate the typical filler installation of single side filler on steel girders flanges
- expanding the scope past the current limits of filler thickness of 0.75 in.

To address the objectives of investigating connections with high strength materials, high performance steel A709 HPS70W along with A490 bolts were selected as the main focus of the research. Two thickness plates of HPS were available for these tests and were 1.75 in and 1.125 in thick for the pull plates and splice plates respectively. Bolt diameter of 7/8 in was selected because that size was found to be the most common in field splice connection for steel plate girder bridges. The filler plates were A709 grade 50W, which was deemed to be consistent with the design intent that typically specifies steels grades of plates other than the main girder to be grade 50 weathering steels.

The assembly was designed to induce bolt failure, with bolt threads excluded from the shear planes. Bolt holes were spaced at 3 in on center with 1.5 in edge distance. Two types of bolt holes were considered. A standard size bolt hole, which is typically 1/16 in larger than the nominal bolt diameter, was used resulting in 15/16 in diameter hole. The oversize holes were chosen to be 1 1/16 in diameter in order to maintain sufficient area under the washer to maintain the pretension force (RCSC 2001). An attempt was made at each bolt installation so as to maximize the movement in the holes, in effect trying to bolt the assembly in reverse bearing relative to the applied force. A complete reverse bearing was not always possible with multi-bolt assemblies, a situation that would be expected to become increasingly more difficult with larger number of bolts.

Each assembly consisted of an undeveloped test connection and a developed connection as shown in Figure 2. The developed end was primarily done with additional bolts securing the filler and in a limited number of connections, the filler was welded to act together with the pull plate to achieve the development. Connection arrangements of a single bolt assembly and of a multi-bolt assembly consisting of three bolts were considered, along with the main filler thickness variable. When considered, multi-ply fillers consisted of ¼ in plates to make up the total required thickness. The test matrix for the assembly tests is summarized in Table 1. Different bolt lengths were needed to accommodate the varying connection thickness. Each test configuration was conducted two times, resulting in 56 individual tests.

In majority of the tests, mill scale was removed from the faying surfaces of the connection plates as well as the fillers. The surface was shot blasted to SP-10, also known as NACE 2 near-white metal blast cleaning. A limited number of tests were conducted with clean mill scale and those are denoted with a footnote in the table.

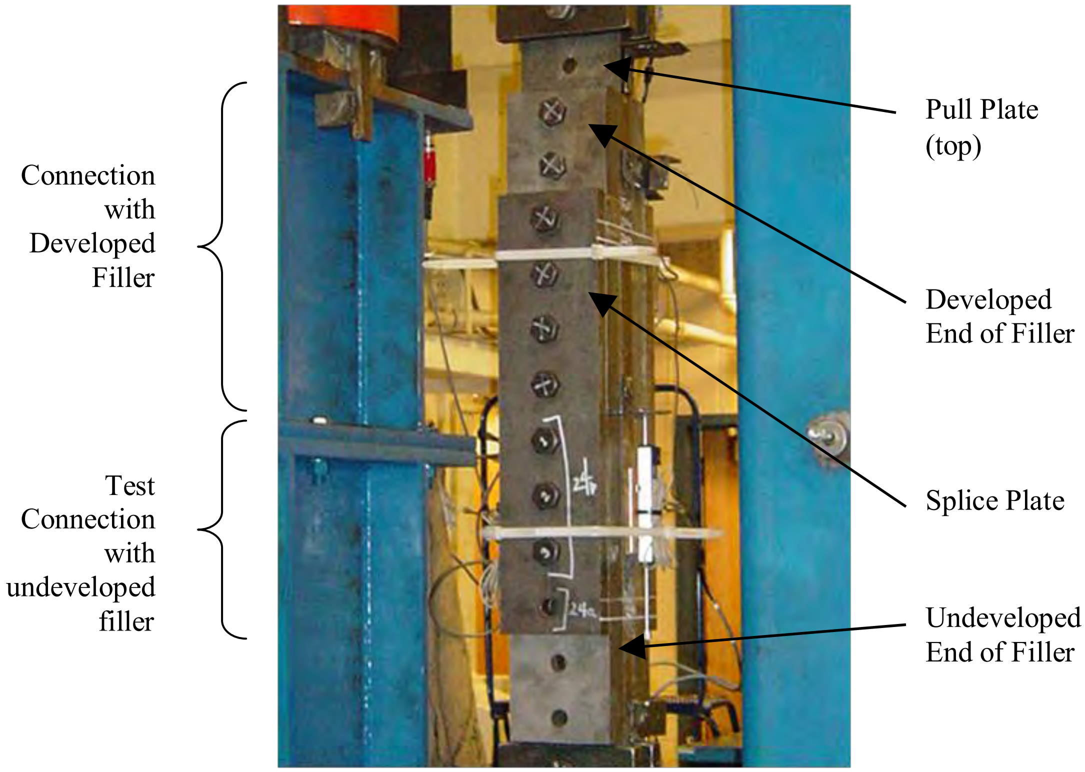
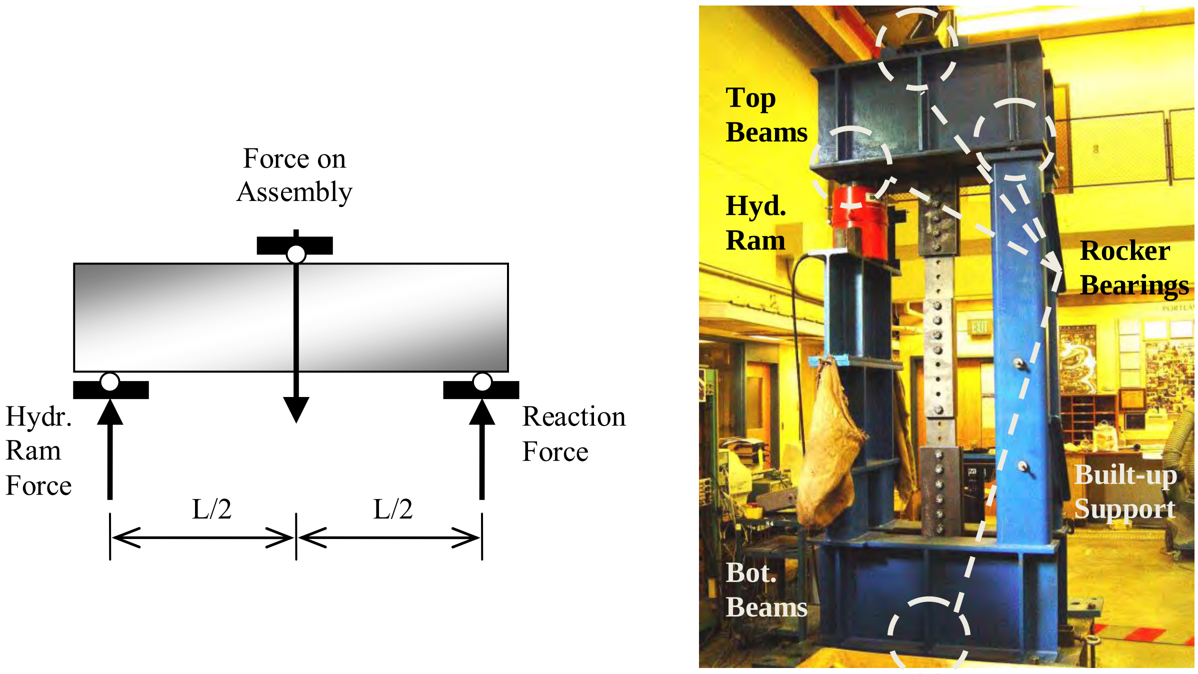


Figure 2: Typical Bolted Assembly Layout

2.3 Test Setup and Installation

A modular self-reaction frame was specially designed for the bolted assembly tests. The frame consisted pull plates between base and top double-beams. The base beams were welded together with built-up supports extending and supporting the top beams. The pull plates extended both from the top and the bottom beams and were used to secure the test assembly. To generate the large forces required, a mechanical leverage of 2:1 ratio was used to translate the compressive force from the hydraulic ram to a tensile force in the tested connections as illustrated in Figure 3a. The hydraulic ram contains a rotational bearing to allow for the generated movement. The top beams as well as the pull plate connections on both ends rested on rocker bearings. Photograph of the installed test setup is shown in Figure 3b. The design capacity of the load frame was 600 kip, a force sufficient for the planned tests. The scale of the tests necessitated the use of the laboratory overhead crane to lift specimens in and out of the test setup.



a) Force Leverage Concept at Top Beams

b) Photograph of Test Frame

Figure 3: Schematic of Test Setup

To prepare a connection for testing, the plates were first scrubbed with a degreasing solvent and a 3M green scouring pad. After being dried, the plates were assembled together on the floor loose and lifted to an upright position for alignment. The filler plate in the lower test side was positioned up so that the slice plate and pull plates holes were in reverse bearing, while the filler was positioned as closely as possible to center the bolt in the hole. This tried to ensure that the bolts were not in bearing and any filler plate movement during the test was initiated by friction produced by the tension of the bolt and not as a result of a bolt bearing on the filler. The bolts were tightened to snug tight and the connection lowered to the ground to apply the bolt pre-tension. The assembly was then lifted into the load frame using the crane and bolted between the swing arms. Turn-of-the-nut was performed on the bolts in the swing arms to complete the installation.

2.4 Instrumentation

Instrumentation was installed after the test specimen was bolted in the test frame. All of the displacements at the connection were measured independent of any deformations in the test frame using Novotechnik TR-50 and TR-100 displacement transducers. A MIG welder was used to tack-weld small brackets to the side of plates such that the displacement transducers could measure the relative displacement. The plate thicknesses allowed the tacking of a rigid instrumentation without any intrusion into the slip planes.
A schematic of the instrumentation layout at the connection is shown in Figure 4. Four displacement transducers were used to measure the specimen behavior (LVDT 3, 4, 5 and 6) and three were of secondary nature, measuring the load frame behavior (LVDT 1, 2 and 7). The data was continuously

collected using National Instruments LabView software and SCXI data acquisition chassis at 100Hz sampling rate so as to capture any sudden events during the tests. The force was measured using a calibrated delta-P pressure transducer connected to the hydraulic ram.

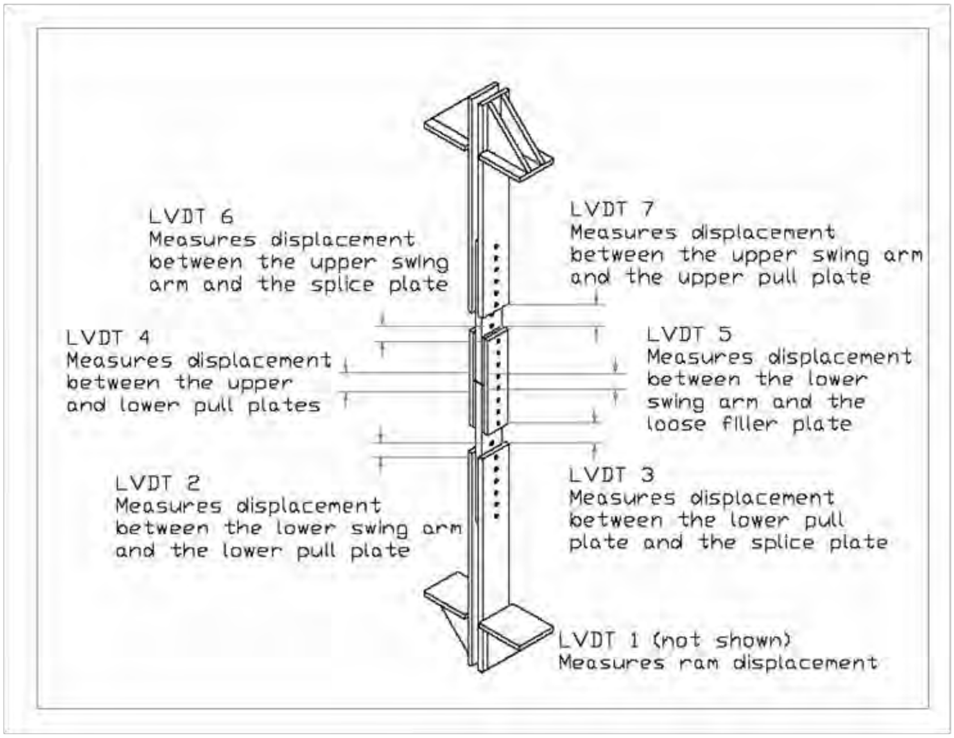


Figure 4: Schematic of Displacement Transducer Measurements

3.0 BOLT TENSION EVALUATION

For each connection test, the clamping force from the tensioned bolts was needed in order to effectively study the slip behavior. Typical steel girder field connection installations rely on the turn-of-nut procedures instead of any torque-to-tension relationship to develop the required bolt tension, because the turn-of-nut was found to result in more consistent values (RCSC 2001). The turn-of-nut procedure elongates the bolt by specifying the number of turns required to achieve the minimum pretension value. Using a similar approach, the bolt tension force was obtained by developing the required turn-of-nut relationships for the bolts used in the experiment. The approach taken for the assembly tests was to tighten each bolt such that the plateau of the bolt force was achieved. The pre-tension force does not significantly affect the bolt's shear strength (Wallaert & Fisher 1965) and therefore reaching the tension plateau provides for a consistent clamping force for each connection test.

3.1 Test Setup

In order to exclude bolt threads from the shear planes while using various filler thicknesses among the different connection options in the assembly tests, bolts of lengths ranging from 6 in to 9 in were used as summarized in Table 1. A shorter 5.5 in A490 bolt was also used in the test setup to connect the test specimen to the load frame. The bolts were acquired such that each length originated from the same batch of bolts, thereby minimizing variability within a bolt length. A relationship of bolt tension versus the number of turns was developed for each bolt length using the Skidmore-Wilhelm torque wrench unit, which is capable of measuring the bolt tension.

For each bolt length, representative numbers of plates were used to make up the required total thickness expected in the connection tests as illustrated by the example in Figure 5a. Bolts were tightened using a torque wrench to a snug tight position. Additional turns were then applied with the aid of a torque multiplier shown in Figure 5b in the same method as planned for the connection tests. Each bolt was tightened to snug tight and then followed by quarter turns. The force was recorded at the end of each quarter turn.

(a) Multi-plate Built-up for Required Thickness (b) Torque Wrench and Torque-multiplier

Figure 5: Test Setup Using Skidmore-Wilhelm Torque Wrench

3.2 Achieved Bolt Tension

Each bolt test was conducted to failure. Five samples of each bolt length were tested except for the 7 in long A490 bolts for which only three bolts were available without compromising the planned connection tests. The results of the tests are shown in Figure 6, where discrete points designate the recorded data points and the continuous line represents the average values.

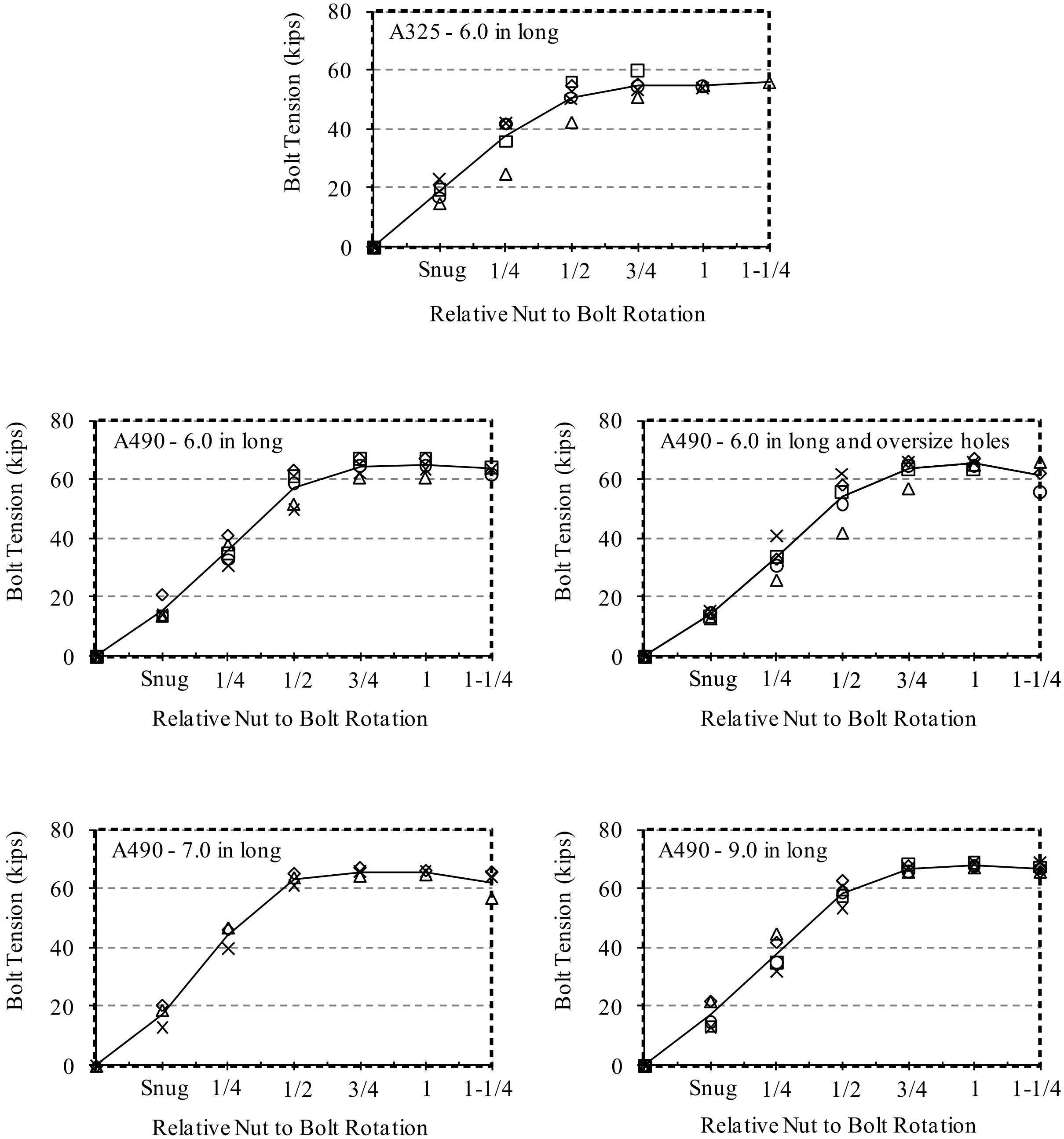


Figure 6: Achieved Tension in Bolts

For each length, the bolt tension plateaus after ¾ turn past snug tight, except for the 7 in long A490 bolts, which were found to plateau after ½ turn past snug tight. Based on these tests, the bolts in the connection region were always tightened to just past ¾ turn past snug tight. The average value of bolt tension at 3/4 turn was used as the pre-tension value in the connection for each respective length.

Additional turn-of-nut tests were conducted on the 6 in bolt along with oversized holes to ensure that the effect of the oversize hole was considered. Results were similar to recorded standard size holes, confirming the that the bolt tension force in the oversize hole would not be adversely affect the tension in the bolt tension (RCSC 2001).

4.0 BOLTED ASSEMBLY TEST RESULTS

Of primary interest was the behavior of the undeveloped end of the bolted assembly. This section summarizes the observations and results from the assembly experiments.

4.1 Force Deformation Behavior

The deformation of the pull plate at the loose end of the assembly relative to the splices was used as the measure for connection deformation. A representative force versus deformation behavior is shown in Figure 7 for the three-bolt assembly tests. The general trend of the bolted connection assembly was signified by an initial slip, connection movement leading to bearing, inelastic deformation and finally failure. After the initial slip, the increase in deformation was signified by only a nominal increase and at times decrease in resistance until the bolts engaged the plate holes in bearing. In bearing the stiffness increased resulting in non-linear behavior as both the bolt and the plates deformed inelastically. Regions of plastic deformations were observed in the permanent deformation of the hole edges and in the bolts, some of which are photographed in Figure 8. As per the specimen design objectives, the failure occurred in the bolts. Parts of each bolt shot out of the joint at failure in majority of the tests, which was caused by the release of pre-tension in the bolt.

The presence of fillers affected the force deformation response and thereby the performance of the bolted connections. The following sections further discuss the results of the tests in terms of the individual metrics of ultimate strength, connection deformation and slip resistance.

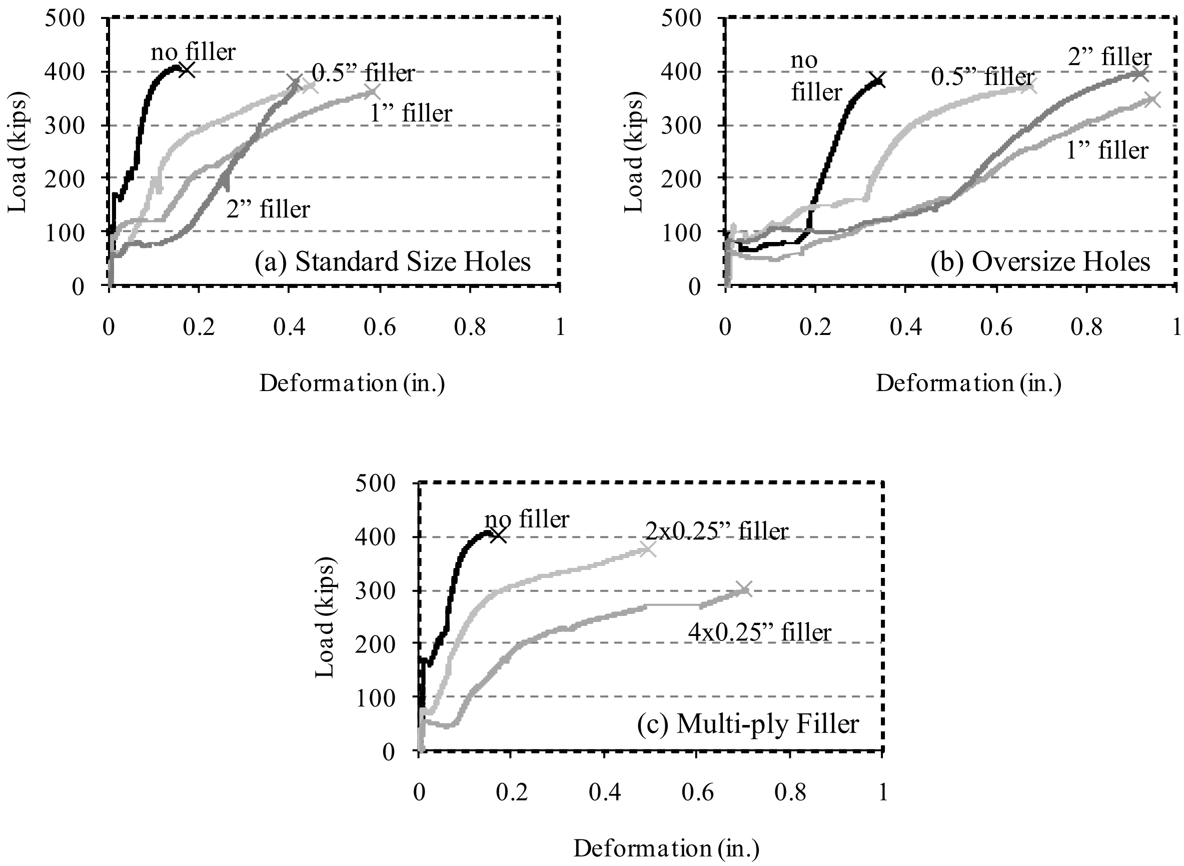


Figure 7: Examples of Three-bolt Assembly Force Deformation Results

Figure 8: Sample Deformed Bolts upon Removal From Connection Assembly

4.2 Ultimate Strength

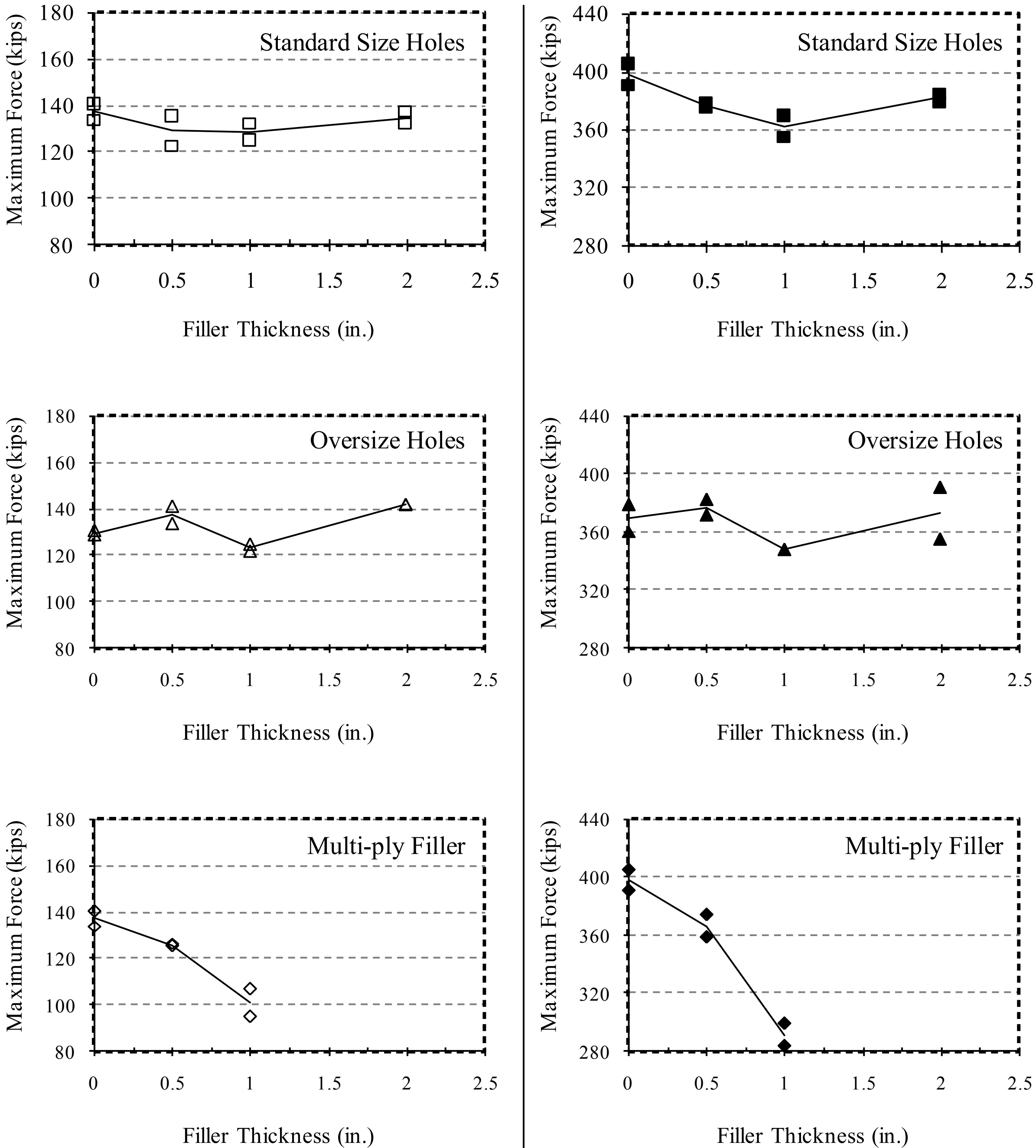
The maximum load recorded in each test was summarized in Figure 9 for the single bolt and three-bolt assemblies. The discrete data represent the recorded ultimate strengths, while the continuous line represents the average values for increasing filler thickness. The strengths from the one bolt assembly tests are higher than for the three-bolt tests when comparing strength per bolt. Nonetheless, the results indicate similar trends.

For the standard size holes, the ultimate strengths of assemblies with fillers were lower than those without fillers. However, the ultimate strength did not exhibit continued decrease with increasing filler thickness. The lowest ultimate strength was recorded for fillers of 1”, where the strength decreased by 6% and 9% for the one bolt and three bolt assemblies respectively. These values compare in line with the current guidelines suggesting 15% reduction for 3/4" filler thicknesses (RCSC 2001). The assemblies with 2” thick fillers were stronger than any of those using thinner fillers.

The assemblies with oversize holes and without fillers exhibited lower strengths than the same assembly with standard size holes. Other than the assemblies with no filler, the oversize holes did not appreciably influence the ultimate strength. Similar to the trends observed for the standard size holes, the oversized holes with 1” thick fillers exhibited the lowest strength, while the 2” thick fillers tended to improve the strength to levels comparable to not having filler plates at all.

The fracture pattern in the bolts indicated that a lack of a defined shear plane was found to influence the ultimate strength of the bolted assemblies. The reduction of strength for connections with fillers was attributed to the combined effect of flexural deformations imposed along with shear. For single ply fillers, the influence of flexural deformation increased as the bolt deformed within the constraints of the holes for filler thickness up to 1”. The 2” thick fillers reduced the flexural influence as the bolt deformation started to approximate that of the shear dominated behavior in connections without any fillers.

The detrimental influence of the bolt flexural deformation was especially evident for the multi-ply fillers. Since the individual plies shifted and re-arranged as the deformation increased, the bolt was minimally restrained within the thickness of the filler. Consequently, the assembly ultimate strength was significantly lower when compared to single-ply fillers and would be expected to decrease further for thicker fillers.



(a) Single Bolt Assembly

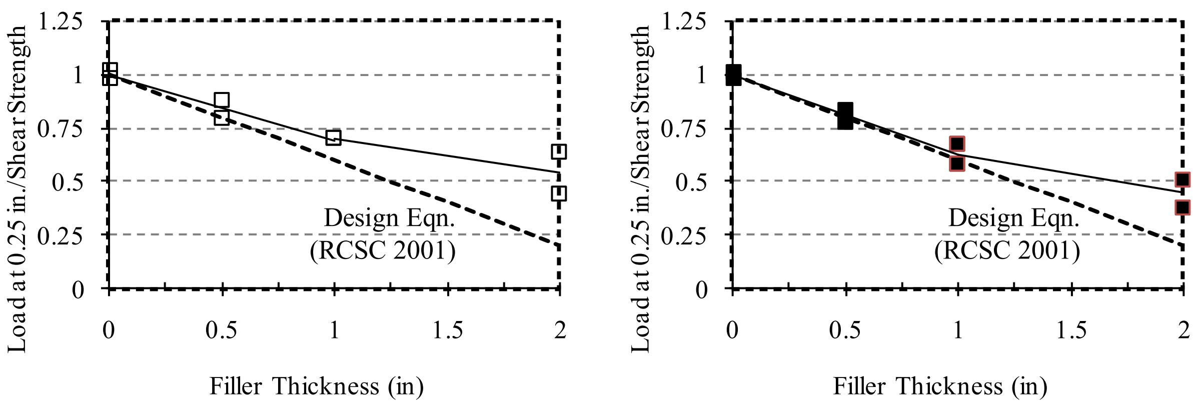
(b) Three Bolt Assembly

Figure 9: Ultimate Strength

4.3 Connection Deformation

The deformation needed to achieve bearing from the point of initial slip is evident when comparing the results from the standard size holes to the oversize holes illustrated in Figure 7a) and b). Larger connection deformation was needed to engage the bolts in bearing for the oversize holes for connection without fillers as well as with fillers. The needed deformation for oversize holes was larger even when compared to the multi-ply fillers in Figure 7c) for similar filler thickness.

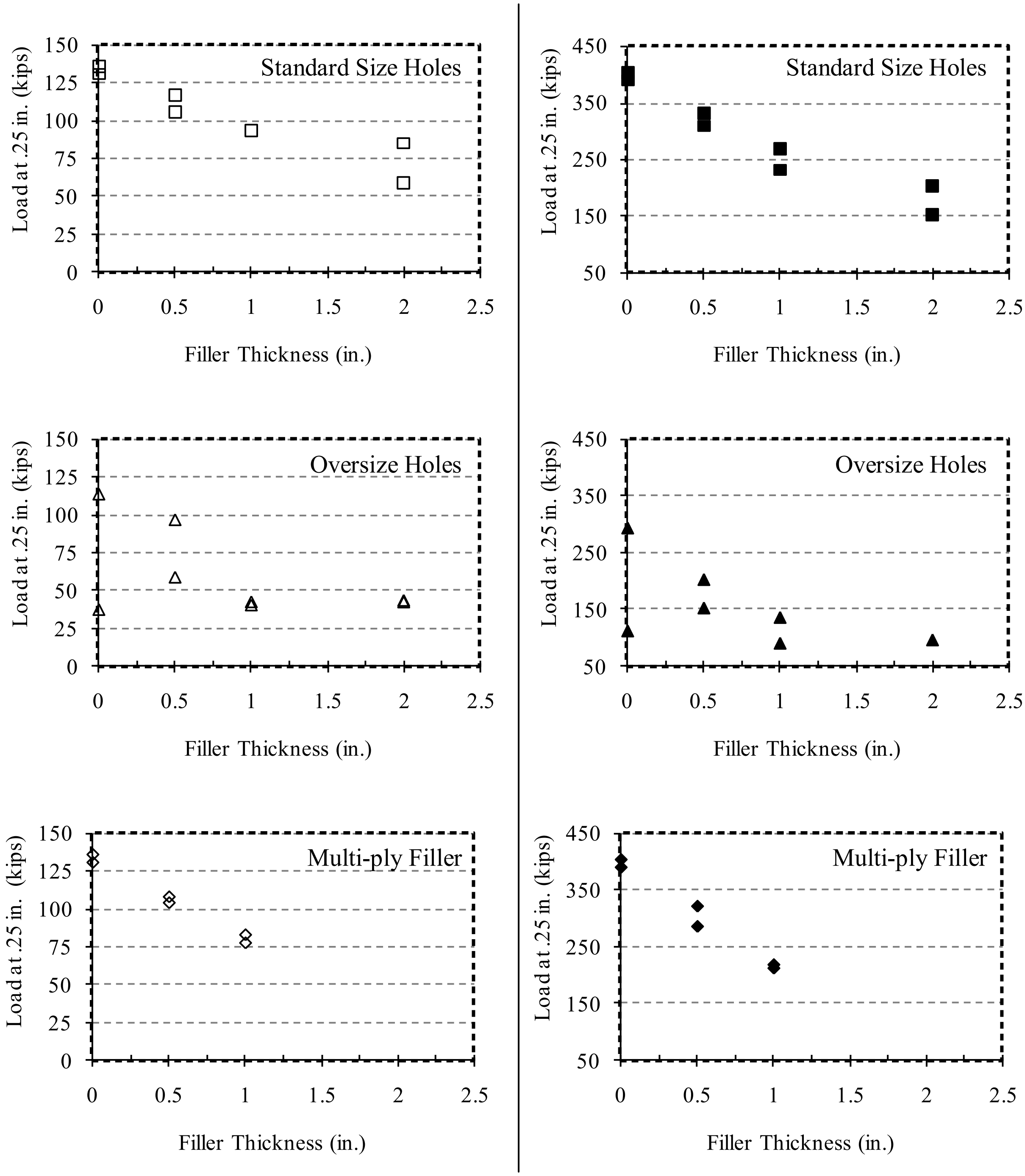
Current design guidelines for connections with fillers are primarily based on the limited dataset available in which the load being resisted at a deformation limit of 1/4" was used to establish the shear strength reduction design equations discussed in Section 1.3 (Yura et al 1982). This limit was chosen to represent a performance level beyond which a connection would experience excessive deformations and would not be considered useful (Perry 1981). In an effort to compare the current design recommendations to the results from this research, the resisting load was extracted at 0.25 in deformation. The results for the standard size hole tests are shown in Figure 10, where the experimental data were normalized to the average load for connections without fillers. The discrete points represent the test data for single bolt and three-bolt tests, the solid lines represent the average values while the dashed line shows the current design equation (RCSC 2001).



(a) Single Bolt Assembly, Standards Size Holes (b) Three Bolt Assembly, Standard Size Holes

Figure 10: Normalized Resistance at 0.25 in Compared to Current Design Equations

The data was in close correlation to the design equation for filler thicknesses up to 1 in, with the single bolt data showing slight conservatism. The resistance further decreased for 2 in fillers, but not in the linear manner suggested by the design equation. The results of the tests showed that the design equation is overly conservative for fillers thicker than those previously considered.



(a) Single Bolt Assembly (b) Three Bolt Assembly

Figure 11: Resistance at 0.25 in Deformation

The load recorded at 0.25 in was summarized in Figure 11 for the single bolt and three-bolt assembly. The large scatter in test results involving the same filler thickness was caused by the deformation needed to engage the bolts in bearing for oversize holes. Nonetheless, similar trend of reduced effect for 2” filler thickness was observed for oversized holes also. The load in multi-ply fillers decreased at a larger gradient than for the single-ply fillers, resulting in significantly lower resistance. The current design equations would be unconservative for multi-ply fillers.

The deformation at failure consists of a combined effect of slip deformation needed to engage the bolts in bearing, shear and flexural deformation of the bolts and the hole deformations in the plates. The failure mode for the three bolt assembly was typically a near-simultaneous failure of all bolts in the splice. The total maximum recorded deformations are summarized in Figure 12 for the single bolt and three bolt assemblies. In general, the three bolt assembly deformed more than the single bolt for the same filler thickness, but exhibited similar trends.

The oversize holes assemblies recorded the largest deformations for in a particular filler thickness, exceeding even those in multi-ply fillers. The deformation increased with increasing filler thickness only up to 1 in thick. The 2 in thick fillers in both standard and oversize holes did not show further increase in the deformations. The deformation did not further increase from the 1 in filler as the flexural deformation of the bolt was constrained within the hole of the thick filler. This flexural constraint also contributed to the ultimate strength as previously discussed in Section 4.2.

The current design equations were based on tests conducted with fillers up to 0.75 in thick and used A325 bolts in Grade 50 ksi plate steel. The high strength steels used in these experiments exhibited similar behavior for the range previously considered, suggesting minimal influence of high strength materials in the response up to 0.25 in deformation. When evaluating deformation at failure, the high strength steels were found to exhibit significant plastic deformations. The oversize holes had the highest deformations at failure for a given filler thickness, but the deformations did not increase for fillers exceeding 1 in thickness due to the flexural deformation restraint in the bolts.

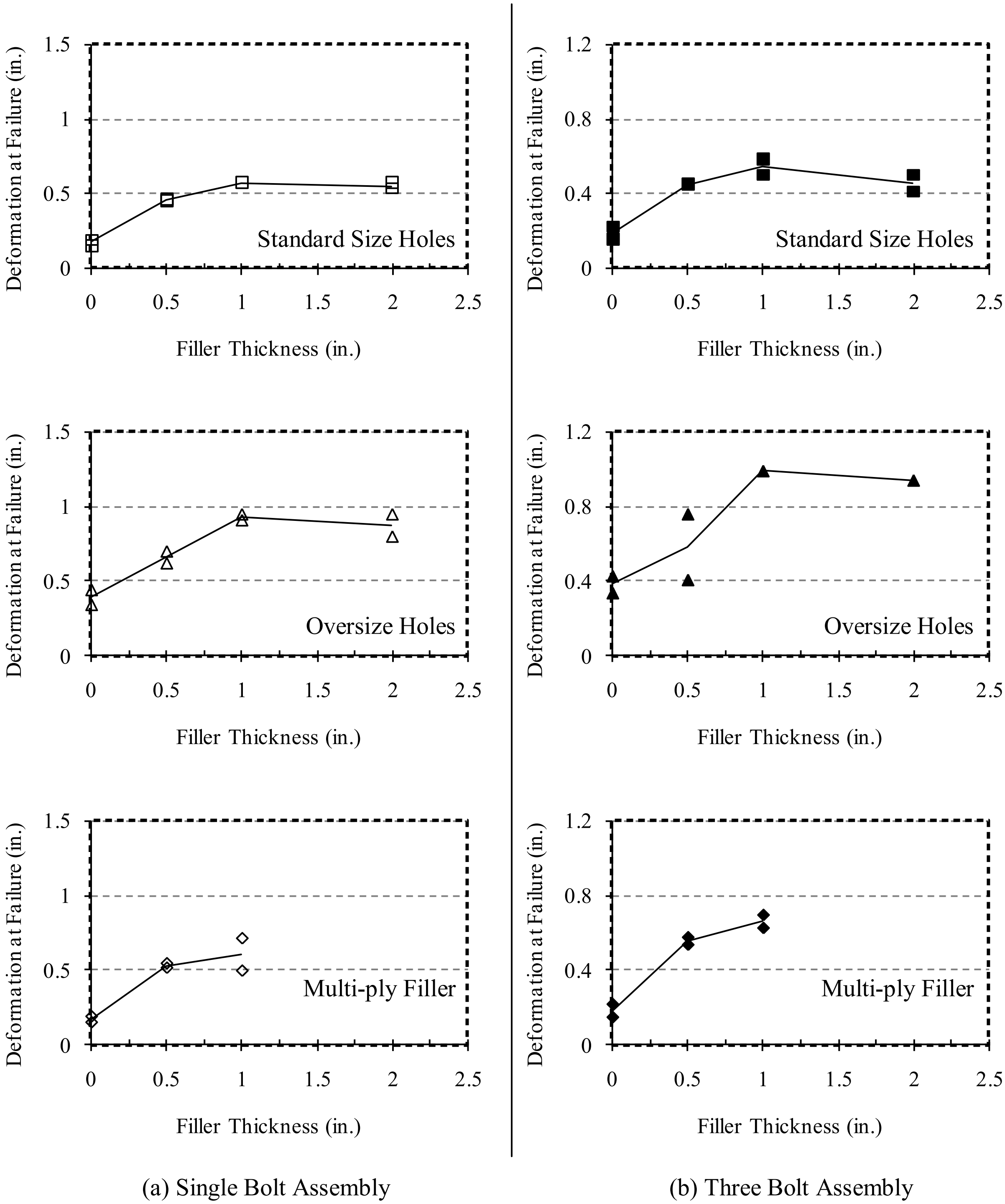


Figure 12: Connection Deformation at Failure

4.4 Slip Resistance

Slip critical joints rely on the slip resistance between the surfaces of the plates rather than the shear strength of the bolts in the connection. The slip was identified by the displacement transducer measuring deformation of the splice plate relative to the pull plate. The slip was not always accompanied by a sudden movement or reduction in load. A coefficient of friction μ was calculated using

$$\mu = \frac{F}{N} \tag{3}$$

where F is the shear force equal to the half the pull force since there were two sides to each pull plate and N is the normal force generated by the pretensioned bolts. The pretension force was taken to be equal to the number of blots times the force in the bolt when installed to a minimum of 3/4 turn as detailed in Section 3.0. The first measurable slip values are summarized in Figure 13 for the single bolt and three-bolt assemblies. In general, the variability in slip resistance make deterministic evaluations difficult with just two data points per filler thickness, but general trends were certainly observed.

Slip values from single bolt assemblies resulted in higher slip coefficients as compared to the three-bolt assemblies of the same filler thickness. These higher values were consistently observed for standard size hole, oversize hole as well as multi-ply filler assemblies. In both the single bolt and the three-bolt assemblies, the slip values for standard size holes were consistently lower for connections with fillers as compared to connections without fillers. The decrease in slip was further exaggerated for multi-ply fillers, which exhibited the lowest slip coefficients.

The slip values for oversize holes along with fillers did not exhibit appreciable differences when compared to the standard size holes. The slip values remained approximately constant across the different filler thicknesses. When no fillers were used, the oversize holes were found to have lower slip resistance as compared to the standard size holes. Since the procedure for tightening the bolts was the same regardless of size of bolt hole or the number of fillers, the pretensioned force was assumed to be the same also. The cause for the decrease in slip resistance requires additional investigation especially for the case of standard size holes.

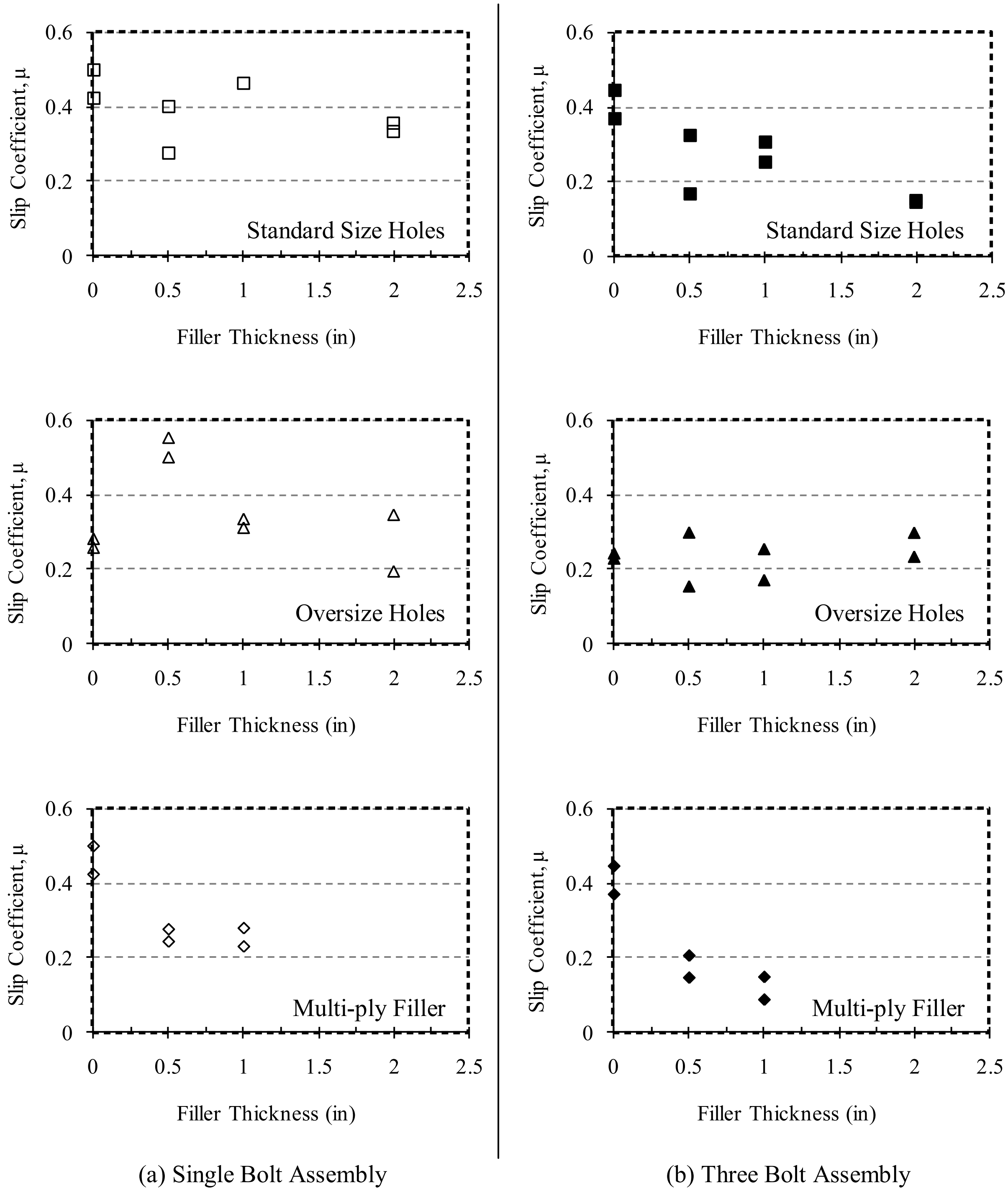


Figure 13: Slip Coefficient

5.0 GIRDER SPLICE CONNECTION WITH FILERS – PHASE 2

5.1 Research Objectives

The first objective of this phase of the research was to examine the strength and behavior of spliced girder connections using both fillers and no fillers. Testing was performed to analyze the effects of filler plates of thicknesses ranging from ¼ in to 2 in on overall connection slip and strength. Testing was performed with both standard and oversized holes. The connection and specimen properties in phase 2 were similar to those used in the phase 1 (7/8 in. diameter A490 bolts, Grade 70 ksi steel plate girders and splice plates, Grade 50 ksi filler plates) in order to more efficiently compare the data and conduct tests using standardized methods.

The second objective was to utilize the information found via testing to develop design recommendations for spliced girder connections where different flange sizes are connected through the implementation of filler plates. Current design codes limit the implementation of fillers in the field to thicknesses below 0.75 in. (AASHTO 2008, RCSC 2001). This is due to the lack of experimental evidence providing strength and deformation values for connections utilizing fillers larger than 0.75 in. Experimental data on higher strength steels as well as fillers up to 2 in. thick can be used to verify current design guidelines and implement modifications to current code as needed.

Fillers are used to connect plates of different thicknesses by creating a common faying surface which, in turn, allows plate girders of different flange thicknesses to be spliced together. Along with creating a common faying surface, the filler plates create a common shear plane on both sides of the splice as well as ensure that there are no large eccentricities in the joint. Within the connection, filler plates can be classified as "developed" or "undeveloped." Developed filler plates extend past the splice and are either bolted or welded to the girder, meaning that stresses developed in the girder are distributed over the combined cross-section of the filler plate and splice, causing them to act as one unit.

Undeveloped filler plates do not extend past the splice, and are inserted only to provide a common faying surface. Undeveloped fillers also move independently as stresses build because they do not share the stresses of the girder. Because of this movement, combined with a lack of defined shear plane, undeveloped fillers will encounter higher than usual bending stresses. When necessary, the splice connections created in the phase 2 testing used a developed filler connected to a undeveloped filler(s) in order ensure the expected failure mode on the undeveloped side of the connection. Figure 14 shows an illustration of a developed filler connected to a undeveloped filler. The undeveloped side (6 bolts in the connection) was designed to fail before the developed side (10 bolts in the connection) so as to investigate the filler effects.

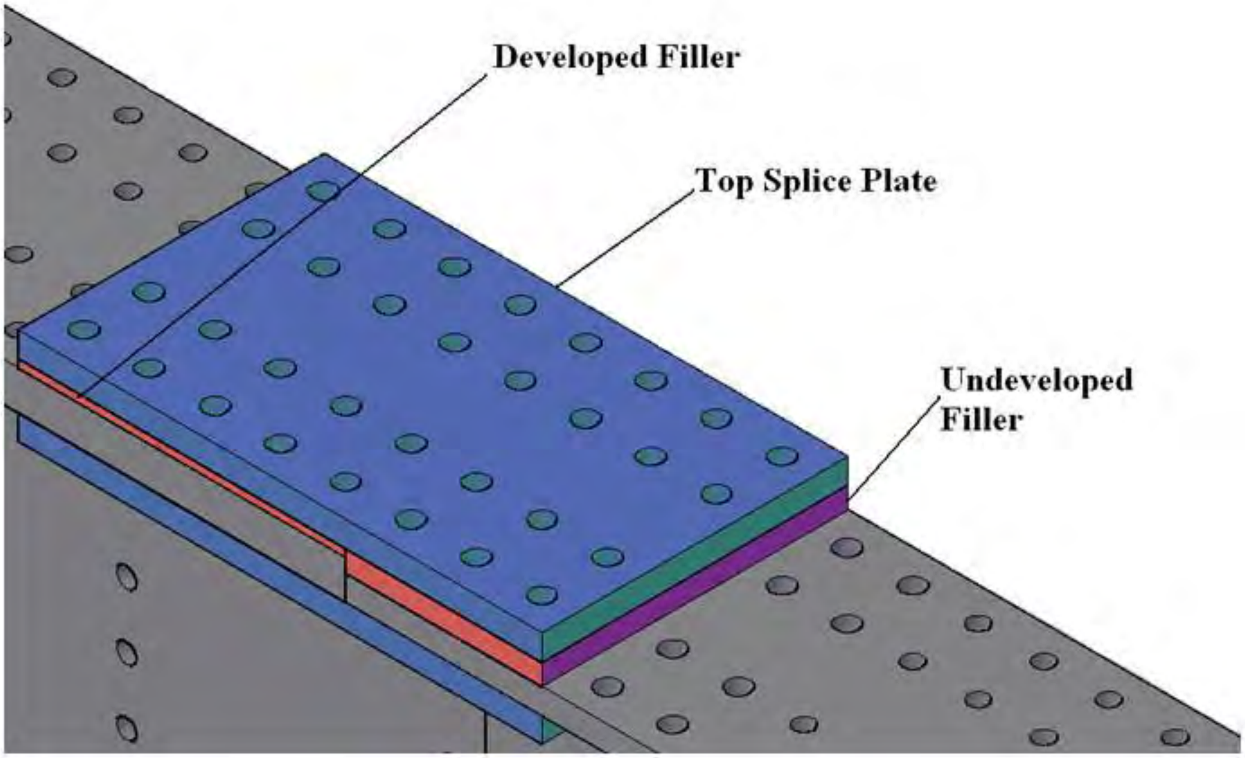


Figure 14: Developed and undeveloped fillers in a splice connection

5.2 Specimen Layout

Figure 15 depicts the general form for the test specimen used in the tests. The specimen consisted of two plate girders, an upper top flange splice plate, two lower top flange splice plates and filler plates of quantity and thickness determined in the test matrix. The two 21 1/2 ft A709 GR70 W high performance steel (HPS) plate girders were connected at mid-span using a splice connection. The girders were identical in geometry with a 1 1/8 in. x 30 in. web, 1 1/8 in. x14 in. flange, and 1 ¾ in. x 14 in. flange (see Figure 16). Figure 15 displays the bolt hole layout where the first five rows of holes on each end of the girder were 1 1/16 in. (oversize) diameter and the next 20 rows of holes on each end were 15/16 in. (standard) diameter. The holes were spaced 3 in. on center and 1.5 in. from the edges.

The upper splice plate was 14 in. x 24 in, and the lower two splice plates were 4 in. x 24 in. All splice plates were 1 1/8 in. thick A709 GR70 W HPS with a hole spacing of 3 in. on center and edge distances of 1.5 in. The hole sizing in the splice plates (standard or oversize) corresponded to the hole sizing in plate girders. The filler plates were made of A572GR50 steel and varied in size from ¼ in. to 2 in. thick.

Splice and filler plates were shot blasted to remove mill scale at a local fabricator, Fought Steel & Co. in Portland, OR. The girders were prepared as a class B surface. This process was selected to remove mill scale and leave a roughened surface consistent with Oregon Department of Transportation specifications.

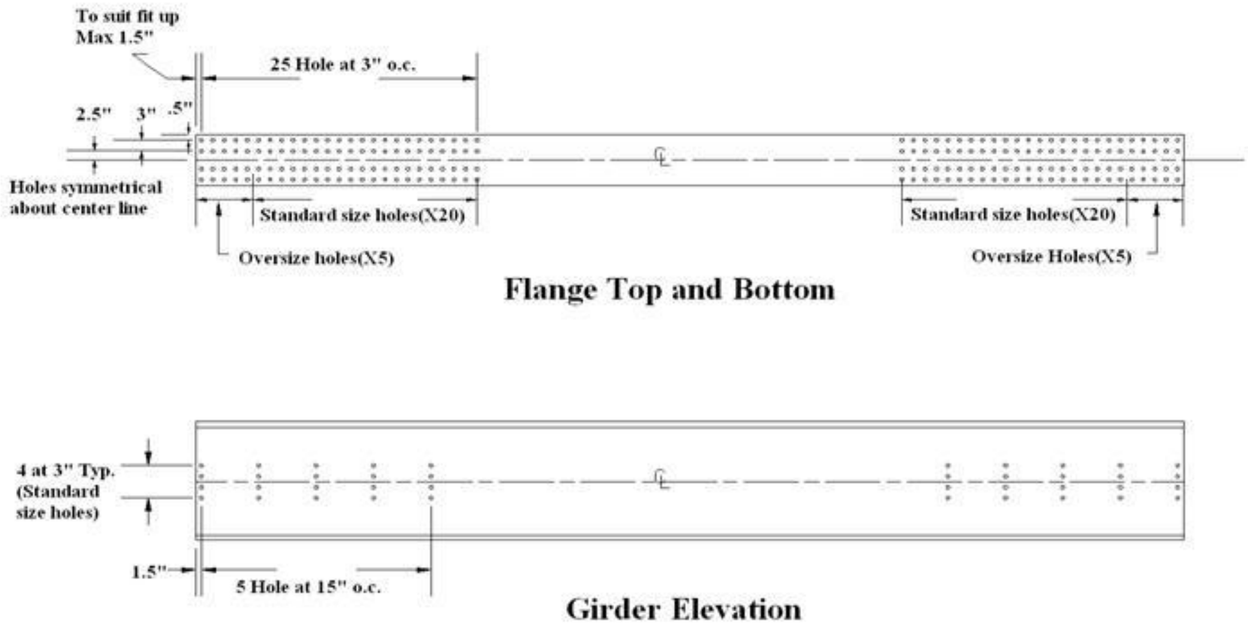


Figure 15: Geometry and hole schematic for the plate girders

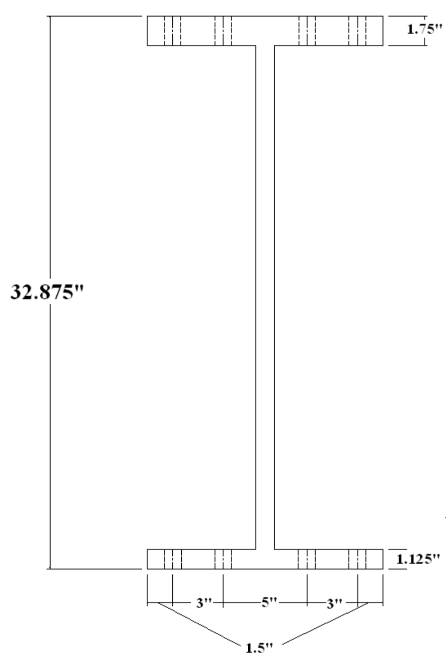


Figure 16: Front view of plate girder

5.3 Girder Splice Test Setup

Figure 17 illustrates the methods by which the splice connection with fillers was tested. The test set up utilized a four point loading system to create a maximum bending moment over the splice connection. The forces were applied from two sets of two SPX Powerteam 60 ton Hydraulic cylinders spaced 32” from the center of the connection on each side. The specimen was constrained by downward forces located 165” from the center of the connection on each side using 1” diameter threaded rods that were bolted into the reinforced concrete laboratory floor. In order to allow pivoting of the girders under the loading, the four point load forces were applied through 2” diameter A36 solid steel rods as shown in Figure 18. As the hydraulic cylinders pushed the connected girders upward, the threaded rod pulled with equal force downward, thus creating the required forces in the bolted connection.

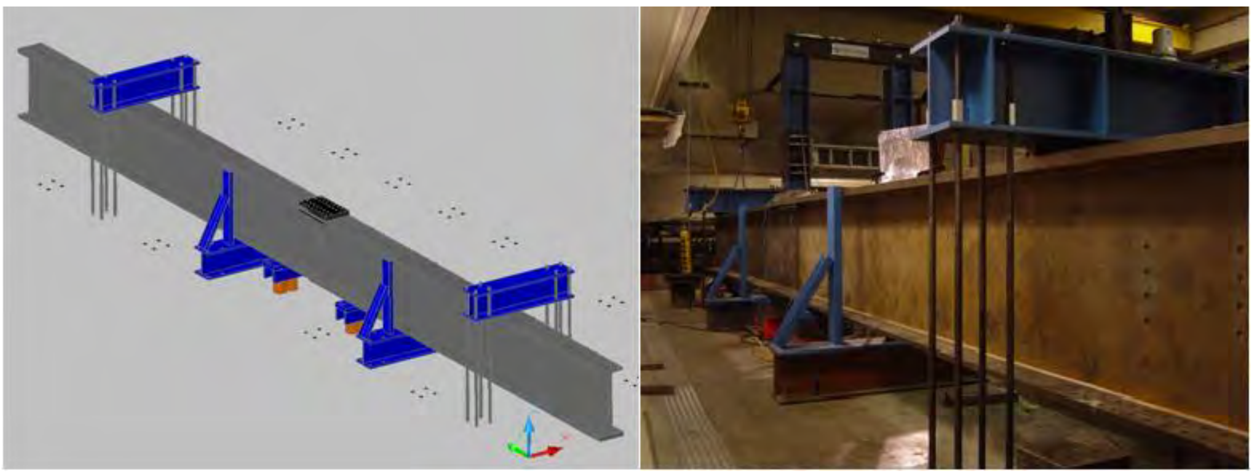


Figure 17: Girder splice test apparatus

Figure 18: Applied loading points for girder splice test.

5.4 Force Measurement

By using a four point loading system, a constant moment was created across the splice connection with minimal shear forces. An analytical model for shear and moment forces throughout the girder setup is depicted in Figure 19. From the shear and moment diagrams illustrated in Figure 19, the theoretical shear at the splice was 0 while the theoretical maximum moment was 2660 kip-ft. The theoretical values were generated assuming the maximum force possible for each ram was 120 kips/ram. Using the data obtained for maximum load capacity for a single bolt failure from phase 1 testing, the four hydraulic cylinders spaced with the given geometry were capable of creating the tensile force necessary to fail a 6 bolt splice connection on the top flange of the girder. The pressure delivered from the hydraulic cylinders while applying load to the specimen was measured using a pressure transducer. The pressure transducer was connected to a data acquisition system (DAQ) and readings were recorded using the computer software LabVIEW 7.

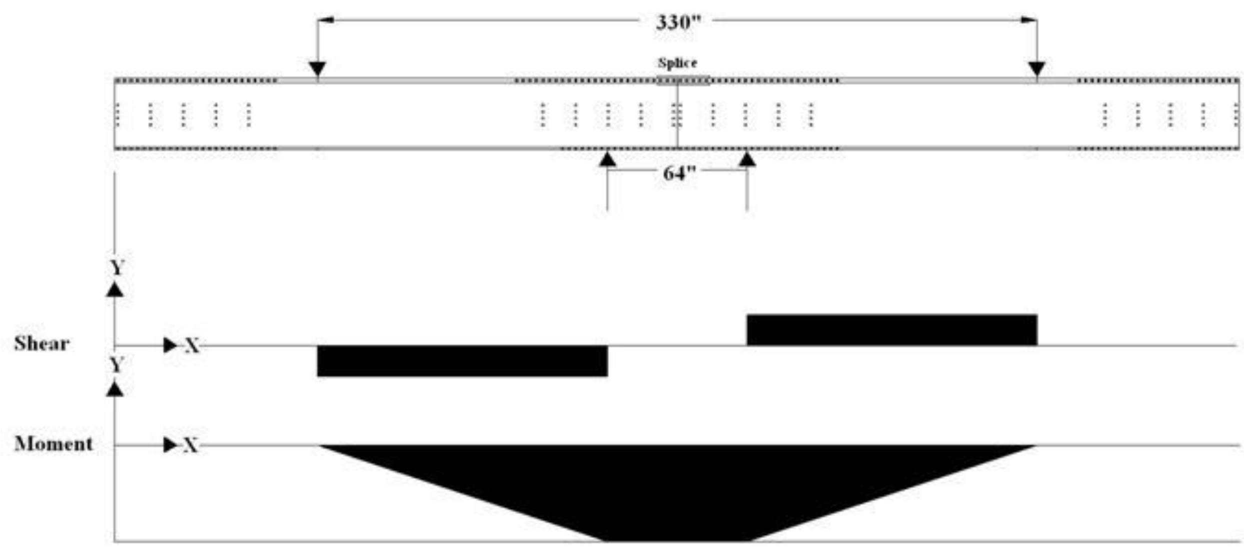


Figure 19: Analytical model for girder splice connection.

5.5 Test Preparation and Instrumentation

Prior to assembly of the splice connection all splice plates, filler plates, and girder portions involved in the connection were scrubbed clean using a degreasing solution and a 3M green scouring pad. After allowing the scrubbed pieces to dry, the girders were connected similar to the illustration in figure 1 using 16 A490 bolts. All girder splices were connected at the upper flange using two splice plates (one on each side of the web) on the bottom side of the upper flange, and one splice plate on the top of the upper flange. The filler plates were inserted between the top splice plate and the top of the upper flange. The bolts were tightened to "snug tight" by hand and the connection was then inspected to ensure all filler and splice plates were in the proper position. Once all plates were properly inspected, a STC5AE Simple Torqon and a TD-1000 torque multiplier were used to perform turn-of-the-nut on the bolts.

All displacement measurements were taken using linear variable differential transformers (LVDT's). The LVDT layout for this experiment is illustrated in Figure 20. LVDT 1 measured the vertical displacement of the bottom of the girders at the splice connection. LVDT's 2 and 3 measured the vertical displacement of the hydraulic cylinders. LVDT 4 measured the displacement between the girders at the middle of the connection. LVDT's 5 and 6 measured the displacement on each side of the connection between the splice plate and the top of the girder. LVDT's 8 and 9 measured the displacement on each of the filler plate(s) relative to the top of the girder. The set-up process was designed in order to have comparable measurements to those taken during phase 1 one testing. All LVDT's were connected to the DAQ, along with the pressure transducer , and readings were recorded in LabVIEW 7 at a rate of 100 samples per second.

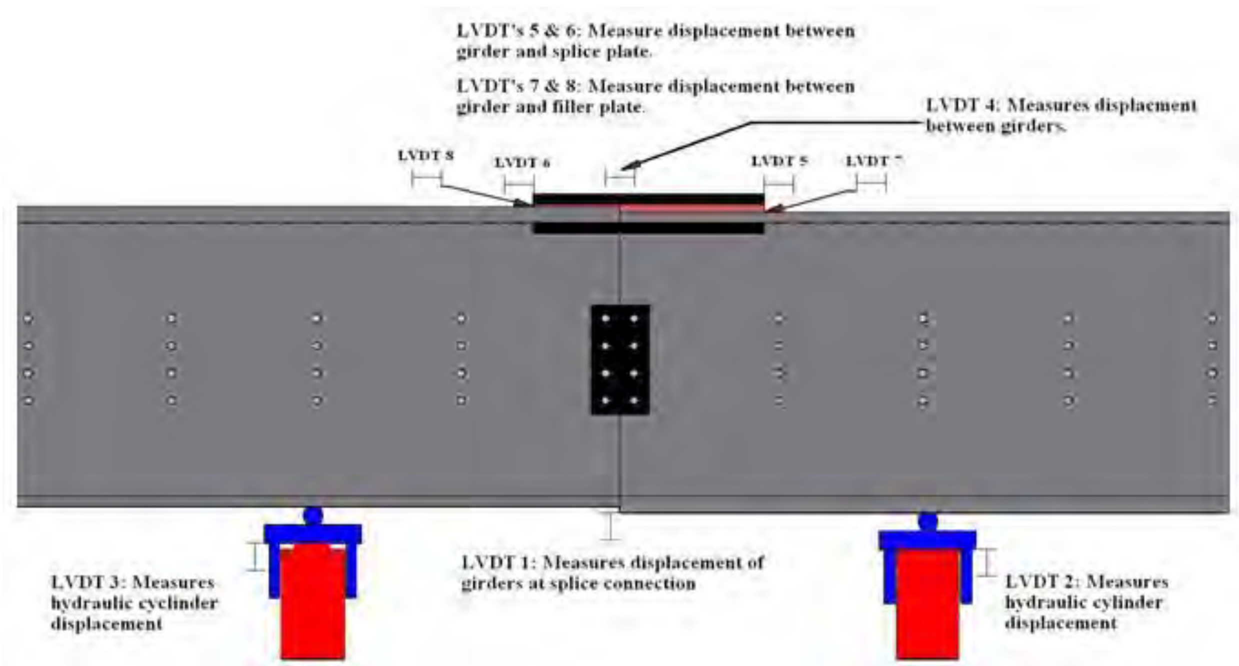


Figure 20: LVDT layout on the test apparatus

5.6 Girder Splice Test Matrix

Each girder had four rows of bolt holes through the flanges (top and bottom) on both ends of the girder. Figure 21 illustrates how each test was performed using only two rows of the holes, thus two tests were performed on the same flange (one test using inner rows and one test using the outer rows). A total of eight tests were conducted (2 tests on each flange, and on both sides of the girder) before the girders were cut to remove the tested bolt holes. All tests used 7/8" A490 bolts. Eight tests were performed using oversized holes, while all the others used standard size.

Table 2 outlines the test matrix for all the testing conducted in this experiment. Each test is labeled with a test number (1 through 28) and lists the associated description of the tests in terms of: Test series identification (standard, oversize or multi-ply), Filler plate size and configuration, Developed and undeveloped flange specification and Inner or outer flange hole use.

Figure 21: Filler 1 in Thick utilizing (a) inner flange holes and (b) outer flange holes.

5.7 Bolt Pre-Tension

A STC5AE Simple Torqon and a TD-1000 torque multiplier were used to perform turn-of-the-nut on the bolts in order to pre-tension them. The pre-tensioning of the bolts was achieved by tightening the bolt until the tension created reached the bolt's yield stress. Although test results from Wallaert and Fisher (1965) show that pre-tension has little effect on a bolt's shear strength, pre-tensioning enables all tests to be conducted with consistent bolt tension and ensures the accuracy of slip values measured during testing.

Four different lengths, from 4.5 to 6.5 in., of A490 bolts were used for the tests. All bolts had a 7/8 in. diameter and for each designated length were all from the same lot. For all tests, the bolt length was such that the shear planes went through the shank and not the thread of the bolt. The bolting up procedure for this report called for performing turn-of-the-nut on each bolt in order to achieve a consistent clamping force. This would take the bolt to its yield point thus producing its maximum and consistent tension. In order to find the amount of turn-of-the-nut needed, tests were performed using a Skidmore-Wilhelm device similar to Phase 1 of this research and photographed in Figure 22 to determine the tension created when the bolt was snug tightened and also at quarter turn intervals thereafter.

Figure 23 outlines the values from the tests conducted for each bolt with an average line running through the data sets. The tension produced is on the ordinate with the relative nut to bolt rotation on the abscissa. Each bolt length (4.5, 5, 5.5 and 6.5 inches) had 5 separate tests. Based on these tests, it was determined that every bolt needed to be turned a ¾ turn in order to bring them to their yield point.

Figure 22: The Skidmore-Wilhelm bolt tensile testing device (left) and plates added to the device to accommodate the length of bolt (right).

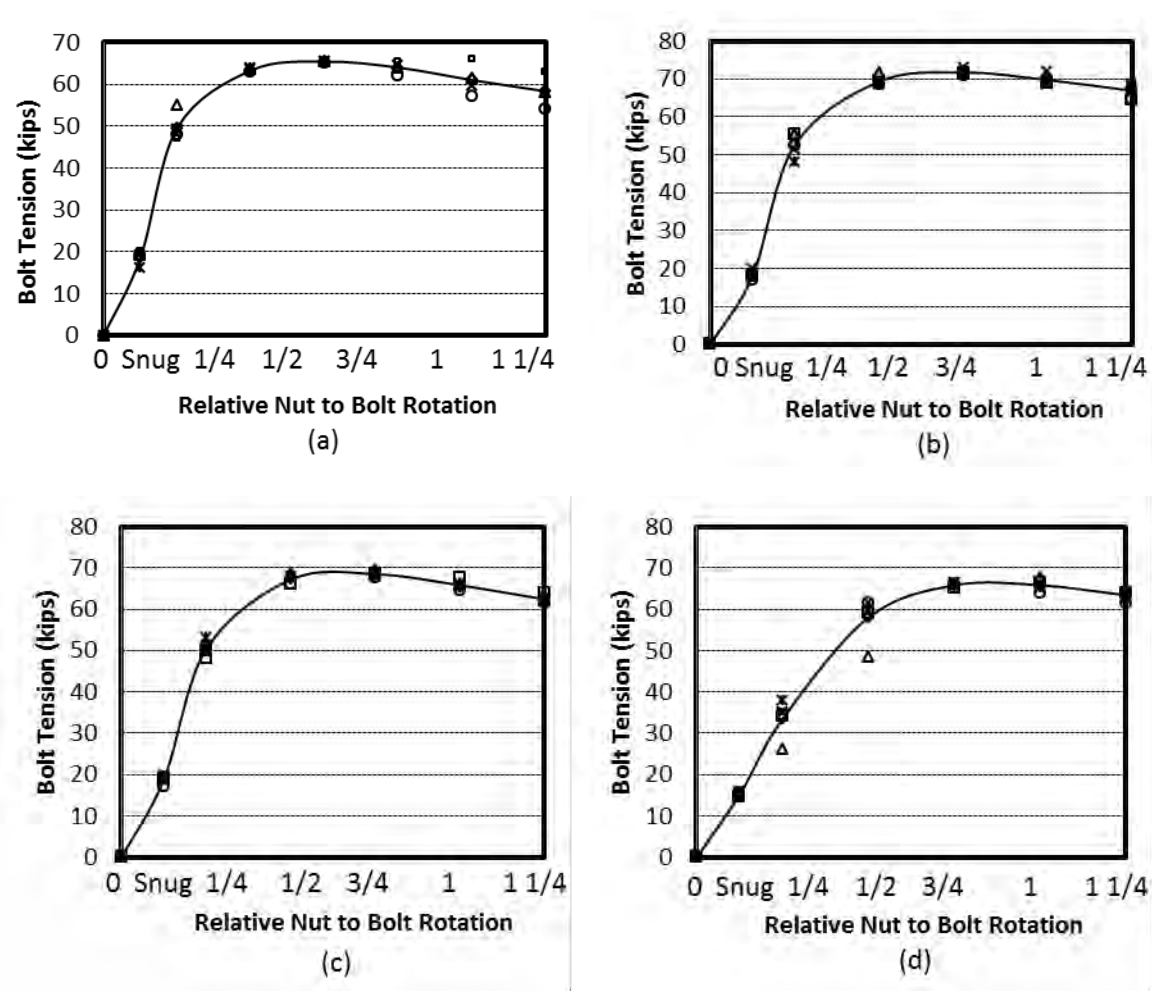


Figure 23: Tension created in bolts of varying lengths as turn-of-the-nut is performed (a) 4.5 in. A490, 5 tests (b) 5 in. A490, 5 tests (c) 5.5 in. A490, 5 tests (d) 6.5 in. A490, 5 tests

6.0 GIRDER SPLICE TEST RESULTS

The behavior of the undeveloped side of the bolted connection was the primary point of interest for this testing, so as to provide a valuable means of comparison to the results found from Phase 1. This section provides a summary of the results for ultimate strength, deformation behavior and slip resistance for standard hole, multi-ply and oversize hole tests.

The results for each test consisted of displacement data from LVDT's 1-8 (see Figure 20), as well as force data recorded from the pressure meter. The pressure transducer recorded the total pressure in the system due to the load of four 60 ton rams. Labview 7 converted this pressure reading into a total force reading in pounds. In order to convert the total pushing force (lbs) from the two point loadings into the tensile force at the top of the connection, the following equation using apparatus geometry was used:

$$F_{T,\ Connection}\ (kips) = [F(lbs)/(2*1000\ lbs/kip)]*[(132in/12in)/(32in/12in)]$$

Where $F_{T,connection}$ = The tensile force in the connection in kips, F = the total force in the entire system (i.e. both vertically pushing point loads) in lbs. as measured by the pressure transducer, 132 in. is the length between top and bottom loading points on one side of the girder and 32 in. is the height of the girder.

For each test, plots were made of the load ($F_{T,connection}$) vs. LVDT displacement, LVDT displacement vs. time and Load vs. Time. Graphs for LVDT's 2 and 3 were for ensuring that both point loads were pushing the girders upward at an even rate, LVDT's 5-8 were for monitoring displacement of the splice plate and fillers on both sides of the connection relative to the girders, LVDT 1 was for monitoring total vertical girder deflection at the connection and LVDT 4 was for measuring separation in the middle of the splice from one girder relative to the other. Figure 24 shows a representative plot of LVDT's 2 and 3 vs. time which illustrates that the specimen was under constant displacement and that nothing unexpected occurred in regards to the loading of the test specimen.

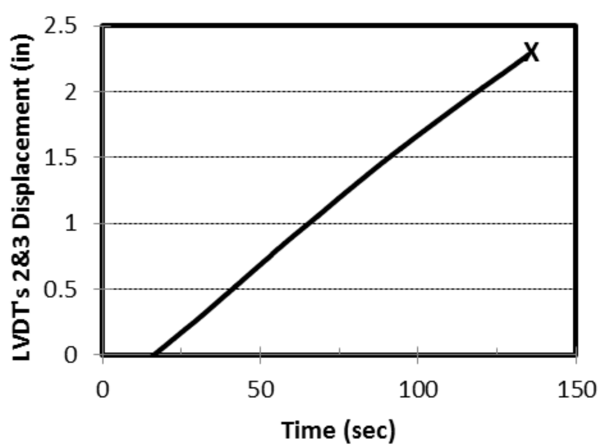


Figure 24: Typical ram velocity for girder splice testing

Figure 25 illustrates the typical load vs. displacement graphs for LVDT 5 or 6 (undeveloped splice plate displacements) and LVDT 7 or 8 (undeveloped filler plate displacements) vs. load for test 5 (1 inch undeveloped filler with oversize holes) and test 13 (1 inch undeveloped filler with standard size holes). The graphs show how connection displacement was larger for oversize hole testing than for standard holes, and how oversize hole tests had a slightly lower ultimate load than standard hole tests. The standard holes had less measured slip before bearing when compared to the oversized holes, and as a result, the oversize hole tests produced larger values of connection displacement. The graphs also illustrate where slip had occurred in the connection both on the 6 bolt side and then the 10 bolt side. Two different slip values were expected because the clamping force for each side was different due to the difference in the number of bolts.

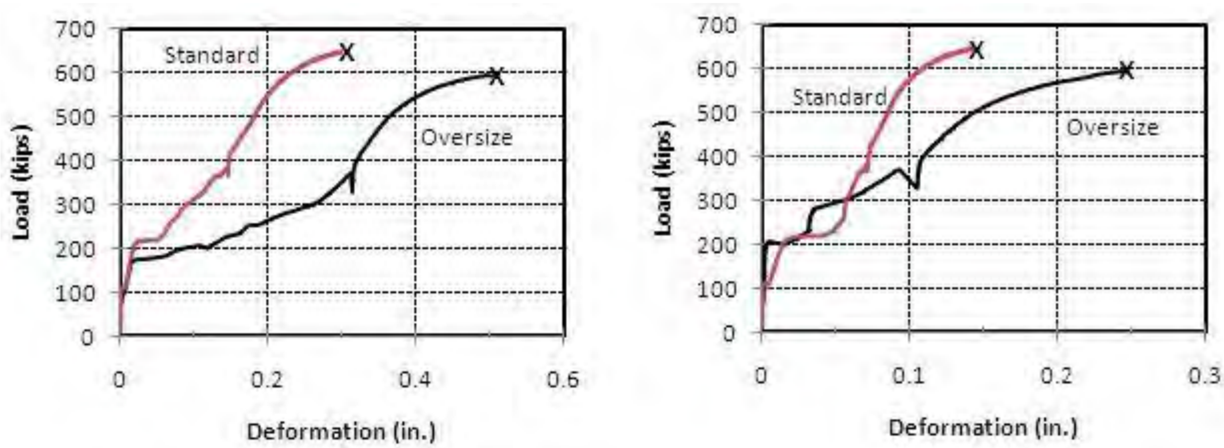


Figure 25: Undeveloped splice plate deformarion and undeveloped filler plate deformation for standard and oversize 1 inch filler tests

Figure 26 shows sheared bolts from the 6 bolt side of the splice connection after testing. One difference from phase 1 testing is the consistently defined shear plane for bolt failure throughout all of the tests with fillers (this includes oversize tests, standard tests and multi-ply tests). Tests without fillers behaved similar to phase 1 no filler tests in that bolts sheared into three pieces along the two defined shear planes between the top splice plate and girder, and between the bottom splice plate and girder. In all other filler tests, bolts sheared into two pieces along the shear plane between the bottom splice plate and the girder. The current design equation attributes the cause for a reduction in ultimate load achieved by a bolt in shear to be because of an undefined shear plane. The load reduction found during phase 1 multi-ply testing due to the undefined bolt shear planes was not found during phase 2 testing. Even for multi-ply filler testing (the most probable tests to create undefined shear planes), the bolts failed on the exact same plane as all the previous filler tests. As a result, there was no recorded reduction in ultimate load from solid filler to multi-ply filler tests where similar net thicknesses were used.

Figure 26: Examples of failed bolts for all filler thicknesses used during girder splice tests.

6.1 Ultimate Strength

The ultimate strength of a splice connection can be altered by factors such as an undefined shear plane in bolt failure and an increase in the bending of a bolt as the splice connection separates. As filler plate thickness increases, the potential for the bolt to bend also increases, thus resulting in a decrease in ultimate load strength. A decrease in ultimate connection strength can also be the result of increased tensile forces in the bolts due to lap plate prying (RCSC 2004). Figures 16, 17 and 19 illustrate the trend that developed between increasing filler plate thickness vs. ultimate load. The RCSC has developed guidelines concerning strength reduction due to the implementation of fillers for 50 ksi steel and A325 bolts, where the use of a0.75 in. filler results in a strength reduction of 15 %. The strength reduction found from girder splice testing for standard size holes on 70 ksi steel and A490 bolts shows an average strength reduction of 8.7% for 0.625 in. fillers, 18.7% for 1 in. fillers, and 21 % for 2 in. fillers. The multi-ply series saw a strength reduction of 10 % for 2x.3125 in. fillers, 15.8% for 4x.25 in. fillers and 22.6% for 0.25 in. + 1.75 in. fillers. The oversized holes saw a strength reduction of 10.8% for 0.625 in. fillers, 21.8% for 1 in. fillers and 29.2% for 2 in. fillers.

In contrast to phase 1 testing, the implementation of a 2 in. thick filler did not as effectively create a scenario where the thicker filler impeded bolt bending and acted like a stiffener. Rather, the 2 in. filler in the spliced girder connection reduced the ultimate strength of the connection to a value less than the 1 inch filler test. Figures 16, 17 and 19 show that the loss in ultimate load from 1 in. to 2 in. filler tests was considerably less than for the no filler to 1 in. filler tests. As a result, the stiffening action of the 2 in. filler was seen to be much less effective in girder tests when compared to pure tension tests. Also of importance was the ability of multi-ply connections to withstand similar ultimate loads when compared to single fillers of equivalent thickness. As previously explained, the consistent shear plane for standard and multi-ply tests provided a bolt shear strength for both types of filler configurations that could withstand similar ultimate loads.

In comparing the ultimate load reductions from phase 1 and phase 2, there is a discrepancy between the magnitude of load lost due to the implementation of fillers in the standard and oversize tests. A possible explanation for this difference is that excess bolt tension developed in the bolts on the undeveloped side of the girder connection due to the prying action of the splice plate. As fillers increased in thickness, the prying action could have also increased. The phase 1 standard and oversize tests did not see the same outward prying action in the connection because the tests were in pure tension. However, Figure 28 illustrates a large ultimate load reduction for phase 1 multi-ply filler tests (even larger than the girder tests). Observation from phase 1 indicated outward "plate fanning" that occurred during the multi-ply tests; which would result in an increase of tensile load in the bolts. The plate fanning observed in phase 1 tension tests was only observed after bolt failure for phase 2 tests with fillers; however the prying action of the tensile/bending load was still assumed to be present. The data suggests that the plate fanning of multi-ply fillers in pure tension is more detrimental to overall connection strength than the prying action observed through the combined tensile/bending load of the girder testing. Since the prying action was observed for all girder tests series, this serves as another explanation why the multi-ply fillers compared much more favorably to standard tests in phase 2, than they did in phase 1. The idea of the "prying action" and its subsequent effects on connection strength is only a possible explanation; therefore a substantial amount of uncertainty exists as to the specific reasoning behind the higher load reduction in phase 2 tests as fillers are introduced into the connection.

Ultimate Loads Reached for Standard Hole Tests

The maximum load recorded for each standard hole test and a comparison to phase 1 testing is summarized in Figure 27. All comparisons to phase 1 testing were done by taking the average ultimate load for each filler thickness during the 3 bolt tension tests, and doubling it (for an equivalent 6 bolt strength). An average line is used to better illustrate load trends for varying thickness of fillers. As the thickness of fillers increases, the ultimate load decreases. Phase 1 testing recorded an ultimate load increase from 1 inch fillers to 2 inch fillers, however this trend is not maintained during the phase 2 testing. As fillers are introduced into the connection a drop in shear strength is recorded due to bending that occurs in the bolts. Previous tension testing found that a much thicker filler (2 in.) created a space too thick for the bolts to bend around, and in turn created a stiffer connection with a defined shear plane. The result of this observation was an increase in ultimate load for the 2 in. filler tests. Phase 2 testing maintained a consistent shear plane throughout all filler size testing, resulting in a consistent drop in ultimate load as filler thickness was increased due to increased bending in the bolts.

Ultimate Loads Reached for Multi-Ply Tests

Figure 28 summarizes the ultimate loads achieved for multi-ply filler testing as well as a comparison to the phase 1, 3 bolt, multi-ply average ultimate strength values. An average value line is also displayed on the figure. The filler thickness values on the abscissa represent the overall thickness of the filler as follows: 0.625 in. = 2 x 0.3125 in., 1 in. = 4 x 0.25 in. (3 tests), 1 in. = 0.25 in. + 0.75 in. (2 tests), 2 in. = 0.25 in. + 1.75 in. Ultimate loads for the multi-ply tests compared favorably with the ultimate loads for the standard tests due to the shear failure plane on the bolts being the same for both test cases. Figure 29 illustrates the bearing impact each individual multi-ply filler (4 x 0.25 in. tests) had on the bolts in the connection, yet the failure still occurred at the interface between the bottom splice plate and girder. The increased bolt bending observed in pure tension (phase 1) due to the movement of each plate was not found to be an issue in decreasing overall strength during girder splice testing (compared to the standard filler tests).

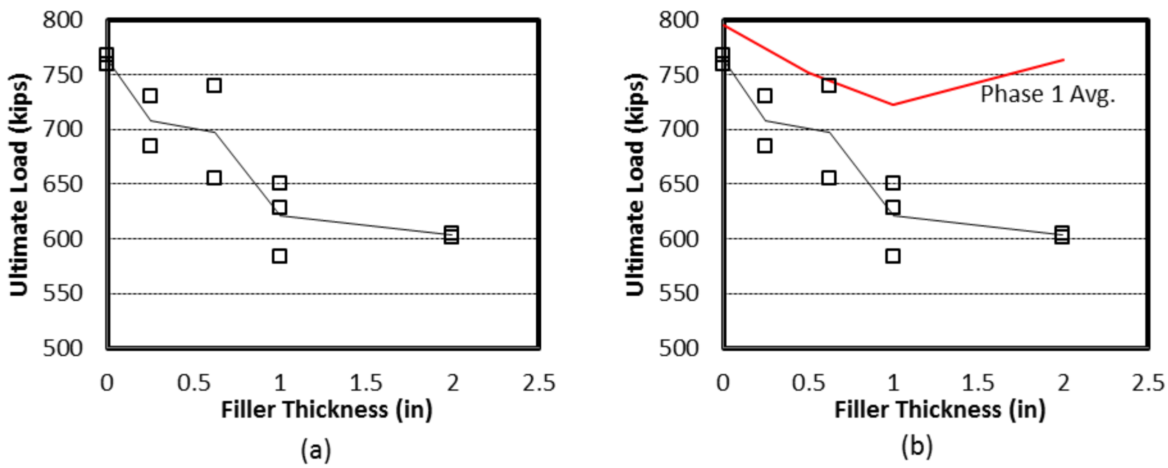


Figure 27: Ultimate loads reached during (a) standard hole tests for every filler thickness and (b) standard holes tests compared to 3 bolt standard hole tests from phase 1.

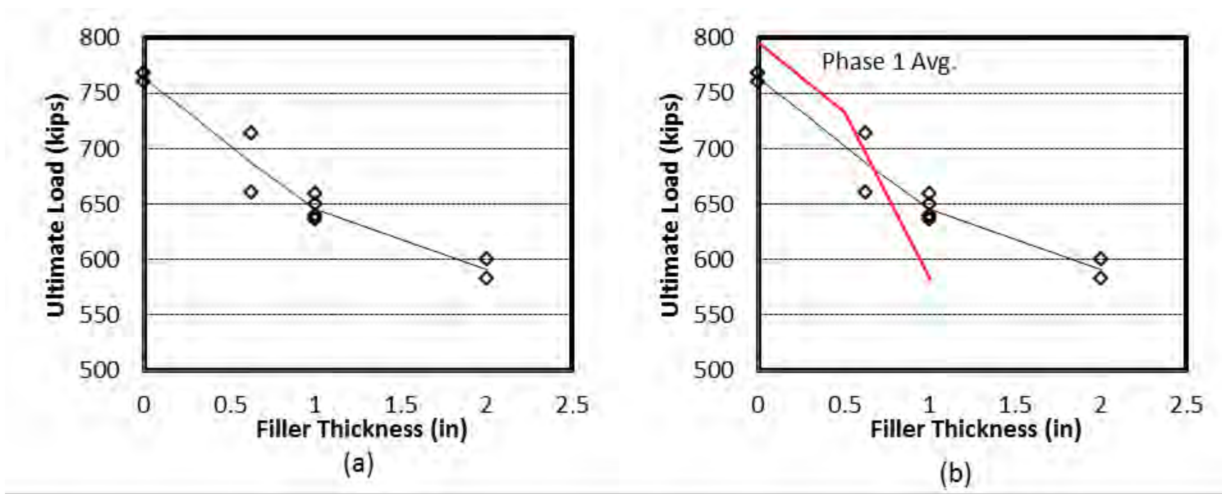


Figure 28: Ultimate loads reached during (a) Multi-ply tests for every filler thickness and (b) Multi-ply tests compared to 3 bolt multi-ply tests from phase 1.

Figure 29: Impact of multi-ply fillers on a bolt and failure plane for a 4 x 0.25 in. test

Ultimate Loads Reached for Oversize Hole Tests

Figure 30 outlines the ultimate loads reached for testing oversize holes with fillers up to 2 in. thick as well as a comparison to the average ultimate load values found from phase 1 testing. The trend of ultimate load vs. filler thickness stayed consistent with the trends found for standard and multi-ply tests; as the filler thickness increased, the ultimate load decreased. The ultimate loads recorded for oversize tests were roughly 4.4% lower than the loads achieved from the standard tests. The largest difference in ultimate loads between standard and oversize cases came with the implementation of the 1 and 2 inch fillers. The ultimate strength comparison between standard and oversize cases for phase 2 testing was similar to the trends produced from phase 1 testing. The slight lowering of ultimate load for oversize holes was expected because the increased hole size allowed for greater bending in the bolt. Figure 31 more effectively illustrates the effect that oversize holes have on bolt bending. The figure illustrates the difference in splice deformation at bearing and failure for standard and oversize cases. The angles were measured from the center of the girder hole to the center of the splice plate hole, and were used as theoretical values to predict the angles the bolts were likely to see in bending. The largest difference between phase 1 and phase 2 testing was that a strength rebound did not occur from the 1 inch filler to the 2 inch filler. The stiffening action of the 2 in. thick filler in the pure tension tests appeared to be non-existent in the tensile/bending loading from performed in the girder tests.

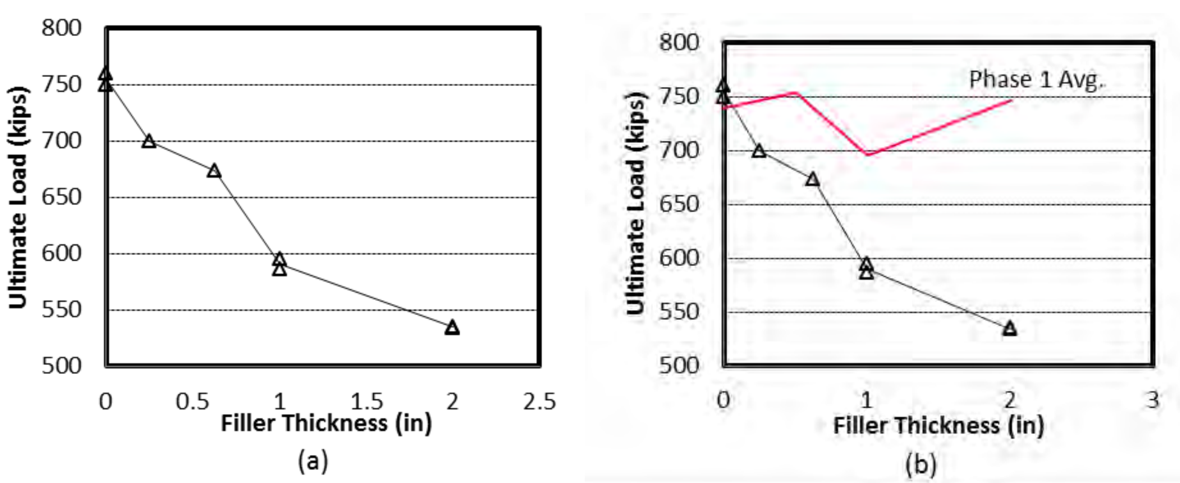


Figure 30: Ultimate loads reached during (a) oversize hole tests for every filler thickness and (b) oversize holes tests compared to 3 bolt oversize hole tests from phase 1.

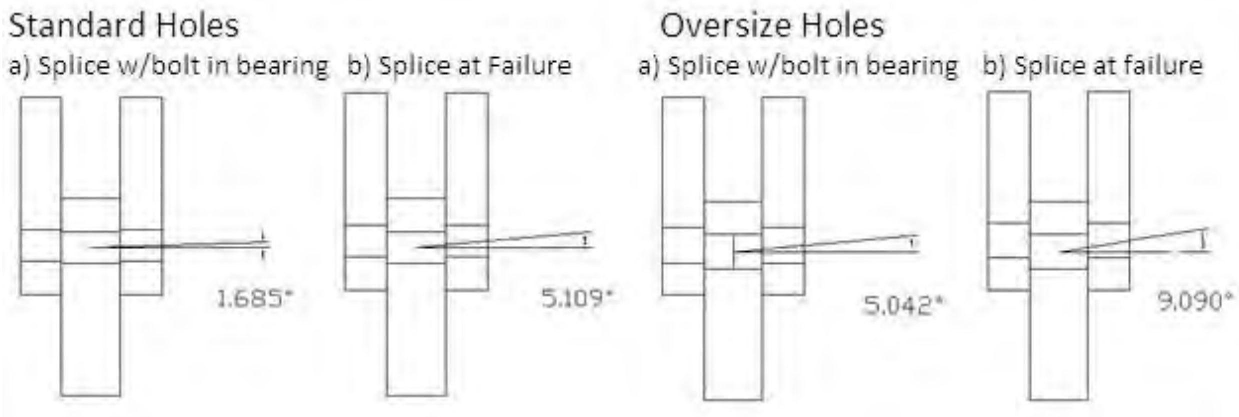


Figure 31: Theoretical bolt deformation angle at bearing and failure for standard and oversize cases.

6.2 Force Deformation Behavior

From Phase 1, the measure of connection deformation was defined as the deformation of the pull plate at the loose end of the assembly relative to the splice plates. The displacement of the undeveloped splice plate with respect to the girder (LVDT 5 or 6 depending the test) most accurately coincides with the measure of connection deformation defined in the previous sentence. A representative force versus total deformation plot for all three cases and different size fillers is shown in Figure 32. As illustrated in Figure 32, the general trend for each of the connections involved an initial slip followed by connection movement leading to bolt bearing, and finally inelastic deformation ending in failure. After the initial slip, the increase in deformation was measured against little or even decreasing resistance until the bolts were fully engaged in bearing. The amount of displacement measured in the connection from the time of initial slip to complete bearing varied from test to test. This issue is addressed in further detail in the sections to follow. After engaged in bearing, the resistance began to rise in a nonlinear fashion, thus causing the bolts and plates to deform inelastically. Permanent deformations in the splice plates as well as the bolts (see Figure 26 and Figure 34) signify regions of plastic deformations within the connection. As per specimen design objectives, failure occurred in the bolts on the undeveloped side of the connection. Tests performed with no filler plates forced the bolts to fail on both shear planes (top splice plate to flange and bottom splice plate to flange), thus causing the bolts to break into three pieces. Tests performed with filler plates caused the bolts to fail on one shear plane (between the bottom splice plate and girder) where the bolt was broken into 2 pieces. The presence of filler plates affected the force deformation as well as the overall performance of the bolted connection.

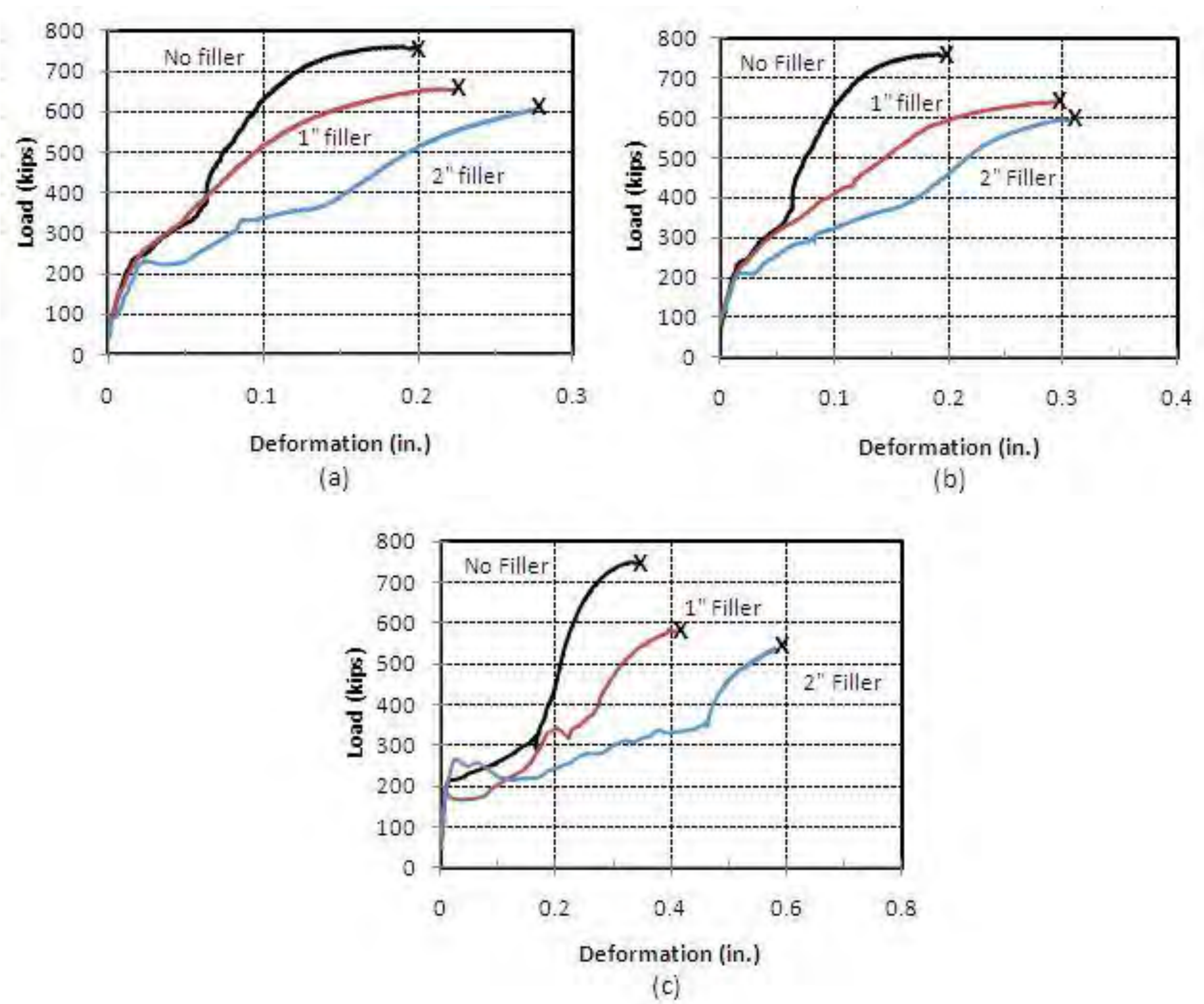


Figure 32: Representative force vs. displacement plots for (a) standard tests, (b) multi-ply tests and (c) oversize tests.

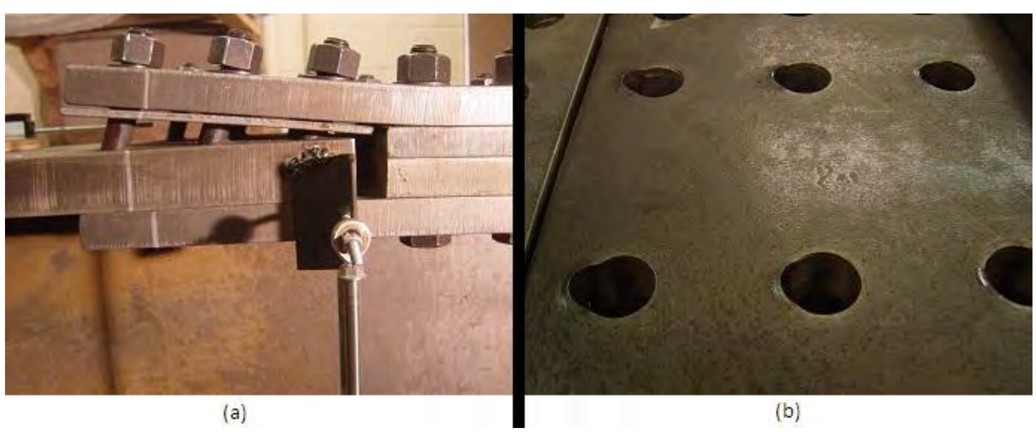


Figure 33: Regions of plastic deformation after girder tests in (a) the top splice plate/bolts and (b) the bolts holes.

Deformation at Failure

Figure 34, Figure 35 and Figure 37 show the deformation of the undeveloped splice at failure plotted against varying filler thickness for all three test cases as well a comparison to the average displacements recorded during phase 1, 3 bolt tests. The failure observed during all of these tests involved an approximate-simultaneous failure of all 6 bolts on the undeveloped side of the connection. For standard tests an average total displacement value for the 2 in. fillers was seen to be lower than that found during the 1 in. filler tests; however this trend was not observed for the multi-ply and oversize series. It is possible that the 2 inch solid filler acted as a restraint to bending in the bolt, thus causing a decrease in overall deformation. A similar trend was found in the phase 1 testing for both standard and oversize tests; however the stiffening action also provided a rebound in strength that the girder tests did not show with regard to strength.

For all three test series the deformation at failure for no filler cases was approximately the same for phase 1 and phase 2 testing; however the inclusion of fillers caused much greater connection deformation in the phase1 tension tests. As previously shown in the ultimate load analysis, the phase 1 filler tests were able to withstand higher (relative) ultimate loads than the phase 2 tests. The four point bending load in phase 2 testing induced bolt bending at lower tensile connection forces than recorded in phase 1. As a result, girder tests failed more quickly and were not allowed the excess load capacity to reach the deformation at failure that was recorded during the phase 1 tests.

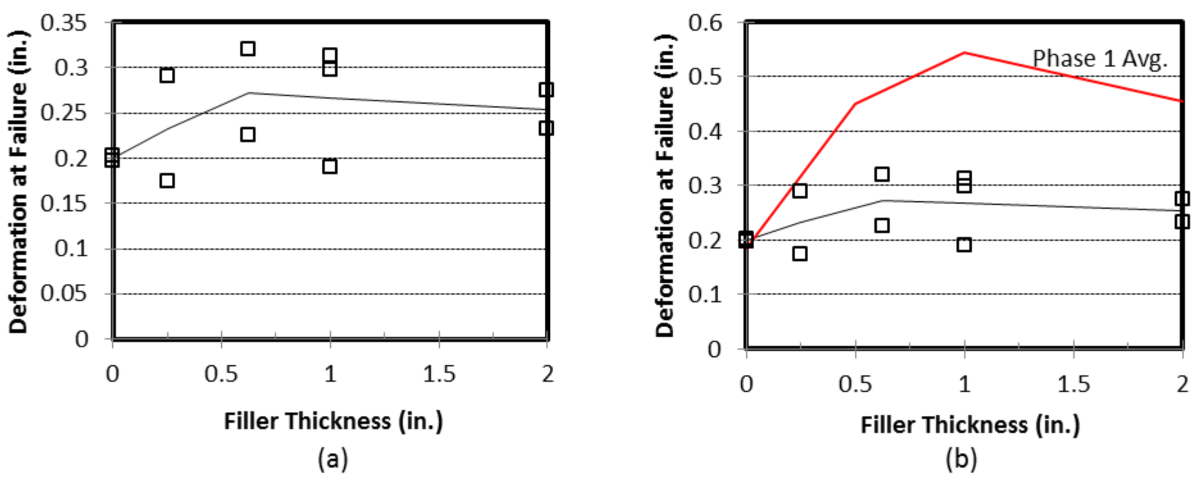


Figure 34: Plate deformation at failure for (a) standard tests and (b) standard tests compared to phase 1, 3 bolt standard tests

Figure 35 illustrates a difference in plate deformation trend from 1 in. to 2 in. filler tests when comparing the standard and multi-ply series. As the plot illustrates, the plate deformation at failure continued to increase as the filler thickness in the connection was increased. It appears that the multi-ply 2 in. thick filler was not able to act as a solid restraint to prevent bolt bending (as seen in the 2 in. standard tests) due to the individual movement of each filler. Figure 36 illustrates the differential movement of each filler with respect to the girder for a 1 in. and 2 in. multi-ply test. Because the restraining action of the 2 in. solid filler was not present during multi-ply tests, the deformation at failure was allowed to increase. Overall, the multi-ply tests had observed deformation at failure values that were comparable to the standard tests (the only exception being the 2 in. filler tests previously explained above). Phase 1 testing showed the multi-ply tests to perform much more poorly in pure tension. This can be attributed to the lack of defined shear plain as well as the fanning action of the fillers.

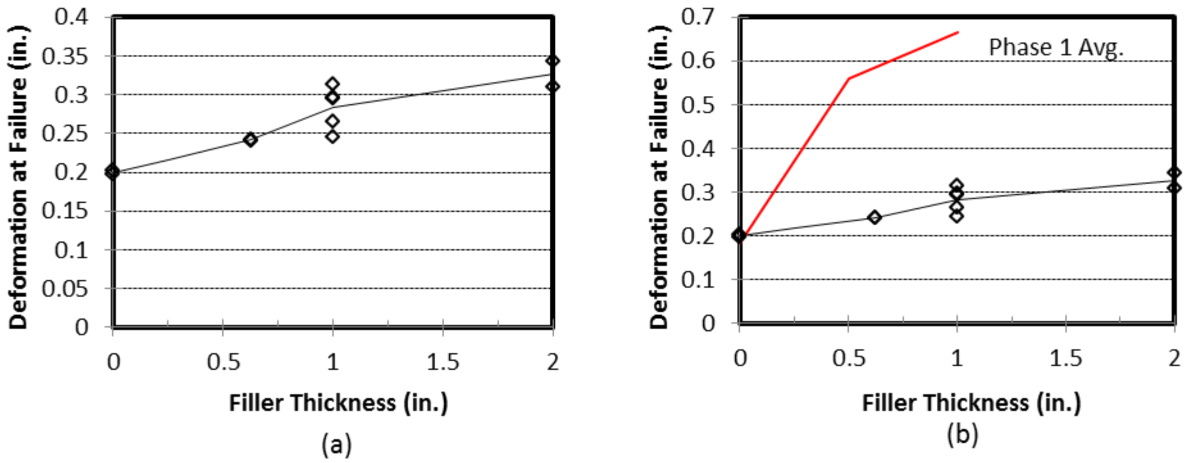


Figure 35: Plate deformation at failure for (a) Multi-ply tests and (b) Multi-ply tests compared to phase 1, 3 bolt multi-ply tests.

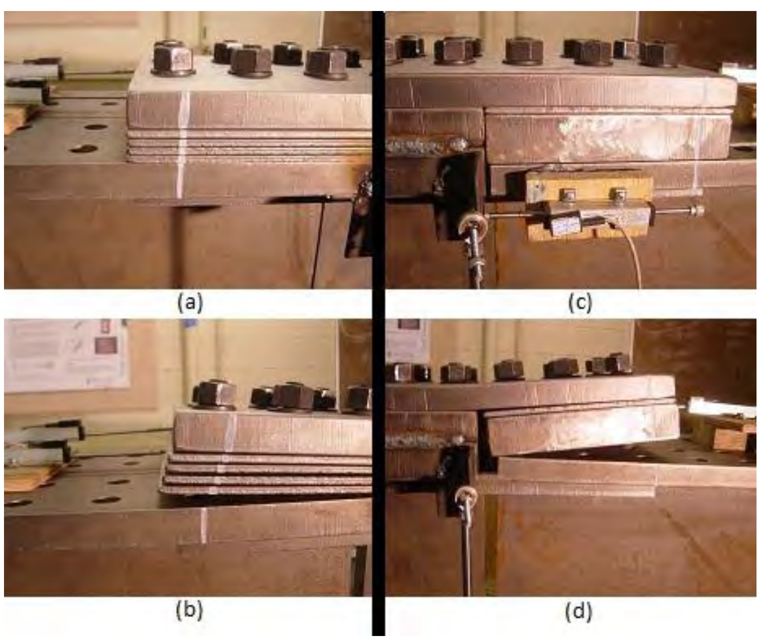


Figure 36: Multi-ply series individual filler plate deformation for 1 in. tests (a) before and (b) after testing; and 2 in. filler tests (c) before and (d) after testing.

The oversize testing displayed a substantially larger deformation at failure when compared to the standard and multi-ply tests. A plot summarizing the plate deformation at failure compared to varying filler thicknesses for oversize holes is found in Figure 37. The oversize testing saw a similar trend to the multi-ply tests in that the failure deformation increased as filler thickness was increased from 1 in. to 2 in. This trend contradicts the phase 1 testing which observed a decrease in failure deformation as fillers were increased from 1 in. to 2 in. It appeared that the oversize holes in the 2 in. filler girder testing allowed for increased bolt bending, thus negating the restraining effect of the 2 in. filler that was observed during the standard test series as well as phase 1 testing.

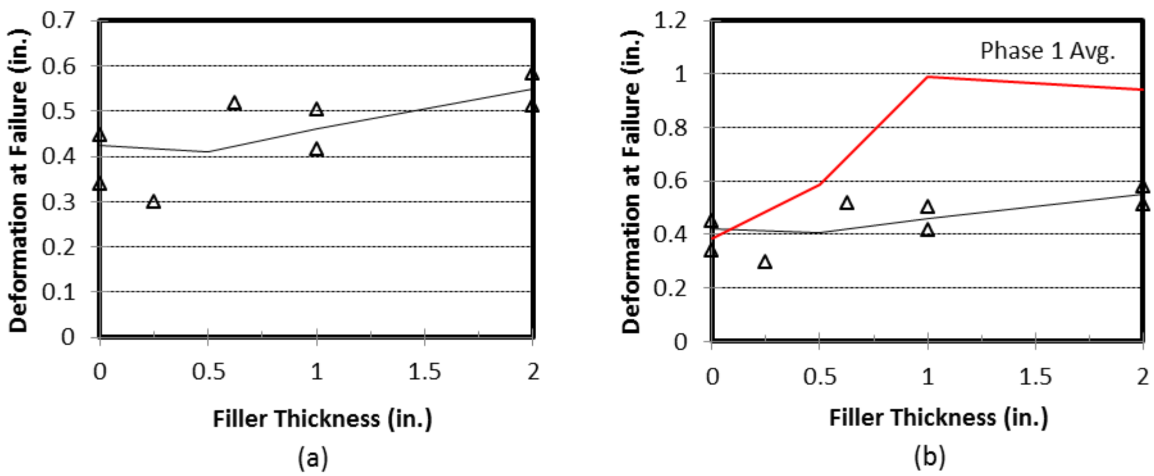


Figure 37: Plate deformation at failure for (a) oversize hole tests and (b) oversize hole tests compared to phase 1, 3 bolt oversize hole tests

Figure 34, Figure 35 and Figure 37 illustrate a considerable amount of difference in deformation at failure from phase 1 to phase 2 testing. Unlike phase 1 testing, where bolts were installed butted against the holes in reverse bearing for standardized deformation until bearing, the phase 2 tests had bolts placed in the holes at unspecified positions due to apparatus constraints. Because of this unknown placement of bolts within the holes, the potential existed for some connections to be brought to bearing with less displacement than others, even if the same size of fillers were used. Since all phase 1 tests recorded displacement from the reverse bearing bolt position, it serves as another explanation why the phase1 values for maximum connection deformation would be greater than the phase 2 values.

The result of the unknown bolt placement within the holes was also seen as reasoning for the data scatter of failure deformation points within each graph of load vs. deformation found in Figure 34, Figure 35 and Figure 37. Figure 38 illustrates the potential maximum displacement of the connection prior to complete bolt bearing. The distance illustrated in Figure 38 represents the maximum possible difference in deformation at failure that could be possible for tests using the same size fillers.

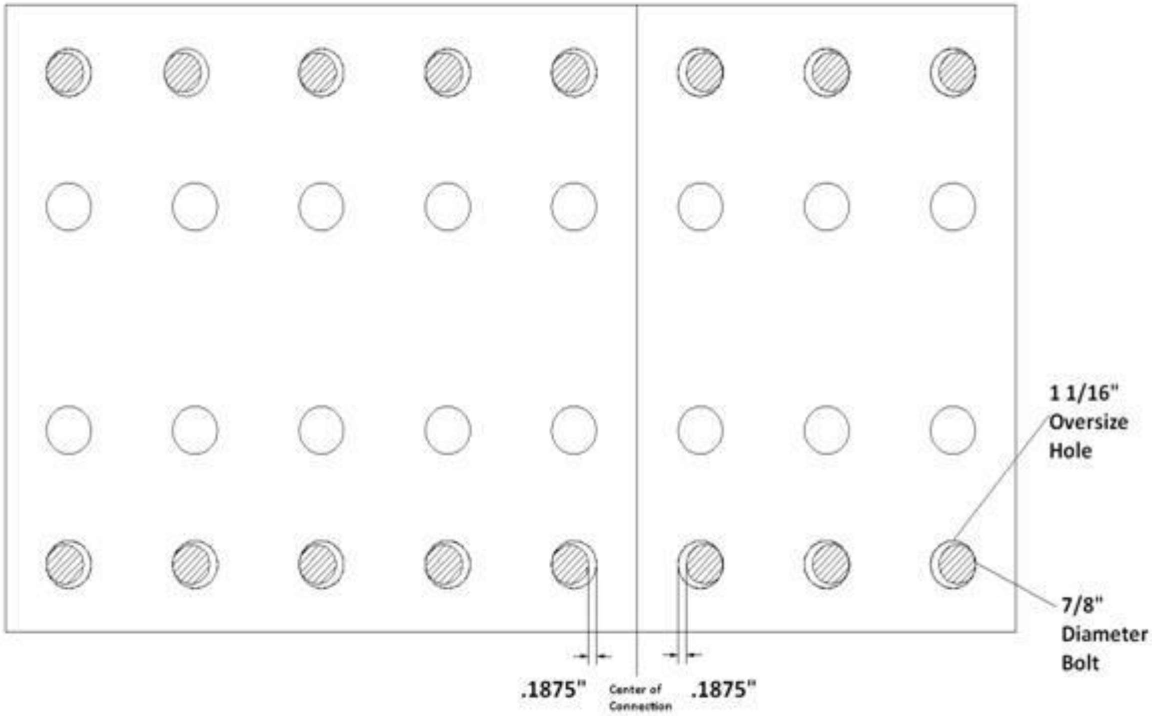


Figure 38: Potential connection displacement prior to complete bolt bearing for oversize holes.

<u>Deformation Limit</u>

Yura et al. specified a deformation limit of 0.25 in. to govern as the point at which a connection was unfit to perform satisfactorily due excessive displacement. Design equations developed by the RCSC as well as the American Institute of Steel Construction (AISC) use this deformation limit of 0.25 in. based upon previous filler testing. Table 4 outlines the loads for all tests at a deformation of 0.25 in. based again upon the deformation of LVDT 5 or 6 (undeveloped splice plate deformation). Figure 39 summarizes the trend for load at 0.25 in. of deformation vs. filler thickness for standard, multi-ply and oversize test series. Figure 39 illustrates that as filler plate thickness increased, the load required to cause 0.25 in. deformation decreased. This effect can again be attributed to the larger filler plates allowing increased bending in the bolts. In both standard and multi-ply tests, the shear strength reduction is not nearly as drastic as with the oversize tests. This is because the oversize holes allow for more connection deformation until the bolts are completely brought into bearing, thus making the loads reached at 0.25 in. considerably lower than the standard and multi-ply cases. Similar to the phase 1 tests, the loads required to reach 0.25 in. deformation for the oversize series appeared to reach a lower limit due to the bolts' lack of time to be fully brought into bearing and increase its resistance against deformation. For this reason, the oversize series saw loads at 0.25 in. of deformation that were similar to the slip loads.

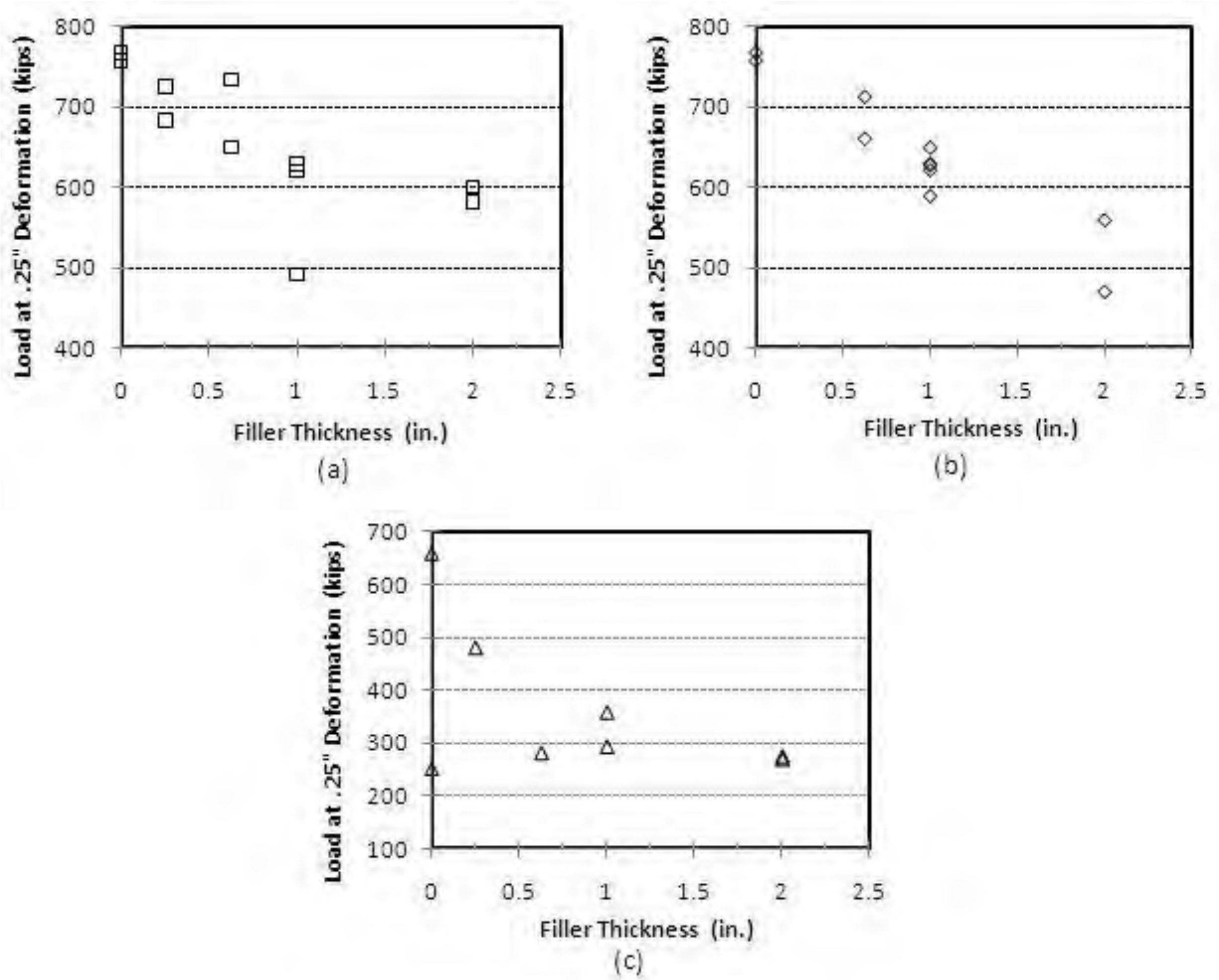


Figure 39: Load at 0.25 in. deformation for (a) standard tests (b) multi-ply tests and (c) oversize tests for every filler plate thickness

Of concern is the reduction in shear strength affiliated with the implementation of fillers in tests with standard size holes. The equation proposed for shear strength reduction by Yura is 1-0.4t, where t is the thickness of the undeveloped filler in the splice connection. The RCSC sites this equation as a general guideline for shear strength reduction in the design of connections implementing fillers with standard holes. This design recommendation follows the trend that as filler plate's thickness increases, the bolts' shear strength decreases. Figure 40 illustrates a comparison of the strength reduction found during girder splice testing and Yura's shear strength reduction equation. The shear strength reduction for each point was calculated as each test's load reached at 0.25 in. deformation divided by the average load observed for the no filler tests. An average line was then calculated (seen overlapping the data in figure 29) and its slope taken as the shear reduction factor. As seen in Figure 40, the results from girder splice testing created a reduction line that was much less conservative (1-0.2t) than Yura's proposed reduction line (1-0.4t) for 0 to 1 inch fillers, and even less conservative (1-0.08t) for fillers from 1 in. to 2 in. Phase 1 testing for the 3 bolt series showed results much closer to Yura's equation (1-0.37t); however the testing recorded a similar trend in the reduction found from 1 in. to 2 in. fillers (1-0.18t). This similarity in trend suggests that (as stated earlier) when a filler plate's thickness is increased from 1 in. to 2 in., a stiffening effect occurs that aids in limiting the amount of bolt bending that would cause the connection deform. This stiffening action was recorded in all test series (ultimate load and deformation behavior) during phase 1 testing, but is not as prevalent in girder splice tests. The stiffening effect of the 2 in. filler was evident in standard series deformation at failure and in standard loads at 0.25 in. deformation, yet was not observed in the analysis of ultimate loads.

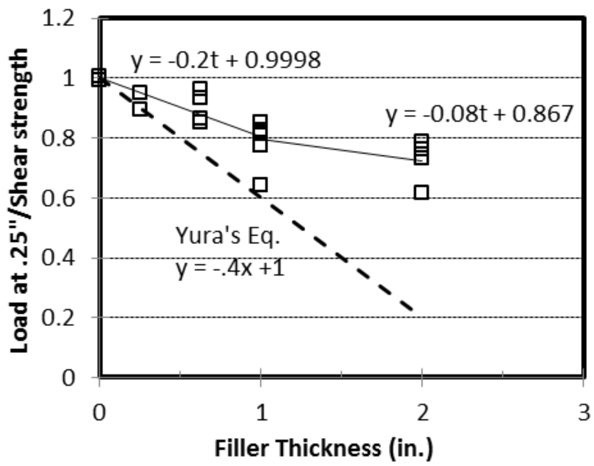


Figure 40: Shear strength reduction lines for girder splice testing compared to Yura's line for suggested shear strength reduction in standard hole tests for every filler plate thickness

It should be noted that the data for load at 0.25 in. deformation showed phase 2 tests to perform well in terms of strength reduction compared to phase 1 tests, whereas the data for ultimate strength showed the phase 2 tests to have a much higher strength reduction than the phase 1 tests. The 0.25 in. displacement data can be misleading when taken out of context. Since most of the phase 2 tests did not have recorded displacements much larger than 0.25 in., their loads were close to ultimate loads at this limit. The phase 1 tests (with higher deformation at failure) were not nearly as close to their ultimate loads at the 0.25 in. displacement limit. As previously explained, the phase 1 connections began displacing from the reverse bearing position, while the positioning for the phase 2 tests was unspecified. This would enable phase 2 tests to resist higher loads (be brought into full bearing) at lower measured displacements. As a result, the comparison of phase1 to phase 2 loads at 0.25” deformation is not an accurate measure of true strength reduction in the connection.

<u>Girder Rotation</u>

Another component that was monitored was the rotation angle of the girder as load was applied. LVDT 1 measured the overall upward deflection of the splice connection vs. force and time. Figure 41 illustrates how the test apparatus geometry along with the recorded displacements of LVDT 1 and LVDT 5/6 were utilized to calculate the rotation angle of girders. Since some girder splice tests recorded larger values of splice plate movement on the developed side of the connection than others, the rotation angle based upon the displacement of LVDT 1 did not show a consistent representation of displacement as a result of the undeveloped filler side. This is why the rotation angle of θ' seen in Figure 41 is calculated as the girder rotation angle due to undeveloped connection deformation only. The results found in Figure 42 illustrate similar trends to those recorded for the deformation at failure in the connection. This makes sense because the upward deflection angle of the girders will directly affect the splice plate displacement relative to the upper flange (pull plate). Each graph has an accompanying average value line overlapping the data to better illustrate the load vs. rotation trends.

The standard series were the only tests to show signs of a recovery in rotation angle as filler thickness was increased from 1 in. to 2 in. The oversize and multi-ply tests recorded a continued increase in girder rotation as filler thickness was increased. Standard tests recorded the lowest overall girder deflection; however multi-ply test deflection calculations were within 10 percent of the standard values on average. The oversize holes had recorded deflection values that were nearly 1.85 times the deflection values of the standard tests. The same explanation for these results was explained earlier in the "deformation at failure" section of the report. These results further support the case that multi-ply filler could be a viable construction option when thicker fillers are not available, while oversize holes would better be suited for slip critical connections, as opposed to bearing type.

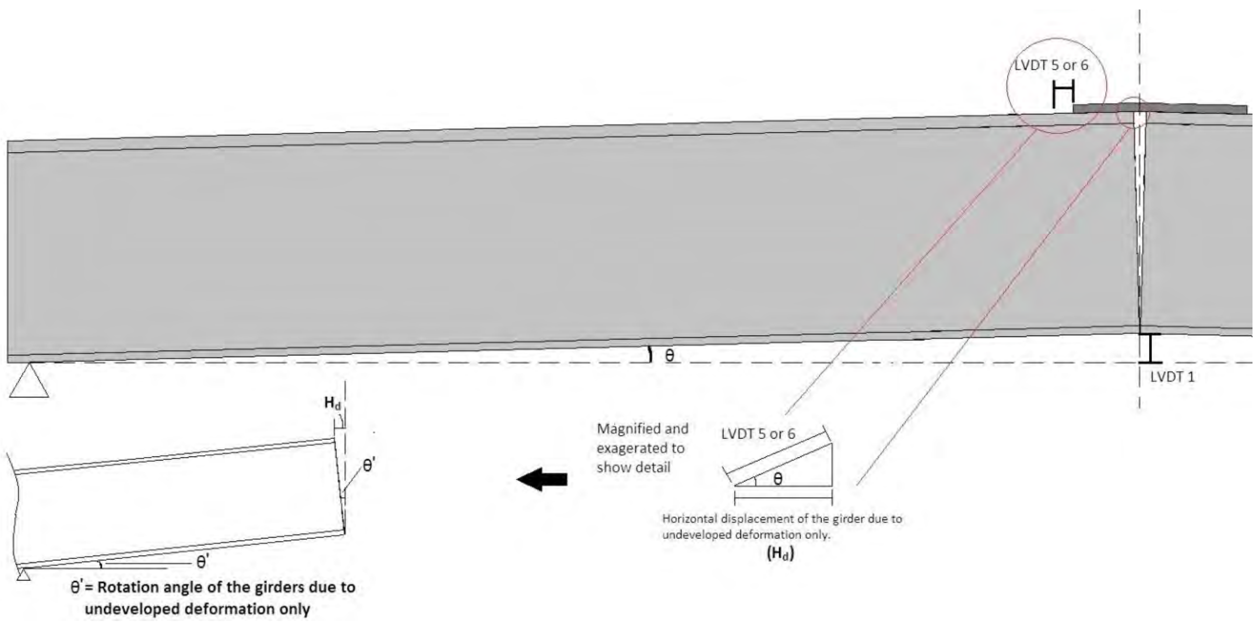


Figure 41: Girder rotation angle analysis.

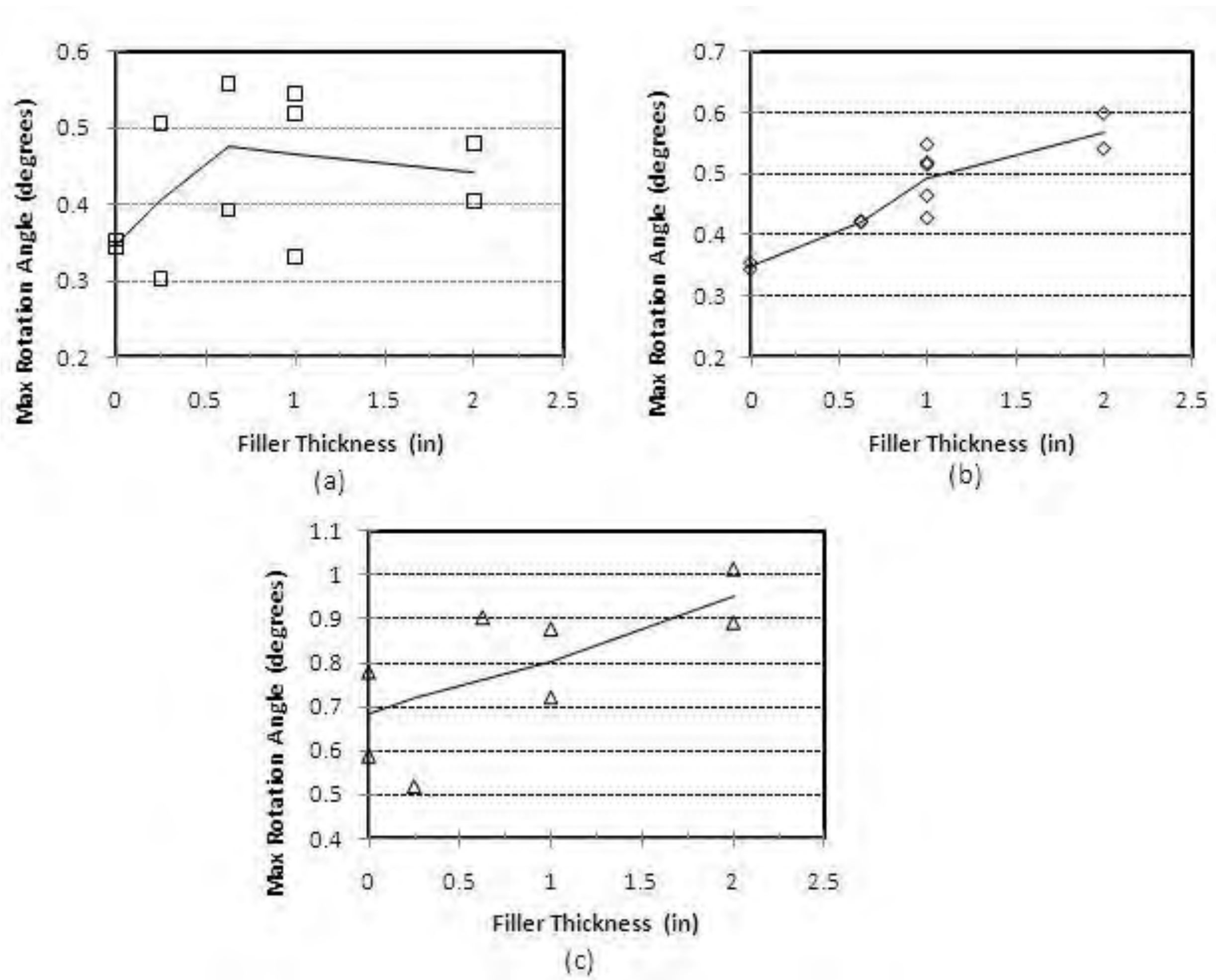


Figure 42: Maximum girder rotation angle for (a) standard tests (b) multi-ply tests and (c) oversize tests for every filler plate thickness.

6.3 Slip Resistance

Slip resistance for each test was analyzed using methods specified by the RCSC. The coefficient of friction, μ, was calculated using

$$\mu = F/N$$

where F is ½ (double shear) the tensile force in the connection immediately prior to the first incidence of slip, and N is the normal force generated by the pre-tensioned bolts. The pretension force of the bolts was equal to the number of bolts in the connection multiplied the by the tensile force in the bolt as a result of the ¾ turn of the nut. The force from the ¾ turn of the nut was known from the bolt tension calibration.

The AISC specifies minimum load capabilities for slip critical joints. A slip critical joint is one which relies on frictional resistance between plate surfaces rather than the shear strength of the bolts in order to hold the connection in place. For this reason, undeveloped fillers are deemed appropriate for use slip critical joints. The AISC minimum load is calculated as

$$\phi R_n = \mu D_u h_{sc} T_b N_s$$

where represents the design shear strength (kips) for the connection, μ is the coefficient of slip (AISC value is taken as 0.5 for an unpainted blast cleaned surface known as class B), D_u is a multiplier that represents the ratio of the mean installed bolt pretension to the specified minimum bolt pretension (taken as 1.13 for general cases), h_{sc} is the hole factor (1 for standard hole, 0.85 for oversize holes), N_s is the number of slip planes (2 slip planes for these tests) and T_b is the minimum fastener tension (AISC value is 49 kips for 7/8 in. bolts). The AISC minimum load value is independent of the inclusion of filler plates. Figures 30, 31 and 32 show a line indicating the AISC design slip resistance for a slip critical joint. From the figures, a comparison was made between the AISC design slip resistance and the tested slip resistance for each thickness of fillers.

<u>Slip Loads for Standard Hole Tests</u>

Figure 43 summarizes the load at which the slip first occurred in the splice connection for standard test cases as well as the corresponding slip coefficient for each test. Previous filler testing by Lee and Fisher indicated that fillers up to a thickness of 1 in. had no effect on the slip load, while Yura. et al. showed a decrease in slip load from the addition of a 0.25 in. filler as well as a further load reduction for the addition of 3 x 0.25 in. fillers (1982). Tests from phase 1 displayed a similar contrast in slip effects where the one bolt tests showed no slip load decrease with the addition of fillers, yet the 3 bolt tests showed a decrease in slip load with an increase in filler thickness. 2001 testing from Sugiyama et al. showed an increase in slip coefficient values in high strength bolted connections with the insertion of fillers. The data in Figure 43 shows that the slip loads appeared to be relatively unaffected by the inclusion of fillers into the connection. A slight lowering of slip load can be seen, however the amount decrease was not nearly as prevalent as seen in the phase 1 testing. Overall, the girder splice testing reinforces the RCSC's conclusion that fillers with surface conditions similar to that of other joint components do not significantly affect the slip resistance of the joint.

Figure 43 shows that most of the slip coefficients for standard tests were in the range of 0.2 to 0.3. These values are similar to the phase 1 testing. Chapter 10 of the RCSC Design Guide for Bolts indicates that slip coefficient values for grit blasted surfaces (class B) are shown to be on the magnitude of 0.5. Because of this disconnect in values, specialized slip testing was conducted to complement the data found during the girder splice testing. These slip tests are discussed in later section. Since surface treatment as well as steel elements were identical for phase 1 and phase 2 testing, a similar explanation was proposed for the low values of slip coefficient.

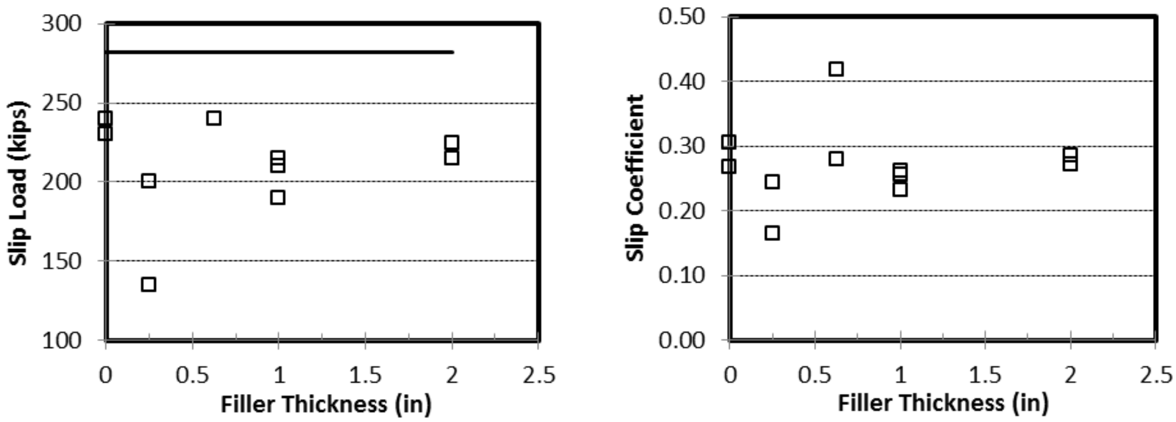


Figure 43: Slip loads (with the AISC slip critical line) and slip coefficients for standard tests.

Slip Loads for Multi-ply Tests

Figure 44 displays the results for slip loads and slip coefficients for the multi-ply cases. The slip loads seen in the multi-ply tests were comparable to those found for the standard cases. Most slip loads appeared in the range between 200 and 250 kips, with the accompanying slip coefficient values in the 0.2 to 0.3 range. A comparison shows that multi-ply fillers had little to no effect on the slip resistance of bolted girder connections.

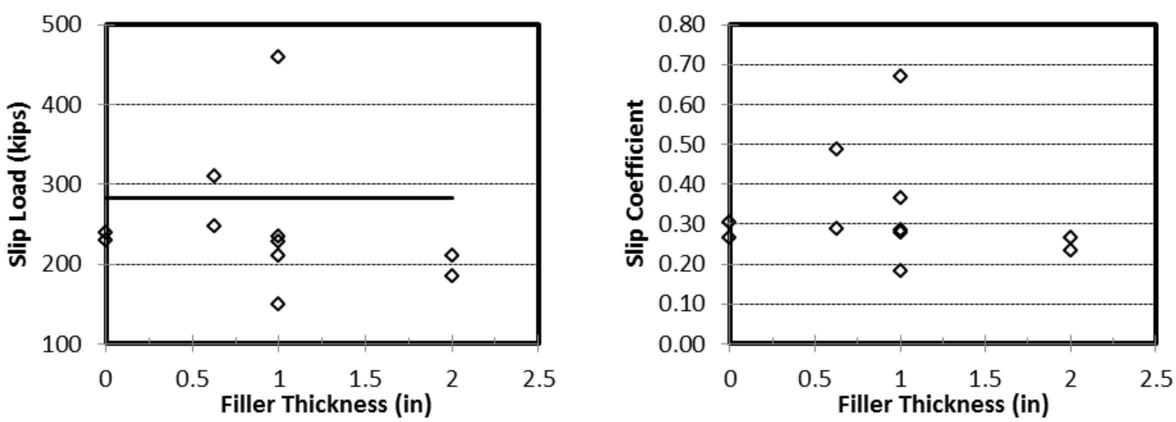


Figure 44: Slip loads (with the AISC slip critical line) and slip coefficients for multi-ply tests.

Slip Loads for Oversize Hole Tests

Figure 45 summarizes the results for slip loads and slip coefficients found for oversize tests. The results indicate that filler thickness appears have had little effect on slip load. Similar to the previous cases, the oversize tests saw slip coefficient values in the 0.2 to 0.3 range. Out of the three test cases, the slip coefficients calculated for the oversize tests were the lowest. The coefficients were on average 3.8 % lower than the standard cases. The slip loads (with the exclusion of the 1 in. fillers) also tended to increase as filler thickness was increased. A similar trend was found in phase 1 testing.

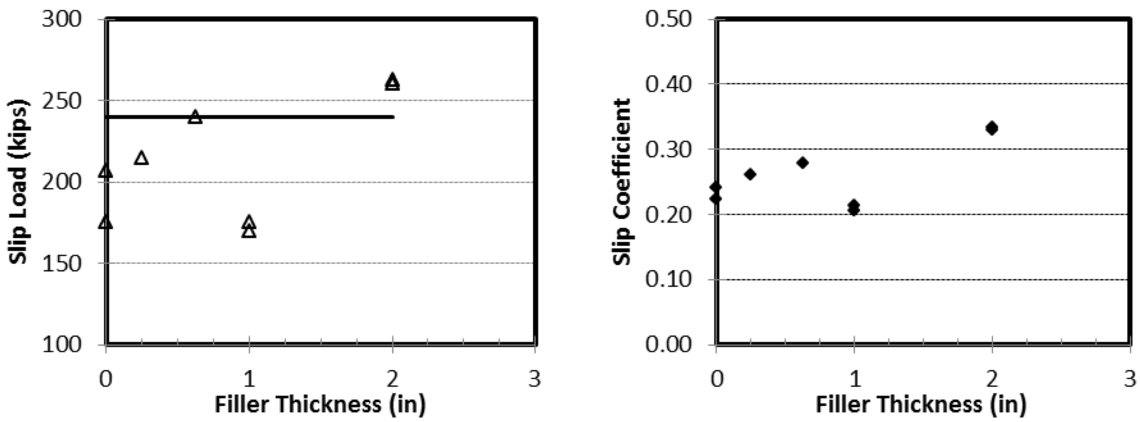


Figure 45: Slip loads (with the AISC slip critical line) and slip coefficients for oversize tests.

7.0 Slip Coefficient Testing

7.1 Introduction

In addition to the slip coefficient calculation using the girder splice test data, 24 compression slip tests were performed on various combinations of A709GR70 W (HPS) and A572GR50 W structural steel with an SP-10 surface treatment identical to that of the girders, splice plates and fillers utilized during the girder splice testing. This testing was done in order to supplement the results calculated from the girder testing and provide a more comprehensive analysis of the plate slip within each connection.

7.2 Test Setup

The dimensions of the steel specimen were selected such that the faying surface area would be the same as that of the specimen specified in the RCSC 2004 *Specifications for Structural Joints*. As illustrated in Figure 46, the coupons had a face that was 5 inches tall by 3 inches wide with a 15/16 inch hole in the center with respect to the width and 3 inches from the bottom with respect to the height (on center). Due to limitations on materials, the thickness of each A709GR70 W (HPS) specimen was 1 3/4 inch while the thickness of each A572GR50 W specimen was 5/8 inch.

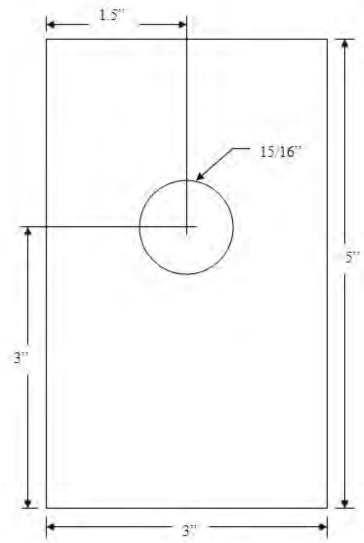


Figure 46: Geometry of coupons used in supplemental slip testing

The coupons were tested using an MTS vertical load frame which was controlled using an MTX 407 controller box. The configuration of the test setup (see Figure 47) was chosen to match the schematic specified by the RCSC (2004). To ensure that the axial force would be applied orthogonally to the clamping force and in order to prevent rotation a spherical head was placed between the actuator head and the specimen. The relative displacement between the coupons was measured using an MTS laser extensometer and the clamping force was measured using a pressure meter. All data was logged using the computer program Labview Data Logger.

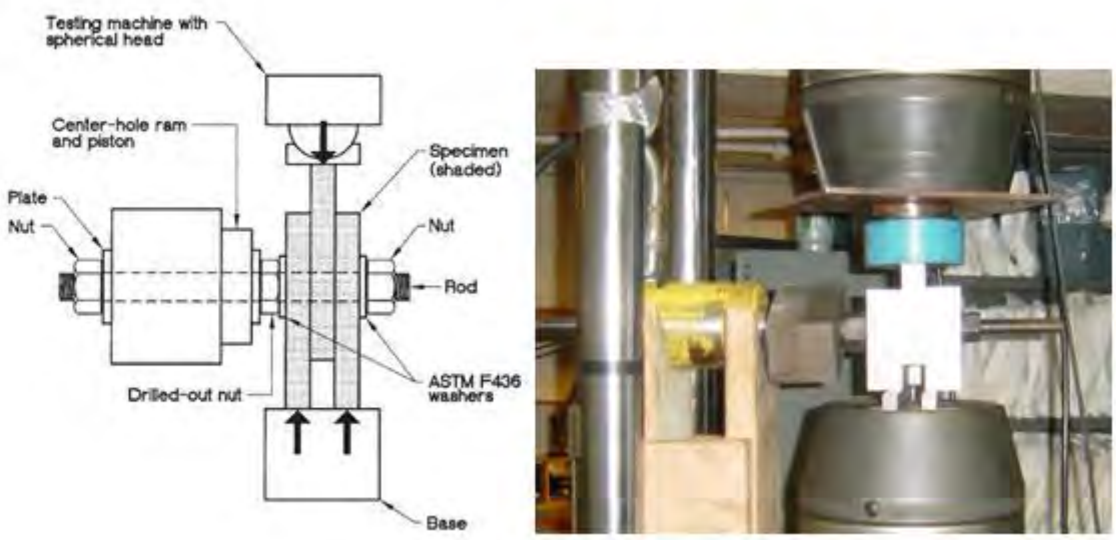


Figure 47: RCSC schematic of the slip test set up compared to actual set up.

7.3 Test Procedure

Prior to assembly the faying surfaces of the coupons were washed using a degreasing agent and a 3M scouring pad. A 7/8 inch diameter threaded rod capable of withstanding a tensile load of 125 kips without yielding was passed through the holes of the coupons which were secured up to a nut and washer against the left plate. A washer and 1.25 inch inner diameter nut were added to the rod and moved up against the right plate. The oversized nut was large enough that it could slide down the rod and was in place to simulate an actual bolted connection. Another washer was added then a hydraulic ram was secured with a final washer and a locking nut. All the washers were ASTM F436 washers and the nuts were DH A563's. The specimen was then loaded into an MTS vertical load frame such that the metal coupons were the only pieces in line with the load frame's actuator head; the hydraulic ram was supported by a frame such that the entire specimen remained orthogonal to the actuator head. The hydraulic ram had a pressure transducer attached on the hydraulic line which was used to determine the clamping force. The coupons were put into reverse bearing so that the greatest amount of distance could be traveled before bearing occurred against the rod and to prevent the rod from taking any shear load. The reverse bearing position was then secured by pressuring the system using the hydraulic ram. Shims were then placed on the bottom at the inner edges (in line with the faying surface) of the right and left coupons and on the top center of the center coupon. The spherical head was then placed on top of the specimen. Finally, the laser extensometer was put into position and initialized.

The extensometer, pressure transducer, and load cell from the load frame were connected to a data acquisition system and the readings were recorded using Labview Data Logger. The clamping force was increased to 49 kips and was maintained throughout testing within 0.5 kips. The rate of loading from the load frame was dictated by the RSCS guidelines which specify that the load placed on the specimen is not to exceed 25 kips per minute and the rate of slip is not to exceed 0.003 inches per minute. The loading rate was controlled using an MTX 407 controller box. The test was terminated when 0.05 in of slip was achieved.

7.4 Results of Slip Tests

Total of 24 tests were conducted to investigate the slip coefficient of the two types of metals used in the girder splice testing. Three different combinations (8 tests each) of coupons were tested as follows: combination 1 – HPS slipped against HPS, combination 2 – Grade 50 slipped against Grade 50, and combination 3 – HPS slipped against Grade 50. The test matrix with accompanying test results for slip loads and slip coefficients is found in Table 8.

The μ values obtained from the slip coefficient testing were analogous to the μ values calculated from the girder splice tests. For HPS to HPS slip tests, $\mu_{average}$ was equal to 0.308; for the Gr50 to Gr50 tests, $\mu_{average}$ was equal to 0.32; and for the Gr50 to HPS tests, $\mu_{average}$ was equal to 0.326. Figure 48 illustrates the comparison between slip coefficient values for all girder splice tests for different filler thicknesses, slip coefficient test values, and phase 1 tests (3 bolt). As previously discussed, the expected slip coefficient values for class B surfaces based upon previous research is around 0.5. Based upon all three phases of research conducted in the *i*STAR laboratory (phase 1 testing, girder splice testing and slip coefficient testing) the average slip coefficient for the steel used in the splice plates, filler plates and girders prepared as a class B surface was in the 0.26 to 0.32 range. There is no definite explanation for this disconnect in slip coefficient values, however the consistency of the slip data for all three phases suggests that the procedures used to for monitoring slip loads (similar steel, similar surface treatment, a known normal force via data from 'turn of the nut') were working correctly.

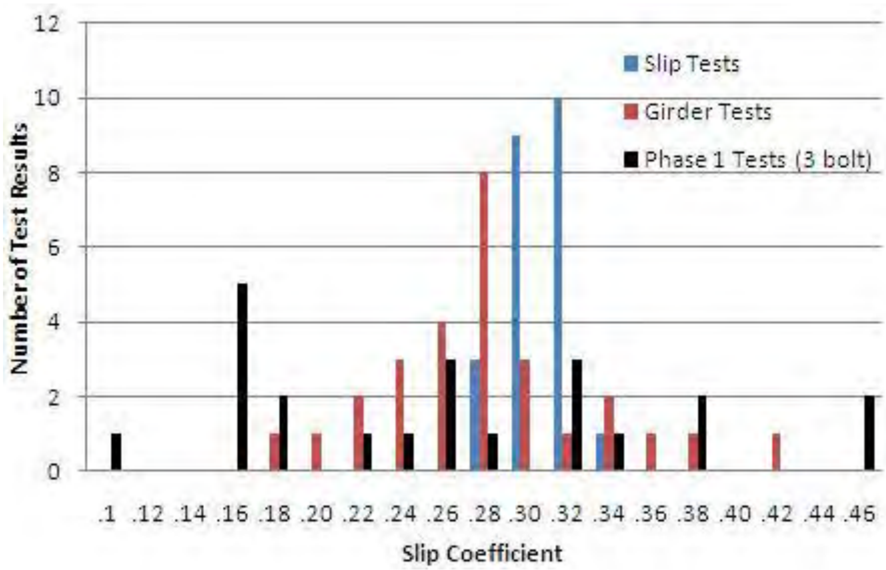


Figure 48: Frequency of slip coefficient values from slip coefficient tests, girder splice tests, and phase 1 tests (3 bolt)

8.0 Summary and Conclusions

In general, the introduction of fillers into a spliced girder connection induced a reduction in ultimate load and an increase in connection deformation. These characteristics are likely due to increased bolt bending as a result of a thicker filler plate. Both previous filler tests showed similar trends for fillers up to 1 in. thick. Pure tension tests (phase 1) found that increasing a filler from 1in. to 2 in. enabled the filler to act as a rigid form reducing the amount of allowed bolt bending, thus preventing increased connection deformation and a loss in ultimate load. Girder testing has revealed that this rebound in connection strength was not present, however a drop in connection deformation was observed for standard series tests. Girder testing has shown that ultimate loads in all test series decreased as larger thicknesses of fillers were used.

The deformation at failure for multi-ply and oversize series increased for all tests as filler plate thickness was increased to 2 in. The deformation at failure for the standard tests experienced a reduction in total deformation from the 1 in. filler test to the 2 in. filler test. This trend was also found in the phase 1 testing. The shear reduction of the bolts based upon loads reached at 0.25 in. deformation for standard hole girder tests also showed a slight strength increase from 1 in. filler tests to 2 in. filler tests. This again showed that the 2 inch filler in the girder testing was much less effective at providing a rigid form to prevent bolt deformation due to the difference in loading from phase 1 testing and girder splice testing.

The main detriment of oversized holes was found to be the allowance of increased bolt bending within the connection which was responsible for the loss recorded in ultimate strength as well as the recorded increase in connection deformation. The oversized hole connections had recorded displacements at failure that were nearly twice the size of standard hole tests on average. The oversized connections also reached 0.25 in. of deformation at loads close to the recorded slip loads. Since slip loads were found to be unaffected by standard or oversized hole use, the use of oversized holes in slip critical connections could be a viable design option; providing that the proper coefficient of slip was used to design the connection.

In a stark contrast to phase 1 testing, multi-ply filler test data proved to be comparable to standard size hole tests in terms of both ultimate load and connection deformation. Ultimate loads for multi-ply tests were seen to be less than 2 % lower than those reached for standard tests on average. Because the shear failure planes in the bolts during multi-ply tests were the same as those observed in the standard tests, there was no significant ultimate load reduction found in the bolts' shear strength. The deformation behavior of the multi-ply tests was found to be nearly 8 % larger than the deformation for standard tests. The additional deformation was likely due to the differential movement of individual filler plates within the connection allowing for slightly more bolt bending; however test data shows a load reduction of only 3 % for loads at 0.25 in. of deformation for multi-ply tests compared to standard tests.

Additional slip coefficient testing provided a more comprehensive slip analysis for the materials and surface treatment used during testing. Results from slip tests where normal force and slip force were continuously monitored proved to be comparable to the results recorded during girder splice tests as well as 3 bolt phase 1 tests. In all three tested cases, the majority (over 65 %) of slip coefficients were found to be in the 0.25 to 0.32 range. These slip coefficients are much lower than the value of 0.5 specified by the AISC and RCSC. The reasoning for this disconnect is not known with certainty, but the slip data from these 3 tests indicates that there may be an issue with the shot blasted surfaces of the A572Gr50 and HPSGr70 steel that was used.

Future experimental research into the effects of fillers in girder splice connections should consider the application of a measured shear force in the splice connection. All testing up to this point has been assumed to be tension loading with minimal (no) shear, thus it could prove beneficial to investigate the behavior of connections with an added shear force.

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APPENDIX – TABLES

Table 1: Test Matrix for Both Single and Multi-bolt Assembly Tests

Bolt Grade	Bolt Length (in)	Filler Thickness (in)	Hole Size	
A325	6	0 (mill scale)	standard	-
A325	6	½ (mill scale)	standard	oversize
A490	6	0 (mill scale)	standard	-
A490	6	0	standard	oversize
A490	6	½	standard	oversize
A490	6	2 x ¼	standard	-
A490	7	1	standard	oversize
A490	7	4 x ¼	standard	-
A490	9	2	standard	oversize

Table 2: Test matrix for girder splice tests.

Test	Holes Used	Filler Thickness 1 1/8" Flange	Filler Thickness 1 3/4" Flange	Test Series
1*	Inner Flange	No Filler Plate U (East)	No Filler Plate D (West)	Oversize
2	Outer Flange	7/8" D (East)	1/4" U (West)	Oversize
3**	Inner Flange	No Filler Plate U (East)	No Filler Plate D (West)	Oversize
4	Outer Flange	5/8" U (West)	No Filler Plate D (East)	Oversize
5	Inner Flange	1" U	3/8" D	Oversize
6	Outer Flange	1" U	3/8" D	Oversize
7	Inner Flange	2" U	1 3/8" D	Oversize
8	Outer Flange	2" U	1 3/8" D	Oversize
9*	Inner Flange	No Filler Plate U (East)	No Filler Plate D (West)	Standard
10	Outer Flange	7/8" D (East)	1/4" U (West)	Standard
11**	Inner Flange	No Filler Plate U (East)	No Filler Plate D (West)	Standard
12	Outer Flange	5/8" U (West)	No Filler Plate D (East)	Standard
13	Inner Flange	1" U	3/8" D	Standard
14	Outer Flange	1" U	3/8" D	Standard
15	Inner Flange	2" U	1 3/8" D	Standard
16	Outer Flange	2" U	1 3/8" D	Standard
17	Inner Flange	2 x 5/16" U	No Filler Plate	Multi-Ply
18	Outer Flange	2 x 5/16" U	No Filler Plate	Multi-Ply
19	Inner Flange	4 x 1/4" U	3/8" D	Multi-Ply
20	Outer Flange	4 x 1/4" U	3/8" D	Multi-Ply
21	Inner Flange	1/4" + 3/4" U	3/8" D	Multi-Ply
22	Outer Flange	1/4" + 3/4" U	3/8" D	Multi-Ply
23	Inner Flange	1/4" + 1 3/4" U	1 3/8" D	Multi-Ply
24	Outer Flange	1/4" + 1 3/4" U	1 3/8" D	Multi-Ply
25	Inner Flange	5/8" U	No Filler Plate	Standard
26	Outer Flange	7/8" D	1/4" U	Standard
27	Inner Flange	1" U	3/8" D	Standard
28	Outer Flange	4 x 1/4" U	3/8" D	Multi-Ply

* = 1.125 in. flange connected to 1.125 in. flange.
** = 1.75 in. flange connected to 1.75 in. flange.
D = Developed side of the connection
U = Undeveloped side of the connection.

Table 3: Bolt tension data from turn of the nut method using the Skidmore-Wilhelm device.

4.5 inch Bolt		Tension (kips)	Tension (kips)	Tension (kips)	Tension (kips)	Tension (kips)	Tension (kips)	Tension (kips)
Test Number		Snug	1/4 turn	1/2 turn	3/4 turn	1 turn	1 1/4 turn	1 1/2 turn
1	0	20	47	62.5	65.5	65.5	66	63
2	0	19	55	63.5	65.5	64	61.5	58
3	0	19	49	63.5	65.5	64.5	59.5	X
4	0	16	49	64	65.5	X	X	X
5	0	19	48	63	65	62	57	54
Avg.	0	18.6	49.6	63.3	65.4	64	61	58.33333

5 inch Bolt

Test Number		Snug	1/4 turn	1/2 turn	3/4 turn	1 turn	1 1/4 turn
1	0	18	55.5	69.5	71.5	69	64.5
2	0	18	55.5	71.5	71.5	69	69
3	0	18	48	68.5	71.5	68.5	68.5
4	0	20	51.5	68.5	73	72	68
5	0	17	52.5	68.5	71	70	64.5
Avg.	0	18.2	52.6	69.3	71.7	69.7	66.9

5.5 inch Bolt

Test Number		Snug	1/4 turn	1/2 turn	3/4 turn	1 turn	1 1/4 turn
1	0	19	48	66	68.5	67.5	64
2	0	19	50	68.5	69.5	66	62
3	0	20	50	68	69	66	62
4	0	19.5	53	67.5	68	65	62.5
5	0	17	51	66	67.5	64.5	61.5
Avg.	0	18.9	50.4	67.2	68.5	65.8	62.4

6.5 inch Bolt

Test Number		Snug	1/4 turn	1/2 turn	3/4 turn	1 turn	1 1/4 turn
1	0	15	26	48.5	65	67.5	65
2	0	14.5	34	59	65	66.5	64
3	0	16	35	61.5	66.5	66	64
4	0	15	38	59.5	66.5	65	62.5
5	0	15.5	34.5	61.5	66	64	61.5
Avg.	0	15.2	33.5	58	65.8	65.8	63.4

Table 4: Test specimen properties.

Component	Material Properties	Size	Surface
Bolts	A490	7/8 in. dia.	-
Filler Plates	A572 Grade 50	.25 to 2 in. thick (Std & Ovr size holes)	SP-10
Splice Plates	A709 GR70 W HPS	1.125 in. thick (Std & Ovr size holes)	SP-10
Girders	A709 GR70 W HPS	-	SP-10
Slip Test Blocks	A709 GR70 W HPS	1.75 in. thick (Std. holes)	SP-10
Slip Test Blocks	A572 Grade 50	.625 in. thick (Std. holes)	SP-10

Table 5: Deformation at failure and load at 0.25 in. deformation for all tests.

Test Series	Test Number	Equivalent Filler Thickness (in.)	Load at 0.25" Deformation (kips)	Deformation at Failure (in.)
Oversize	1	0	253	0.448
Oversize	2	0.25	482	0.299
Oversize	3	0	660	0.339
Oversize	4	0.625	283	0.519
Oversize	5	1	295	0.504
Oversize	6	1	360	0.415
Oversize	7	2	271	0.512
Oversize	8	2	277	0.582
Standard	9	0	768	0.203
Standard	10	0.25	683	0.174
Standard	11	0	758	0.197
Standard	12	0.625	650	0.225
Standard	13	1	620	0.298
Standard	14	1	491	0.313
Standard	15	2	600	0.232
Standard	16	2	582	0.275
Standard	25	0.625	734	0.32
Standard	26	0.25	726	0.29
Standard	27	1	628	0.19
Multi-Ply	17	0.625	713	0.241
Multi-Ply	18	0.625	661	0.242
Multi-Ply	19	1	650	0.266
Multi-Ply	20	1	630	0.295
Multi-Ply	21	1	622	0.297
Multi-Ply	22	1	628	0.314
Multi-Ply	23	2	560	0.31
Multi-Ply	24	2	471	0.343
Multi-Ply	28	1	590	0.245

Table 6: Ultimate loads for all tests.

Oversize Holes		Standard Holes		Multi-Ply	
Equivalent Filler Plate Thickness (in.)	Maximum Force (kips)	Equivalent Filler Plate Thickness (in.)	Maximum Force (kips)	Equivalent Filler Plate Thickness (in.)	Maximum Force (kips)
0	750	0	768	0	768
0	760	0	760	0	760
0.25	700	0.25	685	0.625	714
0.625	673	0.25	730	0.625	661
1	595	0.625	655	1	650
1	586	0.625	740	1	640
2	535	1	650	1	636
2	534	1	584	1	659
		1	628	1	640
		2	601	2	583
		2	605	2	600

Table 7: Measured load in the splice connection at the first occurrence of slip.

Oversize Holes			Standard Holes			Multi-Ply		
Equivalent Filler Plate Thickness (in.)	Force at First Slip (kips)	Coefficient of Slip, μ	Equivalent Filler Plate Thickness (in.)	Force at First Slip (kips)	Coefficient of Slip, μ	Equivalent Filler Plate Thickness (in.)	Force at First Slip (kips)	Coefficient of Slip, μ
0	175	0.223	0	240	0.306	0	240	0.306
0	207	0.241	0	230	0.267	0	230	0.267
0.25	215	0.262	0.25	200	0.243	0.625	310	0.487
0.625	240	0.279	0.25	135	0.164	0.625	248	0.288
1	175	0.213	0.625	240	0.279	1	460	0.669
1	170	0.207	0.625	360	0.418	1	210	0.365
2	260	0.329	1	215	0.262	1	235	0.286
2	263	0.333	1	190	0.231	1	228	0.277
			1	210	0.255	1	150	0.182
			2	215	0.272	2	210	0.266
			2	225	0.285	2	185	0.234

Table 8: Test matrix and results for slip coefficient testing.

Test	Slip Surface	Clamping Force "N" (kips)	Force at Slip "F" (kips)	Slip Coefficient "k"	Clamping Variance
HH1	HPS to HPS	48.43	32.2	0.3324	0.0563
HH2	HPS to HPS	48.80	28.0	0.2869	0.0720
HH3	HPS to HPS	48.75	28.5	0.2923	0.0581
HH4	HPS to HPS	49.52	28.1	0.2839	0.0866
HH5	HPS to HPS	49.19	32.6	0.3314	0.0740
HH6	HPS to HPS	49.91	30.2	0.3020	0.0663
HH7	HPS to HPS	49.47	32.0	0.3233	0.0952
HH8	HPS to HPS	49.51	31.4	0.3171	0.0765
GG1	Gr50 to Gr50	49.55	33.8	0.3412	0.0592
GG2	Gr50 to Gr50	49.61	30.0	0.3024	0.0943
GG3	Gr50 to Gr50	48.87	30.3	0.3096	0.1056
GG4	Gr50 to Gr50	49.47	33.5	0.3386	0.0886
GG5	Gr50 to Gr50	49.21	30.3	0.3079	0.0565
GG6	Gr50 to Gr50	49.86	32.6	0.3269	0.0522
GG7	Gr50 to Gr50	49.43	32.6	0.3297	0.0812
GG8	Gr50 to Gr50	49.96	30.4	0.3042	0.0948
GH1	Gr50 to HPS (HPS in middle)	49.66	31.7	0.3192	0.0718
GH2	Gr50 to HPS (HPS in middle)	49.59	30.8	0.3105	0.1599
GH3	Gr50 to HPS (HPS in middle)	49.33	32.9	0.3334	0.1261
GH4	Gr50 to HPS (HPS in middle)	49.93	32.1	0.3214	0.0989
GH5	Gr50 to HPS (Gr50 in Middle)	49.51	34.0	0.3433	0.1072
GH6	Gr50 to HPS (Gr50 in Middle)	49.81	33.8	0.3393	0.1030
GH7	Gr50 to HPS (Gr50 in Middle)	49.00	31.1	0.3174	0.0856
GH8	Gr50 to HPS (Gr50 in Middle)	49.16	32.1	0.3265	0.0955

Coordination of the Provisions of 2009 RCSC Specification for Structural Joints Using High-Strength Bolts With ANSI/AISC 360-2010, CSA S16-09 (Steel Structures), CSA S6-06 (Bridges) and AASHTO Specification

June, 2011

Committee members: Larry Kloiber, Tom Shlafly, Todd Ude, Gilbert Grondin, Larry Kruth, Peter Birkemoe, Greg Miazga (chair)

The format used in the first part of the report is to reference Sections from the 2009 RCSC Specification followed by comments from committee members regarding the 2010 AISC Specification and CSA S16-09. This is followed by comparisons with CSA S6-06 and AASHTO.

1.2. Loads, Load Factors and Load Combinations

The design and construction of the structure shall conform to an applicable load and resistance factor design specification for steel structures. Because factored load combinations account for the reduced probabilities of maximum loads acting concurrently, the *design strengths* given in this Specification shall not be increased. Appendix B is included as an alternative approach.

LK: The Task Group should evaluate if this format is still appropriate. Since the Canadian Specification is Limit States Design (LSD), changing RCSC to a Unified Format may not be the way to go - perhaps some provision to better accommodate the Unified Format may be possible.

GM: Future editions of S16 and the National Building Code of Canada (NBCC) are expected to be based on LSD principles only.

1.4. Drawing Information

The *Engineer of Record* shall specify the following information in the contract documents:

(1) The ASTM designation and type (Section 2) of bolt to be used;
(2) The *joint* type (Section 4);
(3) The required class of slip resistance if *slip-critical joints* are specified (Section 4); and,
(4) Whether slip is checked at the factored-load level or the service-load level, if *slip-critical joints* are specified (Section 5).

LK: Item 4 needs to be reviewed after revising Section 5. Since slip always is at service load in RCSC this is misleading. This whole section needs work.

GM: In S16-09, slip-critical joints are checked at the service load level only.

3.2.2. Slip-Critical Joints: The *faying surfaces* of *slip-critical joints* as defined in Section 4.3, including those of filler plates and finger shims, shall meet the following requirements:

(c) *Galvanized Faying Surfaces*: *Galvanized faying surfaces* shall first be hot dip galvanized in accordance with the requirements of ASTM A123 and subsequently roughened by means of hand wire brushing. Power wire brushing is not permitted. When prepared by roughening, the *galvanized faying surface* is designated as Class C for design.

LK: AISC has eliminated Class C based on new slip coefficient studies.

2010 AISC, J3.8(i) For Class A surfaces (unpainted clean *mill scale* steel surfaces or surfaces with Class A coatings on blast-cleaned steel or hot-dipped galvanized and roughened surfaces)

$$\mu = 0.30$$

GM: S16-09 has Class C (expected to change with next edition).

3.3.3. Short-Slotted Holes: When approved by the *Engineer of Record*, short-slotted holes are permitted in any or all plies of *snug-tightened joints* as defined in Section 4.1, and *pretensioned joints* as defined in Section 4.2, provided the applied load is approximately perpendicular (between 80 and 100 degrees) to the axis of the slot. When approved by the *Engineer of Record*, short-slotted holes are permitted in any or all plies of *slip-critical joints* as defined in Section 4.3 without regard for the direction of the applied load.

LK: Short slots permitted normal to direction of load unless prohibited by contract documents per AISC.

2010 AISC Spec, J3.2 - *Standard holes* or *short-slotted holes* transverse to the direction of the *load* shall be provided in accordance with the provisions of this specification, unless oversized holes, short-slotted holes parallel to the load, or *long-slotted holes* are approved

GM: S16-09 permits short slots normal to direction of load.

3.4 Burrs

Burrs less than or equal to 1/16 in. in height are permitted to remain on *faying surfaces* of all *joints*. Burrs larger than 1/16 in. in height shall be removed or reduced to 1/16 in. or less from the *faying surfaces* of all *joints*.

GM: S16-09 requires all burrs to be removed (expected to change with the next edition, to be consistent with the Specification).

4.2. Pretensioned Joints

Pretensioned joints are required in the following applications:

(1) *Joints* in which fastener pretension is required in the specification or code that invokes this Specification;
(2) *Joints* that are subject to significant load reversal;
(3) *Joints* that are subject to fatigue load with no reversal of the loading direction;
(4) *Joints* with ASTM A325 or F1852 bolts that are subject to tensile fatigue; and,
(5) *Joints* with ASTM A490 or F2280 bolts that are subject to tension or combined shear and tension, with or without fatigue.

LK: Items 2 & 3 differ from the AISC Appendix 3 which provides for an analysis of base metal fatigue for bolts not pretensioned. Item 4 is technically correct if an analysis shows fatigue controls. AISC does have a general provision that tensioning is required when fatigue or loosening is involved.

2010 AISC Spec, J3.1 Bolts are permitted to be installed to the snug-tight condition when used in:
(a) *bearing-type connections* except as noted in Section E6 or Section J1.10
(b) tension or combined shear and tension applications, for Group A bolts only, where loosening or *fatigue* due to vibration or *load* fluctuations are not design consideration.
Note: AISC in the Appendix on Fatigue in Table A3.1 Section 2 – Mechanically Fastened Joints – Permits bolted material that is not slip critical because base metal controls.

GM: S16-09 does have a general provision that pretensioning is required for connections subject to fatigue and/or tension.

4.3. Slip-Critical Joints

Slip-critical joints are required in the following applications involving shear or combined shear and tension:

(1) *Joints* that are subject to fatigue load with reversal of the loading direction;
(2) *Joints* that utilize oversized holes;
(3) *Joints* that utilize slotted holes, except those with applied load approximately normal (within 80 to 100 degrees) to the direction of the long dimension of the slot; and,
(4) *Joints* in which slip at the *faying surfaces* would be detrimental to the performance of the structure.

LK: Item 1 is probably good advice but base material can be designed per AISC Appendix 3 without using SC joints. Should this be a specification provision?

SECTION 5. LIMIT STATES IN BOLTED JOINTS

When slip resistance is required at the *faying surfaces* subject to shear or combined shear and tension, slip resistance shall be checked at either the factored-load level or service-load level, at the option of the *Engineer of Record*. When slip of the *joint* under factored loads would affect the ability of the structure to support the factored loads, the *design strength* determined in accordance with Section 5.4.1 shall be equal to or greater than the *required strength*. When slip resistance under service loads is the design criterion, the strength determined in accordance with Section 5.4.2 shall be equal to or greater than the effect of the service loads. In addition, slip-critical connections must meet the strength requirements to resist the factored loads as shear/bearing joints. Therefore, the strength requirements of Sections 5.1, 5.2 and 5.3 shall also be met.

LK: This dual system of using either "factored-load level or service-load level" is both confusing and outdated as far as terminology is concerned. The dual system for the same result should be eliminated. Provision should be made for LRFD or ASD loading within in the specification proper. (SEE Comment regarding S1) Commentary needs to make it clear the level of slip resistance that is actually being provided.
The underlined sentence is very important and needs study. If this is to be requirement, guidance should be provided as to when and how this is to be done. The 2005 AISC Specification attempted to provide method how to do this but really limited guidance on when it should be required. The 2010 Specification eliminated the "design at strength level" based on further study and modification of slip coefficients along with some conservative requirements for fillers.

Table 5.1. Nominal Strength per Unit Area of Bolts

Applied Load Condition		*Nominal Strength* per Unit Area F_n, ksi	
		ASTM A325 or F1852 Bolt	ASTM A490 or F2280 Bolt
Tension[a]	Static	90	113
	Fatigue	See Section 5.5	
Shear[a,b]	Threads included in shear plane		
	$L \leq 38$	54	68
	$L > 38$	45	56
	Threads excluded from shear plane		
	$L \leq 38$	68	84
	$L > 38$	56	70

[a] Except as required in Section 5.2.
[b] In shear *connections* that transmit axial force and have length (L), inches, between extreme bolt hole centers measured parallel to the line of force.

5.1. Design Shear and Tensile Strengths

Shear and tensile strengths shall not be reduced by the installed bolt pretension. For *joints*, the design shear and tensile strengths shall be taken as the sum of the strengths of the individual bolts.

The design strength in shear or the design strength in tension for an ASTM A325, A490, F1852 or F2280 bolt is ϕR_n, where ϕ= 0.75 and:

$$R_n = F_n A_b \quad \text{(Equation 5.1)}$$

GM: in S16-09 the ϕ factor is 0.80.

5.1. Design Shear and Tensile Strengths

(1) For fillers or shims that are equal to or less than ¾ in. thick, F_n from Table 5.1 shall be multiplied by the factor [1 - 0.4(t' - 0.25)], where t' is the total thickness of fillers or shims, in., up to ¾ in.;

(4) The *joint* shall be designed as a *slip-critical joint*. The slip resistance of the *joint* shall not be reduced for the presence of fillers or shims.

LK: TABLE 5.1 - AISC has modified the factor for connection length from 0.80 to 0.90. It is my understanding the Canadian Code uses an entirely different approach.

GM: S16-09 Clause 13.12.1.2 uses a connection length factor based on the bolt diameter (when the connection length exceeds 15 bolt diameter), but not less than 0.75. Grondin has proposed changes to this, likely to be incorporated into the next edition.

LK: comment to point (1) - Based on research AISC has revised Filler design requirements. The ¾" max limit has been eliminated. The reduction factor need not be less than 0.85 regardless of filler thickness.

2010 AISC Spec, J3.5.2 Fillers in Bolted Connections

When a bolt that carries *load* passes through *fillers* that are equal to or less than 1/4 in. (6 mm) thick, the shear strength shall be used without reduction. When a bolt that carries load passes through fillers that are greater than 1/4 in. (6 mm) thick, one of the following requirements shall apply:

(a) The shear strength of the bolts shall be multiplied by the factor

$$1 - 0.4(t - 0.25)$$

[S.I.: $1 - 0.0154(t - 6)$]

but not less than 0.85, where t is the total thickness of the fillers;

LK comment to RCSC note (4) – This note appears to conflict with previous requirement to check SC joints as shear-bearing joints. It seems to say that when checking SC joints as bearing connections you can ignore the presence of fillers

AISC now has a provision that fills can be developed with SC joints when using class B surfaces or turn-of-nut tensioning. This is based on the higher resistance to slip of these joints. It would be clearer if this section on fillers was only for shear-bearing connections.

The provision about fills not reducing the slip resistance of joints conflicts with the new AISC provisions for multiple fillers. Research needs to be done to clarify this issue.

2010 AISC Spec, J3.5(d) The joint shall be designed to prevent *slip* in accordance with Section J3.8 using either Class B surfaces or Class A surfaces with turn-of-nut tightening.

NOTE: This has to do with eliminating the need to check the SC joint for bearing.

Commentary 5.3 - The design bearing strength has been expressed as that of a single bolt, although it is really that of the connected material that is immediately adjacent to the bolt. In calculating the design bearing strength of a connected part, the total bearing strength of the connected part can be taken as the sum of the bearing strengths of the individual bolts.

2010 AISC Spec, J3.10 User Note: The effective strength of an individual *fastener* is the lesser of the fastener shear strength per Section J3.6 or the bearing strength at the bolt hole per Section J3.10. The strength of the bolt group is the sum of the effective strengths of the individual fasteners.

5.3. Design Bearing Strength at Bolt Holes

For *joints*, the design bearing strength shall be taken as the sum of the strengths of the connected material at the individual bolt holes.

The design bearing strength of the connected material at a standard bolt hole, oversized bolt hole, short-slotted bolt hole independent of the direction of loading or long-slotted bolt hole with the slot parallel to the direction of the bearing load is ϕR_n, where ϕ = 0.75 and:

(1) when deformation of the bolt hole at service load is a design consideration;

$$R_n = 1.2 L_c t F_u \le 2.4 d_b t F_u \quad \text{(Equation 5.3)}$$

(2) when deformation of the bolt hole at service load is not a design consideration;

$$R_n = 1.5 L_c t F_u \le 3 d_b t F_u \quad \text{(Equation 5.4)}$$

The design bearing strength of the connected material at a long-slotted bolt hole with the slot perpendicular to the direction of the bearing load is ϕR_n, where ϕ = 0.75 and:

$$R_n = L_c t F_u \le 2 d_b t F_u$$

GM: S16-09 does not have deformation of the bolt hole as a design equation consideration and the expression for resistance perpendicular to long slotted holes is slightly different.

5.4. Design Slip Resistance

μ = *mean slip coefficient* for Class A, B or C *faying surfaces*, as applicable, or as established by testing in accordance with Appendix A (see Section 3.2.2(b))
= 0.33 for Class A *faying surfaces* (uncoated clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel)
= 0.50 for Class B surfaces (uncoated blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)
= 0.35 for Class C surfaces (roughened hot-dip galvanized surfaces);

LK: The slip coefficients should be modified based on recent research to match AISC values of 0.30 for Class A and 0.50 for Class B and move galvanized to Class A pending *more research. Karl Frank recommended in Cleveland that RCSC should adopt these values and it should have at least been balloted in the 2009 Specification. These changes come from the research at U of Alberta by Grondin. This work should be reviewed and a proposal developed.*

2010 AISC Spec, J3.8 μ = mean slip coefficient for Class A or B surfaces, as applicable, and determined as follows, or as established by tests:

(i) For Class A surfaces (unpainted clean *mill scale* steel surfaces or surfaces with Class A coatings on blast-cleaned steel or hot-dipped galvanized and roughened surfaces)

$\mu = 0.30$

(ii) For Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)

$\mu = 0.50$

GM: S16-09 has Class A, B and C mean slip coefficients, and a table of coefficients to recognize the variability resulting from different bolt grades and types (and methods used to pretension bolts).

5.4. Design Slip Resistance

LK comment: RCSC does not account for multiple fillers in the joint

2010 AISC Spec h_f = factor for fillers, determined as follows:

(i) Where there are no fillers or where bolts have been added to distribute loads in the filler

$h_f = 1.0$

(ii) Where bolts have not been added to distribute the *load* in the filler:

(a) For one filler between connected parts

$h_f = 1.0$

(b) For two or more fillers between connected parts

$h_f = 0.85$

This is still subject to some research and should be evaluated further. The research at UIUC by Hajjar, UT Austin by Yura and Frank and Portland State by Dusicka should be reviewed (See AISC Commentary for J3.8 for more info.)

(1) In built-up compression members, such as double-angle struts in trusses, a small relative slip between the elements especially at the end *connections* can increase the effective length of the combined cross-section to that of the individual components and significantly reduce the compressive strength of the

strut. Therefore, the *connection* between the elements at the ends of built-up members should be checked at the factored-load level, whether or not a *slip-critical joint* is required for serviceability. As given by Sherman and Yura (1998), the required slip resistance is $0.008P_uLQ/I$, where P_u is the axial compressive force in the built-up member, kips, L is the total length of the built-up member, in., Q is the first moment of area of one component about the axis of buckling of the built-up member, in.^{3}, and I is the moment of inertia of the built-up member about the axis of buckling, in.^{4};

LK: This commentary has very important information but there is no specification section that covers this requirement. Is it necessary to run the above equation? AISC simply requires Class A surfaces and pretensioned bolts while the connection is designed as a shear-bearing connection. If more than this is required then both RCSC and AISC should put definite design requirements in the specification proper.

The following is a comparison summary between the RCSC 2009 Specification and CSA S6-06 (Canadian Highway Bridge Design Code), prepared by Gilbert Grondin:

Bolts in shear

RCSC 2009 – $R_n = F_n\ A_b = 0.9 \times 0.62 \times F_{ub}\ A_b$ for joints shorter than 38 in.

$R_n = F_n\ A_b = 0.75 \times 0.62 \times F_{ub}\ A_b$ for joints longer than 38 in.

CSA-S6-06 – $R_n = 1.0 \times 0.6 \times F_{ub}\ A_b$ for joints shorter than 30 in.

$R_n = 0.85 \times 0.6 \times F_{ub}\ A_b$ for joints longer than 30 in.

For threads in the shear plane, RCSC uses a reduction factor of 0.8, S6-06 uses a reduction factor of 0.7.

The resistance factor for bolt shear in RCSC is 0.75 while S6-06 uses $\phi = 0.8$

	RCSC 2009	S6-06
Short joints		
Joint length limit	38 in.	30 in.
No threads in shear planes	$\phi R_n = 0.42\ F_{ub}\ A_b$	$\phi R_n = 0.48\ F_{ub}\ A_b$
With threads in shear planes	$\phi R_n = 0.33\ F_{ub}\ A_b$	$\phi R_n = 0.34\ F_{ub}\ A_b$
Long joints		
No threads in shear planes	$\phi R_n = 0.35\ F_{ub}\ A_b$	$\phi R_n = 0.41\ F_{ub}\ A_b$
With threads in shear planes	$\phi R_n = 0.28\ F_{ub}\ A_b$	$\phi R_n = 0.29\ F_{ub}\ A_b$

Joints with fillers

RCSC 2009 – For bolts that carry load through fillers that are greater than ¼-in, but less than ¾-in the shear strength reduction factor is equal to $\left[1-0.4\left(t'-0.25\right)\right]$. Also has the option of developing the filler.

CSA-S6-06 – For fillers thicker than ¼-in, the fillers must be developed.

Bearing resistance

RCSC 2009 – $\phi R_n = \phi 1.5 L_c\, t\, F_u \leq \phi 3 d_b\, t\, F_u$ where $\phi = 0.75$

CSA-S6-06 – $\phi R_n = \phi 3 d_b\, t\, F_u$ where $\phi = 0.80$. The commentary states that the bearing capacity can be limited by end tearout, calculate using the block shear design equation.

$$\phi R_n = \phi 1.2 \left(\frac{F_y + F_u}{2} \right) L_e\, t$$

Slip resistance

RCSC 2009 – $\phi R_n = \phi \mu D T_m N_b$ where $\phi = 1.0$ for standard holes, = 0.85 for oversized and short-slotted holes, = 0.70 for long slotted holes loaded perpendicular to the slot, = 0.60 for long slotted holes loaded parallel to the slot. $D = 0.80$ to reflect the distribution of actual slip coefficient and the difference between the actual and nominal bolt pretension. $\mu = 0.33$, 0.50, 0.35 for Class A, B, and C surfaces, respectively.

CSA-S6-06 – $\phi R_n = \phi \mu D T_m N_b = c_1 k_s\, 0.53 A_b\, F_u\, N_b$. For regular size holes, the only difference between two equations is the value of c_1 and k_s. The value of c_1 varies from 0.78 to 0.90 and is equivalent to *D*. The slip coefficients are the same for Class A and B surfaces, but S6 uses a value of 0.4 for galvanized surfaces rather than 0.35.

The following is a comparison summary between the RCSC 2009 Specification and the AASHTO Specification, prepared by Todd Ude:

In support of Charlie Carter's initiative, and under Greg Miazga's leadership, I assembled this review of the present disposition of the RCSC Specification and RCSC activities in research, in relation to the various AASHTO standards and specifications for road and bridge construction. The exercise has reinforced to me how AASHTO and transportation engineering practice has historically created and operated under specifications and standards that are most accurately described as independent peer documents to the perhaps more familiar specs of AISC, ASTM, RCSC and others organizations.

Select Comparisons of AASHTO LRFD Bridge Design Specification and the RCSC Specification:

Here are two tables to give a sampling of the fit / lack-of-fit between RCSC and AASHTO specifications. This is far from exhaustive. It is intended to be enough of a summary to give a flavor of the agreements / disagreements, but short enough to get through in one reading. I think the upshot of the two tables is a recognition that the AASHTO has diverged from its peer specs in both implementation and in syntax to make any consideration of line-by-line synchronization unadvisable.

This first table addresses sections **other than** Section 5 – Limit States in Bolted Joints.

RCSC Section	RCSC Spec	AASHTO Spec
1	Refers to standards and specs from AISC and ASTM. Makes commentary reference to ASCE-7 for loads and load combinations	Refers to AASHTO's own material and testing specs. Defines its own loads and load combinations. In particular: 1.25 D1 + 1.5 D2 + 1.75 (LL+Imp) for strength checks 1.00 D1 + 1.0 D2 + 1.30 (LL+Imp) for slip 1.50 (LL+Imp) for infinite fatigue life 0.75 (LL+Imp) for finite fatigue life
3	Discussion of bolt holes covers standard, OS, short and long slots, with repeated references to the authority of the EOR.	Comparable (not identical) advice on types of holes and their applicability, but no reference to the authority of the EOR. Instead of "When approved by the EOR, oversized holes are permitted...", AASHTO will say "Oversized holes are permitted..."
4	RCSC defines "snug tight", "pre-tensioned", and "slip-critical" joint types	AASHTO recognizes "bearing" and "slip-critical" joint types. Bearing connections correspond to RCSC's snug tight and are restricted to joints in compression and joints in bracing members. All other connections are slip critical (load reversal / fatigue concerns).
App B	Service Load (ASD) design provisions	Service load design is being sunset. All new design is according the AASHTO LRFD provisions. Rehab or widening work on older structures designed under prior specifications (which included ASD provisions) may continue using those provisions.

The extension of this table to compare RCSC sections 7, 8 and 9 (Pre-Install Verification, Installation, and Inspection) against the AASHTO **Construction** spec (as opposed to its **Design** spec) would be interesting, but is beyond my available resources at the moment.

The following table is a more focused comparison of the criteria which actually govern the number of bolts designed into a connection, following Section 5 – Limit States in Bolted Joints. Still nothing resembling an exhaustive comparison. The wording of the two specs are too different to make such an exercise advisable.

RCSC Section	RCSC Spec	AASHTO Spec
5.2 Shear	Shear capacity based on tabulations of a fraction of Fu. Different Fu for A325 <, > 1" dia not recognized? Resistance factor $\phi = 0.75$	Shear capacity based on explicit multiplication of Fu and a fractional coefficient, with the Fu distinction for A325 explicit in code. The products come out to be slightly less than RCSC tabulated values. But: Resistance factor $\phi = 0.80$.
5.2 Shear	20% reduction in shear capacity for joints > 50" in length	Similar

RCSC Section	RCSC Spec	AASHTO Spec
5.2 Tension	Tension capacity based on tabulations of a fraction of Fu. Different Fu for A325 <, > 1" dia not recognized? Resistance factor ϕ = 0.75	Tension capacity based on direct multiplication of Fu by 0.76, with the Fu distinction for A325 explicit in code. Nominal capacities come out similar to RCSC tabulated values. And: Resistance factor ϕ = 0.80.
5.1 Shear & Tension	Elliptical interaction equation with commentary reference to a prior tri-linear approximation.	Elliptical formulation for tension capacity when shear exceeds a threshold value; neglect of shear effect on tension capacity when shear is below threshold.
5.5 Fatigue	Tabulated acceptable stress ranges for different cycle regimes.	An "infinite life" fatigue check uses stress range thresholds similar to RCSC's > 500,000 cycles check. Rather than the two lesser cycle ranges of RCSC, AASHTO implements a more direct S-N curve type of calculation for "finite life" regime.
N/A	RCSC spec does not codify treatment of prying action on bolt tension.	AASHTO prescribes prying amplification of bolt tension (3b/8a – t^3/20).
5.4 Slip	RCSC defines one slip capacity for factored loads and one for service loads	AASHTO formulates a single slip capacity, to be checked against the "Service II" load combination: 1.0 DL + 1.3 (LL+Imp).
	RCSC uses hole factor ϕ	AASHTO uses hole factor Kh – same values
	RCSC uses slip coefficient μ	AASHTO uses slip coefficient Ks – same values except 0.33 for Class C, not 0.35 as in the RCSC spec.
	RCSC uses specified pre-tension Tm	AASHTO uses specified pre-tension Pt – same values
	RCSC include number of bolts and predicts strength of connection	AASHTO predicts strength per fastener
	RCSC includes D, "probability factor"	AASHTO has no such factor
	RCSC includes reduction due to applied tension, including again the D factor	AASHTO describes a numerically comparable reduction due to applied tension, neglecting again any D factor. Also AASHTO separates it from the slip section, "hiding" it in the combined tension and shear discussion.
5.3 Bearing	"...where hole deformation is a concern", bi-linear check of bearing capacity in shear joints	Similar bi-linear limitation prescribed for all standard, OS and short-slot holes (phrased differently)
	"... where hole deformation is not a concern", a more lenient bi-linear check for bearing is given	No such check in AASHTO. i.e. hole deformation is always a concern
	"... long slots loaded perpendicular", a more stringent bi-linear check of bearing capacity	Similar bi-linear limitation for similar conditions (phrased differently)

AASHTO / RCSC "Interface Points"

Compared to the RCSC, or even AISC, AASHTO is kind of sprawling and loosely organized. In response to Greg's suggestion that we attempt to identify what direction our peer organizations (such as AASHTO) are heading, here are three apparent contact points within AASHTO.

Committee T-14, Structural Steel Design - Ed Wasserman, Chair This committee authors the section of the **design** specification dealing with steel structures and bolted connections. Limit states and resistance factors in the code have been tuned to work with specific load combinations and load factors defined elsewhere within the code. High-strength bolted connection criteria are in general agreement with the RCSC specification, but far, far from line-by-line agreement. Some representative examples of agreement / disagreement are discussed in Section 3 below. Based on the differences, it seems clear that near-line-by-line synchronization of RCSC and AASHTO specifications would not be possible, nor of much concern to T-14. From speaking with Ed Wasserman, I do not get the impression that the section of their steel design specification dealing with bolted connections arises very often as controversial or in need of further development. And when it does, as you can imagine it is competing with curved girder behavior, box girder design, steel structure stability, fatigue and fracture, and other such issues for a fraction of the T-14 committee's attention.

Committee T-4, Construction –Shoukry Elnahal, Chair This committee supervises assembly and maintenance of the bridge **construction** specification (with support from committees like T-14 on material-specific issues). I spoke it's chair about a couple years ago at the AASHTO annual meeting, asking him if installation, inspection, testing, etc. of bolted connections was an issue of much discussion within T-4. He indicated that it has not been a recurring or contentious issue during his tenure. I have not undertaken a section-by-section comparison of the RCSC spec with the AASHTO construction spec, but there is obviously common interests. The AASHTO spec, for example, describes the qualification of bolt-nut assemblies. It also sanctions installation methods including turn-of-nut, calibrated wrench, Alternative Designs (twist-offs), and DTI's. Given the attention these items receive annually in the RCSC meeting, there may well be differences between AASHTO and RCSC which are less intentional and more a result of organizational drift in the absence of a formal liaison.

Committee T-11, Research - Tom Domagalski, Chair T-11 entertains research proposals and statements of research need from the US states, reviews them, and forwards funding recommendations to a federal-level funding authority (the National Cooperative Highway Research Program). I have spoken with Tom about RCSC's research activities and our general openness to cooperation / collaboration. There's no obvious mechanism by which research funding would flow from something like NCHRB to RCSC. On the other hand, much of the research supported by RCSC is of interest to the transportation industry (Dusicka work on fills in bridge girder splices, Birkemoe work on twist-offs, Brahimi work on coating A-490's). We should probably discuss with RCSC members who are more in tune with the funding mill (e.g. Frank, Ricles) if and how RCSC research funds might be leveraged into larger projects, or how RCSC expertise might be brought to influence connection research undertaken at NCHRB (with or through AASHTO).

RCSC Research Committee Report

Jim Ricles, Chair Research Cmt

2011 RCSC Annual Meeting

Current Research Projects

- *Delayed Installation of ASTM F1852 Fasteners - 2nd Phase*
  University Of Toronto
  Dr. Peter C. Birkemoe
- *Effect of Material Characteristics and Surface Processing Variables on Hydrogen Embrittlement of Steel Fasteners*
  Salim Brahimi
  IBECA Technologies Corp.
- *Fatigue Resistance of High Strength Bolts in Tension*
  Professor Gilbert Y. Grondin
  University of Alberta
- *Effect of Fillers on Steel Girder Field Splice Performance*
  Professor Peter Dusicka
  Portland State University

Research Project Reports

Peter Dusicka- “Effect of Fillers on Steel Girder Field Splice Performance”.

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Final Report to
Research Council on Structural Connections

EFFECT OF FILLERS ON STEEL GIRDER FIELD SPLICE PERFORMANCE

by

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15 May 2012

Completed Research Project Reports

- Evaluation of the Current Resistance Factors for Bolted Connection Strength
- Effects of Head Size on the Performance of Twist-Off Bolts
- Qualification of Dacromet® for Use With ASTM A490 High-Strength Structural Bolts
- Installation Characteristics of ASTM F1852 Twist-Off Type Tension Control Structural Bolt/Nut/Washer Assemblies